ENERCON	CALCULATION COVER SHEET	CALC. NO. TXUT-001-FSAR-2.4.4-CALC-015
		REV. 1
		PAGE NO. 1 of 103
Title:	Brazos River Dam Failures Analysis for Comanche Peak Nuclear Power Plant Units 3 and 4.	**Client:** Luminant
		Project: MITS004 – Luminant COLA

Item	Cover Sheet Items	Yes	No
1	Does this calculation contain any open assumptions that require confirmation? (If **YES**, identify the assumptions)________		X
2	Does this calculation serve as an "Alternate Calculation"? (If **YES**, identify the design verified calculation.) **Design Verified Calculation No.**________		X
3	Does this calculation supersede an existing calculation? (If **YES**, identify the superseded calculation.) **Superseded Calculation No.**________		X

Scope of Revision: The calculation has been revised to include a qualitative analysis for upstream dams for existing and future conditions, account for maximum water surface elevation for antecedent reservoir conditions, include wind setup for dam failure, and include tailwater effects. Additionally, the downstream effects of the resulting flow from the De Cordova Bend Dam failure are no longer included in this calculation. The resulting flow is analyzed by separate calculation (TXUT-001-FSAR-2.4.3-CALC-012) by integration into the river analysis model.

Revision Impact on Results: The effect of the proposed Cedar Ridge Reservoir has changed controlling dam failure scenario. The controlling dam failure scenario is now the domino-type failure of Fort Phantom Hill Dam, Cedar Ridge Reservoir Dam, Morris Sheppard Dam, and De Cordova Bend Dam. In addition, the Lake Stamford Dam failure is included simultaneous with the Cedar Ridge Reservoir Dam failure. The resulting flow from De Cordova Bend Dam failure including the effects from upstream dam failures is 6,730,000 cfs.

Study Calculation	☐	**Final Calculation**	☒
Safety-Related	☒	**Non-safety-Related**	☐

(Print Name and Sign)

Originator: Kurt Roblyer *Kurt Roblyer*	**Date:** 06/03/2010
Design Verifier: Bryan Cline *Bryan A. Cline*	**Date:** 06/03/2010
Approver: Joseph Mancinelli *JMancinelli*	**Date:** 6/24/10

CALCULATION REVISION STATUS

REVISION	DATE	DESCRIPTION
0	August 25, 2008	Initial Issue
1		Revision 1 –Include qualitative analysis, account for maximum water surface elevation for antecedent conditions, include wind setup for dam failure, include tailwater effects.

PAGE REVISION STATUS

PAGE NO.	REVISION	PAGE NO.	REVISION
All	1		

APPENDIX REVISION STATUS

APPENDIX NO.	PAGE NO.	REVISION NO.	APPENDIX NO.	PAGE NO.	REVISION NO.
A	All	1	F	All	1
B	All	1	G	All	1
C	All	1	H	All	1
D	All	1	I	All	1
E	All	1			

Calculation Design Verification Plan:

Apply CSP Number 3.01, Revision 6, Section 4.5.a, Design Review Method and to include at a minimum:

1. Review the changes due to the initial review and determine if the calculation provides a reasonable estimate of the site's maximum water surface elevation for antecedent conditions to include wind setup for dam failure and tailwater effects.
2. Review the design methodology and determine if it is appropriate and correctly applied and is accurate.

(Print Name and Sign for Approval – mark "N/A" if not required)

Approver: Joseph Mancinelli J. Mancinelli	**Date:** 6/24/10

Calculation Design Verification Summary:

I have reviewed the Brazos River Dam Failures Analysis for Comanche Peak Nuclear Power Plant Units 3 and 4 calculation and have made the following conclusions:

1. The guidelines for the site coincident dam failure analysis calculation are appropriate and applicable for the determination of the maximum wind wave activity.
2. The calculation for the site coincident dam failure analysis report follows the correct procedures.
3. The site coincident dam failure analysis summary and calculations have been independently verified.
4. The Originator has addressed the recommendations given during the review process.

Based on the above summary, the calculation is determined to be acceptable.

(Print Name and Sign)

Design Verifier: Bryan Cline Bryan A. Cline	**Date: 06/03/2010**
Others:	**Date:**

Item	Cover Sheet Items	Yes	No	N/A
1	**Design Inputs** – Were the design inputs correctly selected, referenced (latest revision), consistent with the design basis and incorporated in the calculation?	X		
2	**Assumptions** – Were the assumptions reasonable and adequately described, justified and/or verified, and documented?	X		
3	**Quality Assurance** – Were the appropriate QA classification and requirements assigned to the calculation?	X		
4	**Codes, Standard and Regulatory Requirements** – Were the applicable codes, standards and regulatory requirements, including issue and addenda, properly identified and their requirements satisfied?	X		
5	**Construction and Operating Experience** – Have applicable construction and operating experience been considered?			X
6	**Interfaces** – Have the design interface requirements been satisfied, including interactions with other calculations?	X		
7	**Methods** – Was the calculation methodology appropriate and properly applied to satisfy the calculation objective?	X		
8	**Design Outputs** – Was the conclusion of the calculation clearly stated, did it correspond directly with the objectives and are the results reasonable compared to the inputs?	X		
9	**Radiation Exposure** – Has the calculation properly considered radiation exposure to the public and plant personnel?			X
10	**Acceptance Criteria** – Are the acceptance criteria incorporated in the calculation sufficient to allow verification that the design requirements have been satisfactorily accomplished?	X		
11	**Computer Software** – Is a computer program or software used, and if so, are the requirements of CSP 3.02 met?	X		

COMMENTS:

(Print Name and Sign)	
Design Verifier: Bryan Cline Bryan A. Cline	**Date: 06/03/2010**
Others:	**Date:**

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Appendices

1.0 Purpose And Scope

To determine the peak flow from failure of De Cordova Bend Dam due to the effects of a Probable Maximum Flood (PMF) coincident with assumed hydrologic failure of upstream dams.

2.0 Summary Of Results And Conclusions

The controlling dam failure scenario includes the overtopping domino-type failures of Fort Phantom Hill Dam, the proposed Cedar Ridge Reservoir Dam, Morris Sheppard Dam, and De Cordova Bend Dam. In addition, overtopping failure of Lake Stamford Dam is included simultaneous with the Cedar Ridge Reservoir Dam failure. The total breach flow from De Cordova Bend Dam to be transposed downstream without any attenuation to the confluence with the Paluxy River is 6,730,000 cfs.

3.0 References

1. American Nuclear Society, "American National Standard for Determining Design Basis Flooding at Power Reactor Sites," ANSI/ANS-2.8-1992, July 28, 1992.
2. Autodesk, AutoCAD LT software 2006.
3. Bentley Systems, Inc., FlowMaster computer software, Service Pack 3.
4. Brazos G Regional Water Planning Group, Brazos G Regional Water Planning Area, Initially Prepared 2011 Brazos G Regional Water Plan, March 2010.
5. Brazos River Authority, Website, http://www.brazos.org/, accessed February 2008.
6. Brunner, G.W., "HEC-RAS, River Analysis System Hydraulic Reference Manual," U.S. Army Corps of Engineers CPD-69, November 2002.
7. Chow, V.T., "Open Channel Hydraulics," McGraw-Hill, New York, 1959.
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9. Federal Energy Regulatory Commission, "Environmental and Public Use Inspection Report, Morris Sheppard (Possum Kingdom)", August 5, 1999.
10. Freese and Nichols, Inc., Brazos River Authority Morris Sheppard Dam Breach Analysis Report, September 2001.
11. Fugro-McClelland (Southwest), Inc., "Geotechnical Remedial Work, Morris Sheppard Dam, Palo Pinto County, Texas, Stability Analyses Report," September 24, 1993.
12. Llano Estacado Regional Water Planning Group, Llano Estacado Regional Water Planning Area, Initially Prepared Regional Water Plan, March 2010.
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17. Texas Water Development Board, Volumetric Survey of Hubbard Creek Reservoir, March 10, 2003.

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19. Texas Water Development Board, Volumetric Survey Report of Lake Granbury July 2003 Survey, September 2005.

20. Texas Water Development Board, Volumetric Survey of Lake Stamford, January 24, 2000.

21. Texas Water Development Board, Volumetric Survey Report of Possum Kingdom Lake December 2004-January 2005 Survey, May 2006.

22. U.S. Army Corps of Engineers, "Engineering and Design Hydrologic Engineering Requirements for Reservoirs," EM 1110-2-1420, October 31, 1997.

23. U.S. Army Corps of Engineers, "Flood Emergency Plans, Guidelines for Corps Dams," RD-13, June 1980.

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25. U.S. Army Corps of Engineers, National Inventory of Dams, Website, http://crunch.tec.army.mil/nidpublic/webpages/nid.cfm, accessed June 2006.

26. U.S. Department of the Interior, Bureau of Reclamation, Freeboard Criteria and Guidelines for Computing Freeboard Allowances for Storage Dams, ACER Technical Memorandum No. 2, December 1981.

27. U.S. Geological Survey, Water-Data Report 2008, 08088500 Possum Kingdom Lake near Graford, TX, Website, http://wdr.water.usgs.gov/, accessed May 2010.

28. U.S. Geological Survey, Water-Data Report 2008, 08090900 Lake Granbury near Granbury, TX, Website, http://wdr.water.usgs.gov/, accessed May 2010.

29. U.S. Geological Survey, Water-Data Report 2009, 08083500 Fort Phantom Hill Reservoir near Nugent, TX, Website, http://wdr.water.usgs.gov/, accessed May 2010.

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31. U.S. Geological Survey, Water-Data Report 2009, 08086400 Hubbard Creek Reservoir nr Breckenridge, TX, Website, http://wdr.water.usgs.gov/, accessed May 2010.

32. U.S. Geological Survey, Quadrangles, Website, http://www.usgs.gov, accessed April 2010.

33. U.S. Nuclear Regulatory Commission, "Combined License Applications for Nuclear Power Plants (LWR Edition)," Regulatory Guide 1.206, June 2007.

34. U.S. Nuclear Regulatory Commission, "Design Basis Floods for Nuclear Power Plants, Appendix B, Alternative Methods of Estimating Probable Maximum Floods," Regulatory Guide 1.59, August 1977.

35. U.S. Nuclear Regulatory Commission, "Standard Review Plan," NUREG-0800, March 2007.

36. U.S. Nuclear Regulatory Commission, "Flood Protection for Nuclear Power Plants", Regulatory Guide 1.102, September 1976.

37. U.S. Nuclear Regulatory Commission, "Standard Format and Content of Safety Analysis Reports for Nuclear Power Plants", Regulatory Guide 1.70, November 1978.

38. U.S. Nuclear Regulatory Commission, "Early Site Permits; Standard design Certifications; and Combined Licenses for Nuclear Power Plants", 10 CFR Part 52, August 2007.

39. U.S. Nuclear Regulatory Commission, "Industry Guidelines for Combined License Applicants under 10 CFR Part 52", NEI 04-05, October 2005.

4.0 Assumptions

Dam failures are assumed to occur by overtopping. Dam failures are assumed to occur coincident with the peak of the PMF. Failure during the PMF exceeds the regulatory guidance for hydrologic events coincident with seismic failure.

PMF estimates are determined by the alternative generalized methods described in Nuclear Regulatory Commission (NRC) Regulatory Guide 1.59 (RG 1.59) (Reference 34). Although RG 1.59 noted that results from this method are highly conservative, the RG 1.59 alternative methods were developed prior to the release of the most recent hydrometeorological reports. Therefore, there was some variation in the current degree of conservative nature.

The approach used in the calculation assumes failure of dams coincident with the PMF for a dam's respective watershed. The resultant breach flow is transposed downstream without attenuation.

The approach includes the conservative assumptions that the PMF for tributary dams and the Brazos River occur coincidentally, and that no attenuation of the flood flow is considered. The conservative nature of these assumptions is expected to outweigh any discrepancies between the RG 1.59 alternative methods to determine the PMF and the most recent hydrometeorological reports to determine the PMF.

Breach characteristics are assumed based on U.S. Army Corps of Engineers (USACE) RD-13 (Reference 23). When a range is provided, parameters are assumed to be at the more conservative end of the range.

Water surface elevation calculations at dams with gates assume the spillway gates are closed. Additionally, gated spillway and outlet capacities are assumed to be unavailable. These assumptions increase the overtopping depth resulting in greater breach flows.

Antecedent water surface elevations are assumed to be at the maximum historical water surface elevation. In some cases the antecedent water surface elevation is assumed to be at the spillway crest or dam crest, but only if this results in a higher elevation than the maximum historical water surface elevation.

The core wall in the Morris Sheppard Dam embankment section is assumed to fail during the embankment breach.

The total breach flow is assumed to be the sum of the overtopping flow and the breach flow. There is no reduction in overtopping flow to account for the breached section of the dam.

When determining tailwater elevations at the dam, level pool from the downstream cross section is assumed. This assumption neglects any increase to the tailwater elevation based on backwater effects. A lower tailwater elevation will maximize the water height component of the dam failure equation and the resulting dam failure flow.

5.0 Design Inputs

The qualitative analysis is performed using data for dams from the National Atlas (Reference 13), the USACE National Inventory of Dams database (Reference 25), information from the Somervell County Water District (Reference 15), the Brazos Regional Water Plan (Reference 4), the Llano Estacado Regional Water Plan (Reference 12), and spatial information using ESRI ArcGIS (Reference 8) data in AutoCAD (Reference 2).

The qualitative analysis is performed using NRC RG 1.59 Appendix B (Reference 34) figures of enveloping PMF isolines for drainage area sizes to determine the PMF.

Information for dams and reservoirs is from the National Atlas (Reference 13), USACE National Inventory of Dams database (Reference 25), Texas Water Development Board volumetric surveys (Reference 16 through Reference 21), USGS Water Year Reports (Reference 27 through Reference 31), and the following 7.5 minute USGS quadrangles (Reference 32):

Acton, TX
Antelope Hills, TX
Breckenridge, TX
Buck Mountain, TX
Collins Creek SW, TX
Crystal Falls, TX
Edwards Branch, TX
Eolian, TX
Fortune Bend, TX
Hamby, TX
Lake Stamford East, TX
Lake Stamford West, TX
Luenders East, TX

Luenders NE, TX
McCatherine Mountain, TX
Murphy Creek, TX
Nemo, TX
Nugent, TX

The 7.5 minute USGS quadrangles (Reference 32) are also used along with the Texas Water Development Board volumetric surveys (Reference 16 through 21) to determine wind setup characteristics. The wind speed is determined using input from ANSI/ANS-2.8-1992 (Reference 1).

Additional details for Morris Sheppard Dam are determined as follows:
The Brazos River Authority Morris Sheppard Dam Breach Analysis Report (Reference 10) is used to identify the total length of the concrete buttress section is 1,640 ft. The Federal Energy Regulatory Commission Environmental Use and Inspection Report (Reference 9) is used to identify the spillway length is 707 ft. with 9-73.6 ft. wide gates. The geotechnical stability analysis report (Reference 11) is used to identify the concrete core wall, which is 2 ft. wide and extends into the foundation.

6.0 Methodology

The listed design guidelines are used as both industry standard and compliance references for evaluating the potential dam failures coincident with PMF. All other procedures, instructions and design guides listed in section 5.4 of Project Planning Document (PPD No. TXUT-001, Rev. 2) are not specifically applicable to the Brazos River Dam Failures Analysis for Comanche Peak Nuclear Power Plant Units 3 and 4.

- U.S. Nuclear Regulatory Commission, "Standard Review Plan," NUREG-0800, March 2007 (Reference 35).
- U.S. Nuclear Regulatory Commission, "Design Basis Floods for Nuclear Power Plants, Appendix B, Alternative Methods of Estimating Probable Maximum Floods," Regulatory Guide 1.59, August 1977 (Reference 34).
- American Nuclear Society, "Determining Design Basis Flooding at Power Reactor Sites," ANSI/ANS-2.8-1992, July 28, 1992 (Reference 1).
- U.S. Nuclear Regulatory Commission, "Standard Format and Content of Safety Analysis Reports for Nuclear Power Plants", Regulatory Guide 1.70, November 1978 (Reference 37).
- U.S. Nuclear Regulatory Commission, "Flood Protection for Nuclear Power Plants", Regulatory Guide 1.102, September 1976 (Reference 36).
- U.S. Nuclear Regulatory Commission, "Combined License Applications for Nuclear Power Plants (LWR Edition)," Regulatory Guide 1.206, June 2007 (Reference 33).
- U.S. Nuclear Regulatory Commission, "Early Site Permits; Standard design Certifications; and Combined Licenses for Nuclear Power Plants", 10 CFR Part 52, August 2007 (Reference 38).
- U.S. Nuclear Regulatory Commission, "Industry Guidelines for Combined License Applicants under 10 CFR Part 52", NEI 04-05, October 2005 (Reference 39).

Qualitative analysis is provided to determine the critical dam failure scenarios to examine further by quantitative analysis. The qualitative analysis considers both existing conditions and future conditions. The qualitative analysis is performed based on a comparison of distance, reservoir storage, dam height, and drainage area. Quantitative analysis is provided to determine the critical dam failure scenario and the resulting flow for the Brazos River.

The potential effects from flooding on the Brazos River are examined based on the PMF coincident with assumed hydrologic dam failures. The PMF is based on the NRC RG 1.59 (Reference 34). RG

1.59 Appendix B charts along with the drainage area for the dam are used to select the PMF estimates that occur coincident with the dam failures.

The antecedent reservoir elevation prior to application of the PMF is based on the maximum historical water surface elevation. In the cases of Hubbard Creek Dam, Fort Phantom Hill Dam, and Lake Stamford Dam, the spillway or crest elevation is used and exceeds the maximum historical water surface elevation.

Domino-type failures and simultaneous failures are postulated when applicable. Dam failures are assumed to occur by overtopping. Overtopping is modeled using the standard broad crested weir flow equation defined by the HEC-RAS reference manual (Reference 6, Equation 6-14).

$Q = C * L * H^{1.5}$

Where:
Q = flow (cfs)
C = weir flow coefficient
L = weir length (ft.)
H = weir energy head (ft.)

In all cases, the equation is modified to solve directly for water surface elevation. The weir energy head, H, is replaced with (Z – E).

Where:
Z = water surface elevation
E = crest, spillway, or top of gates elevation

The HEC-RAS reference manual indicates that under free flow conditions, discharge is independent of tailwater. The weir flow coefficient may range from 2.5 to 3.1 as the coefficient increases with head depth. This calculation utilizes the HEC-RAS reference manual recommended weir flow coefficient of 2.6. As applied to the broad crested weir flow equation in this calculation, 2.6 is at the conservative end of the range. A lower weir flow coefficient will result in a higher overtopping headwater for a given flow rate. A higher overtopping headwater will result in greater breach flow.

Irregular shaped overtopping sections are also evaluated using a method of segments identified in the Federal Highway Administration guidance for roadway overtopping contained in Hydraulic Design Series Number 5 (Reference 14).

Tailwater depth is determined using FlowMaster (Reference 3) and the Manning friction formula based on the overtopping flow at a downstream cross section. The 7.5 minute USGS quadrangles are inserted into AutoCAD (Reference 2) to determine channel distances, slope, and cross section elevations. The Manning coefficient of 0.025 is applied to the channel and overbank areas. This is the minimum value for main stream and flood plain areas based on Chow (Reference 7). For the purpose of dam failure evaluation, it is more conservative to use a lower value because it results in a lower tailwater elevation. A lower tailwater elevation will maximize the water height component of the dam failure equation and the resulting dam failure flow.

In cases when it is determined the discharge is not independent of the tailwater, the effects on the weir flow coefficient are determined using the Federal Highway Administration guidance for roadway overtopping contained in Hydraulic Design Series Number 5 (Reference 14). The weir flow coefficient is reduced as necessary using the Hydraulic Design Series Number 5 (Reference 14) charts. A reduction to the weir flow coefficient is conservative and will increase the overtopping headwater elevation.

Wind setup is added to the overtopping elevation using the equation provided in USACE EM 1110-2-1420 (Reference 22, Equation 15-1).

$S = U^2 * F / (1{,}400 * D)$

Where:
S = wind setup (ft.)
U = average wind velocity over fetch distance (mph)
F = fetch distance (mi.)
D = average depth of water generally along the fetch line (ft.)

Wind setup is calculated in English units. The units for fetch and depth are not provided in the USACE EM 1110-2-1420 (Reference 22). However, in a technical memorandum by the U.S. Bureau of Reclamation (Reference 26), the same equation is referenced to determine wind setup. The units for fetch are indicated to be miles and the units for depth are indicated to be feet.

Wind speed is based on the guidance provided in ANSI/ANS-2.8-1992 (Reference 1, Section 9.2.1.1). The longest fetch distance is determined based on the maximum water surface elevation due to overtopping flows. The average depth of water is determined based on the bottom surface profile along the fetch distance.

The bottom surface profile along the fetch distance is created using the 7.5 minute USGS quadrangles (Reference 32) and bathymetry from reservoir volumetric reports (Reference 16 through Reference 21). An average depth along the fetch distance is determined using the formula for hydraulic depth.

$$E = \frac{\left(\frac{Y_1 + Y_2}{2}\right) * (X_2 - X_1) + \ldots + \left(\frac{Y_{n-1} + Y_n}{2}\right) * (X_n - X_{n-1})}{X_n - X_1}$$

Where:
E = average depth bottom surface elevation relative to overtopping water surface elevation (ft.)
Y_1 = elevation of first point along fetch line (ft.)
Y_2 = elevation of second point along fetch line (ft.)
Y_{n-1} = elevation of next to last point along fetch line (ft.)
Y_n = elevation of last point along fetch line (ft.)
X_1 = distance of first point along fetch line (ft.)
X_2 = distance of second point along fetch line (ft.)
X_{n-1} = distance of next to last point along fetch line (ft.)
X_n = distance of last point along fetch line (ft.)

Dam failure is evaluated based on two methods. A breach wave height is computed using the formula identified in ANSI/ANS-2.8-1992 (Reference 1, Section 5.1.3.2).

$h = 4 * (\text{headwater} - \text{tailwater}) / 9$

Where:
h = breach wave height

A dam failure flow is calculated using a USACE dam breach equation and breach parameters. HEC-HMS version 2.2.0 release notes (Reference 24, Page 8) are used to evaluate embankment failure that includes side slopes.

$$Q = 1.7 * W_b * h^{1.5} + 1.35 * S * h^{2.5}$$

Where:
Q = flow (cfs)
W_b = breach width (ft.)
h = water height
S = side slope horizontal component

The water height is equal to the smaller of the head difference between headwater and tailwater or the head difference between headwater and the breach bottom invert elevation.

Breach parameters for an earth fill dam are determined using USACE RD-13 (Reference 23, Table 1, Page 17). The greatest breach width and side slopes are used to maximize the resulting breach flow. The breach width, W_b, is three times the dam height, and the side slopes of the breach are 1:1 (horizontal:vertical).

Dam failure flow for concrete sections breach flow is determined using the USACE EM-1110-2-1420 (Reference 22, Page 16-3, equation 16-1) dam break equation.

$$Q = (8 / 27) * W_b * g^{0.5} * h^{1.5}$$

Where:
Q = flow (cfs)
W_b = breach width (ft.)
g = acceleration of gravity coefficient 32.2 ft/sec^2
h = water height (ft.)

It is noted that the concrete section dam break equation is equivalent to the embankment failure equation with vertical side slopes. The $(8 / 27) * g^{0.5}$ portion of the calculation solves to be 1.68, and when rounded to 1.7 is equivalent to the coefficient used in the embankment failure equation.

Breach parameters for concrete sections are determined using USACE RD-13 (Reference 23, Table 1, Page 17). For concrete gravity dams, the breach width is a multiple of the monolith widths. The side slopes are 0:1 (horizontal:vertical), which is equivalent to vertical. To maximize the resulting breach flow, entire concrete gravity sections are used rather than individual monolith widths.

The total breach flow is assumed to be the sum of the overtopping flow and the breach flow. There is no reduction in overtopping flow to account for the breached section of the dam. The breach flow and breach wave heights are transposed downstream without attenuation.

USGS quadrangles (Reference 32) and other locations and elevations discussed herein are relative to the Texas State Plane coordinate system, North American Datum of 1927, and National Geodetic Vertical Datum of 1929.

Excel mathematical functions are used throughout the calculation and provide results with the highest precision.

AutoCAD LT software 2006 (Reference 2) is used to determine spatial characteristics, such as length.

As previously identified, FlowMaster (Reference 3) is used to determine the tailwater elevation.

Excel, AutoCAD LT 2006, and FlowMaster software have been verified and validated in accordance with ENERCON's Corporate Standard Procedure Number 3.02, Revision 5, Control of Computer Software. The verification and validation documents are maintained by Enercon as part of the Quality Assurance program.

7.0 Calculations

Introduction

The NRC NUREG-0800 (Reference 35) and Regulatory Guide 1.206 (RG 1.206) (Reference 33) identify analysis of dam failure for hydrologic conditions. ANSI/ANS-2.8-1992 (Reference 1) defines the seismic dam failure combinations as the safe shutdown earthquake coincident with the peak of the 25-year flood, or the operating basis earthquake coincident with the peak of the one-half PMF or 500-year flood, whichever is less.

The PMF is a more extreme event than the hydrologic events combined with seismic dam failure. Therefore, seismic dam failure coincident with lesser flooding would result in lower flood elevations and has not been analyzed. Postulated dam failure does not assume that the dams are unsafe or will fail in the manner prescribed.

Qualitative Analysis

Existing Conditions

Potential dam failures have been considered for dams located in the Whitney Lake watershed. Whitney Lake Dam is located on the Brazos River approximately 56 river miles downstream from the confluence with the Paluxy River. The site is located on the Squaw Creek Reservoir (SCR) approximately 5 river miles upstream from the confluence of the Brazos River and the Paluxy River.

The distance from the confluence, reservoir storage, dam height, and drainage area are used as the basis for a qualitative assessment of dams to determine dam failure permutations that would warrant a quantitative assessment. Considering existing conditions, information for dams located in the Whitney Lake watershed has been obtained from the National Atlas (Reference 13), supplemented with information obtained from the U.S. Army Corps of Engineers National Inventory of Dams database (Reference 25), and is provided in Appendix A, Table A-1. Wheeler Branch Dam and the associated Paluxy River Channel Dam are recently completed structures and have not been included in the National Atlas. Data for these structures have been obtained from the Somervell

County Water District (Reference 15) and the 2011 Brazos G Regional Water Plan (Reference 4). The locations of the dams are shown on Appendix A, Figure A-1.

There are three dams (Lake Pat Cleburne Dam, Cleburne State Park Lake Dam, and Lake Virginia Dam) located upstream from Whitney Lake but downstream from the confluence. The total maximum storage capacity of the three dams is approximately 71,000 ac.-ft. Failure effects of these structures would continue downstream to Whitney Lake. Failure effects at the confluence from any combination of these structures would not exceed more critical dam failure permutations discussed below.

There are a number of dams located upstream of the confluence in the Paluxy River watershed. Including the recently completed Wheeler Branch Dam and associated Paluxy River Channel Dam, the total maximum storage capacity is approximately 42,000 ac.-ft. Failure effects at the confluence from any combination of these structures would not exceed more critical dam failure permutations discussed below.

Lake Granbury, formed by De Cordova Bend Dam, is the largest reservoir (136,823 ac.-ft. normal storage capacity and 240,640 ac.-ft. maximum storage capacity) in the immediate vicinity of the confluence and is located approximately 33 river miles upstream on the Brazos River. There are no other dams located on the Brazos between Lake Granbury and the confluence.

Possum Kingdom Reservoir, formed by Morris Sheppard Dam, is the largest reservoir (the normal and maximum storage capacity is listed as 556,220 ac.-ft.) immediately upstream from Lake Granbury. Morris Sheppard Dam is located on the Brazos River approximately 129 river miles upstream of De Cordova Bend Dam. Failure of Morris Sheppard Dam would enhance the postulated failure at De Cordova Bend Dam.

Upstream of Lake Granbury, Lake Palo Pinto Dam was also considered as a candidate that would enhance the postulated failure at De Cordova Bend Dam and the effects at the confluence. Although Lake Palo Pinto Dam is closer to Lake Granbury than Morris Sheppard Dam, Lake Palo Pinto (44,100 ac.-ft. normal storage capacity and 170,735 ac.-ft. maximum storage capacity) is significantly smaller. The quantitative assessment is based on breach flow and breach wave height and is dependent on the headwater and dam height. Additionally, the failure effects are transposed downstream without attenuation. The dam height of Morris Sheppard Dam is higher than Lake Palo Pinto Dam. Therefore, it would be more conservative to consider the added effects from Morris Sheppard Dam failure in the quantitative analysis. The other dams in the Brazos watershed between Morris Sheppard Dam and De Cordova Bend Dam do not exceed 20,000 ac.-ft. and were not considered further.

Upstream from Morris Sheppard Dam, there are seven dams (Graham Dam, Hubbard Creek Dam, Millers Creek Dam, Fort Phantom Hill Dam, Lake Stamford, John T. Montford Dam, and White River Dam) with reservoirs greater than 50,000 ac.-ft. Each of the seven dams is located on a separate tributary or multiple tributaries that precludes domino-type failure with dams other than Morris Sheppard Dam. Hubbard Creek Dam forms the reservoir with the greatest storage capacity (317,750 ac.-ft. normal storage capacity and 720,000 ac.-ft. maximum storage capacity), has the largest drainage area, and is located approximately 99 river miles upstream of Morris Sheppard Dam.

Only Graham Dam is located closer to Morris Sheppard Dam. However, even when considering the storage capacity of the reservoir formed by Eddleman Dam, which is connected to the reservoir formed by Graham Dam, the combined storage capacity is much less than the reservoir formed by Hubbard Creek Dam. Additionally, Hubbard Creek Dam has a greater dam height. Furthermore, the quantitative assessment failure effects are transposed downstream without attenuation. Therefore, it would be more conservative to consider the added effects from Hubbard Creek Dam failure in the quantitative analysis.

Only John T. Montford Dam has a dam height greater than Hubbard Creek Dam. However, John T. Montford Dam is approximately 351 river miles upstream from Morris Sheppard Dam, whereas Hubbard Creek Dam is 99 river miles upstream. Although the quantitative assessment does not consider attenuation, there would be significant attenuation over 351 river miles compared to 99 river miles if more rigorous methods were introduced. The Hubbard Creek Dam also has a greater drainage area of 1,107 sq. mi., whereas the John T. Montford Dam drainage area is only 394 sq. mi. The quantitative assessment includes the PMF flow for the local watershed, which is greater for the larger drainage area. The quantitative assessment does not attenuate the combined PMF and failure effects from the Hubbard Creek Dam. Therefore, it would be more conservative to consider the added effects from Hubbard Creek Dam failure in the quantitative analysis.

Hubbard Creek Dam is closer to Morris Sheppard Dam, has a greater dam height, has a larger drainage area, and has a greater storage capacity than Millers Creek Dam, Fort Phantom Hill Dam, Lake Stamford Dam, and White River Dam. Therefore, it would be more conservative to consider the added effects from Hubbard Creek Dam failure in the quantitative analysis. Considering existing conditions, the limiting dam failure permutation for additional quantitative analysis is the domino-type failure of Hubbard Creek Dam, Morris Sheppard Dam, and De Cordova Bend Dam.

Future Conditions

Future conditions have been considered based on the information provided in the 2011 Brazos G Regional Water Plan (Reference 4) and the Llano Estacado Regional Water Plan (Reference 12). There are nine alternatives in the Brazos G Regional Water Plan and available details are provided in Appendix A, Table A-2. There are three alternatives in the Llano Estacado Regional Water Plan and available details are provided in Appendix A, Table A-3. The locations of the potential sites for each alternative are shown on Appendix A, Figure A-1. Although potential sites are identified in the regional water plans, not all alternative potential sites are considered proposed dams as discussed below.

The Brazos G Regional Water Plan identifies sites to assess the potential for development in the Brazos River watershed. Some of the potential sites have not been identified as recommended water management strategies and are not considered to be proposed reservoirs because there are no intentions or actions to develop the potential sites. There have been no efforts to perform design work, identify budgets, procure necessary property, or execute any type of construction activity for the South Bend Reservoir, the two Double Mountain Fork Reservoir alternatives, the Lake Palo Pinto Off-Channel Reservoir, or the Throckmorton Reservoir. Therefore, these sites are not considered proposed reservoirs. Additionally, the two Double Mountain Fork Reservoirs are not concurrent alternatives. The plan identifies either the east or west alternative as a potential site, but not both.

The Turkey Peak Reservoir is a recommended water management strategy and is considered a proposed reservoir. The Turkey Peak Reservoir (22,577 ac.-ft. storage capacity) would be located approximately 3 river miles downstream from Lake Palo Pinto Dam. Turkey Peak Reservoir has been proposed to recover lost storage capacity of the reservoir formed by Lake Palo Pinto Dam due to sedimentation. A recent volume survey determined the reservoir storage capacity to be 63 percent of the normal capacity.

Turkey Peak Reservoir would have the same water surface elevation as the reservoir formed by Lake Palo Pinto Dam. Portions of the Lake Palo Pinto Dam would be removed to allow the two reservoirs to be connected at an upper elevation. Additionally, a pipe will connect the two reservoirs at a lower elevation. This configuration would reduce the failure effects of Lake Palo Pinto Dam compared to existing conditions because of the normal high tailwater on the downstream face of Lake Palo Pinto Dam. Although, the Turkey Peak Reservoir Dam would be higher than the Lake Palo Pinto Dam, the height would not be expected to exceed the height of Morris Sheppard Dam. Additionally, the combined storage capacity is much less than the storage capacity at Morris Sheppard Dam. Therefore, as previously discussed for the existing Lake Palo Pinto Dam, the failure effects from a combined Lake Palo Pinto Dam and Turkey Peak Reservoir Dam failure would not exceed the existing limiting dam failure permutation.

The Millers Creek augmentation is a recommended water management strategy and is considered a proposed alternative. The Millers Creek augmentation consists of a proposed diversion dam on Lake Creek and a proposed dam on Millers Creek approximately 4 river miles downstream of the existing Millers Creek Dam. Both structures are to be located upstream of Morris Sheppard Dam. The diversion dam is a low head structure only 8 ft. high and anticipated to maintain a small storage capacity. There are no downstream structures between the diversion dam and Morris Sheppard Dam. Therefore, dam failure of the diversion dam would not exceed the existing limiting dam failure permutation that includes Hubbard Creek Dam.

The new Millers Creek Dam would have a water surface elevation just 18 ft. below the existing Millers Creek Dam. Therefore, the new reservoir would back up to the existing dam, causing a normal high tailwater on the downstream face of the existing dam. This configuration would reduce the failure effects of the existing Millers Creek Dam compared to current conditions. The height of the new Millers Creek Dam would not be expected to exceed the height of Hubbard Creek Dam. Additionally, the combined storage capacity of the existing and new Millers Creek Dams is much less than the storage capacity at Hubbard Creek Dam. There are no downstream structures between the new Millers Creek Dam and Morris Sheppard Dam. Therefore, the failure effects from the combined existing and new Millers Creek Dam failures would not exceed the existing limiting dam failure permutation as previously determined.

The Cedar Ridge Reservoir is a recommended water management strategy and is considered a proposed reservoir. The Cedar Ridge Reservoir (227,127 ac.-ft. storage capacity) would be located on the Clear Fork of the Brazos River approximately 172 river miles upstream from Morris Sheppard Dam. Fort Phantom Hill Dam (70,036 ac.-ft. normal storage capacity and 127,000 ac.-ft. maximum storage capacity) is located approximately 41 river miles upstream from the proposed Cedar Ridge Reservoir on a tributary of the Clear Fork of the Brazos River. Domino-type failure of Fort Phantom Hill Dam and Cedar Ridge Reservoir Dam would enhance the postulated dam failure effects at Morris Sheppard Dam.

Furthermore, Lake Stamford Dam (57,927 ac.-ft. normal storage capacity and 150,000 ac.-ft. maximum storage capacity) is located about 10 miles to the northwest of Cedar Ridge Reservoir on Paint Creek, a tributary of the Clear Fork of the Brazos River. Although it is not located upstream from Cedar Ridge Reservoir, Lake Stamford Dam is also located approximately 170 river miles upstream from Morris Sheppard Dam. Simultaneous failure of Lake Stamford Dam and Cedar Ridge Reservoir Dam would also enhance the postulated dam failure effects at Morris Sheppard Dam.

The three alternatives from the Llano Estacado Regional Water Plan are all proposed to be developed in series on the North Fork Double Mountain Fork of the Brazos River. Lake 7 (20,700 ac.-ft. storage capacity) is proposed to be developed immediately upstream from McMillan Dam (4,200 ac.-ft. normal storage capacity and 8,280 ac.-ft. maximum storage capacity). Post Reservoir (56,000 ac.-ft. storage capacity) is proposed to be developed approximately 41 river miles downstream from McMillan Dam. Diversion Reservoir (1,000 ac.-ft. storage capacity) is proposed to be developed approximately 21 river miles downstream of Post Reservoir and just upstream of the confluence with the South Fork Double Mountain Fork of the Brazos River.

The three proposed reservoirs in conjunction with the existing reservoir formed by McMillan Dam contain relatively small storage capacities compared to the reservoir formed by John T. Montford Dam (115,937 ac.-ft. normal storage capacity and 354,500 ac.-ft. maximum storage capacity) on the South Fork Double Mountain Fork of the Brazos River. Considering domino-type failure of the three proposed structures and the existing McMillan Dam, there would be some attenuation between each successive failure. Because John T. Montford Dam contains a much greater storage capacity and is considered as previously discussed, the three proposed structures have not been considered further.

Considering future conditions, the limiting dam failure permutation for additional quantitative analysis is the domino-type failure of Fort Phantom Hill Dam, Cedar Creek Reservoir Dam, Morris Sheppard Dam, and De Cordova Bend Dam along with the simultaneous failure of Lake Stamford Dam.

Quantitative Analysis

Based on the qualitative analysis described above, there are two scenarios for further evaluation. Under existing conditions, the limiting scenario is the domino-type failure of Hubbard Creek Dam, Morris Sheppard Dam, and De Cordova Bend Dam. Under future conditions, the limiting scenario is the domino-type failure of Fort Phantom Hill Dam, Cedar Ridge Reservoir Dam, Morris Sheppard Dam, and De Cordova Bend Dam along with the failure of Lake Stamford Dam simultaneous with the failure of Cedar Ridge Reservoir. Both scenarios have the Morris Sheppard Dam and De Cordova Bend Dam failures in common. To determine the critical scenario, each limiting scenario is evaluated to determine the dam failure effects at Morris Sheppard Dam. The results are compared and the critical scenario is evaluated further to include the Morris Sheppard Dam and De Cordova Bend Dam failures.

Existing Conditions - Hubbard Creek Dam Failure Scenario

PMF Hubbard Creek Dam Watershed

Based on the NID database (Reference 25), Hubbard Creek Dam is located at the coordinates, latitude 32.8283° and longitude -98.9633°.

The NRC RG 1.59 Appendix B (Reference 34) method is used to determine the PMF for Hubbard Creek Dam. This method is based on the location of the site and utilizes charts of enveloping PMF isolines for various watershed drainage areas. Figure 7-1 is a typical chart showing the location of Hubbard Creek Dam. Table 7-1 presents the results for each chart corresponding to the drainage area provided. Straight line interpolation is used between isolines.

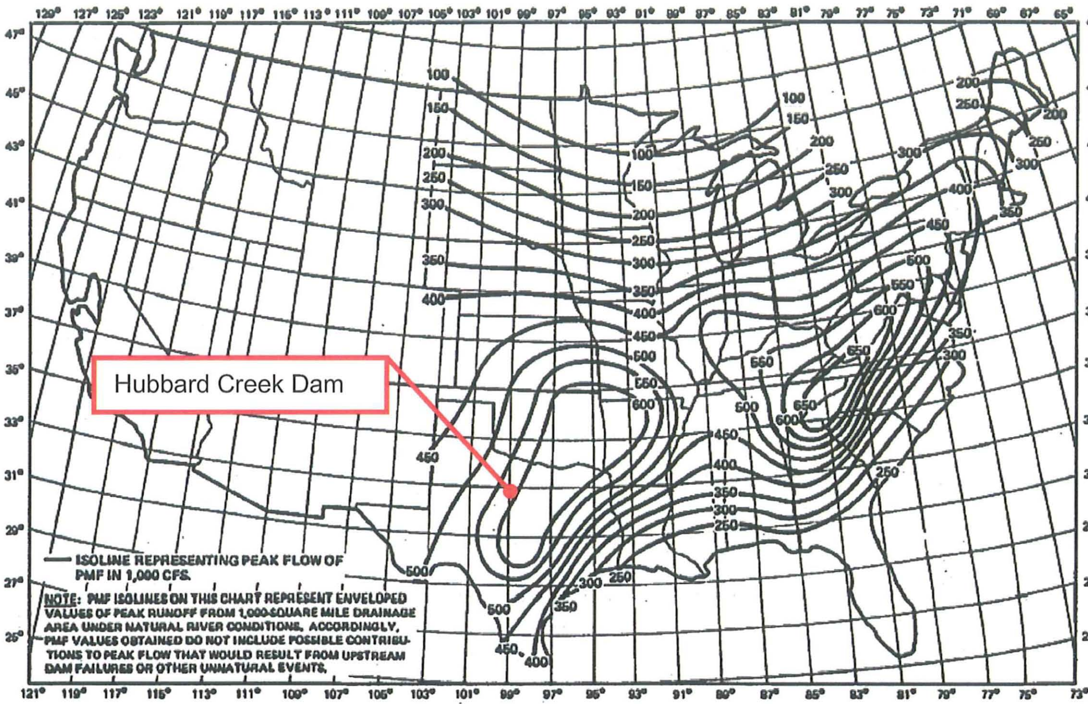

FIGURE B.4 PROBABLE MAXIMUM FLOOD (ENVELOPING PMF ISOLINES) FOR 1,000 SQUARE MILES

Figure 7-1. RG 1.59 1,000 sq. mi. Enveloping PMF Isolines Hubbard Creek Dam

Table 7-1. RG 1.59 Appendix B PMF Results for Hubbard Creek Dam

RG 1.59 chart	Drainage Area (sq. mi.)	PMF (cfs)
B2	100	190,000
B3	500	430,000
B4	1,000	580,000
B5	5,000	1,040,000
B6	10,000	1,330,000
B7	20,000	1,500,000

The results are plotted on a log-log scale and a smooth curve is fitted to the points, as shown in Figure 7-2. The drainage area for Hubbard Creek Dam is then used to determine the PMF for Hubbard Creek Dam.

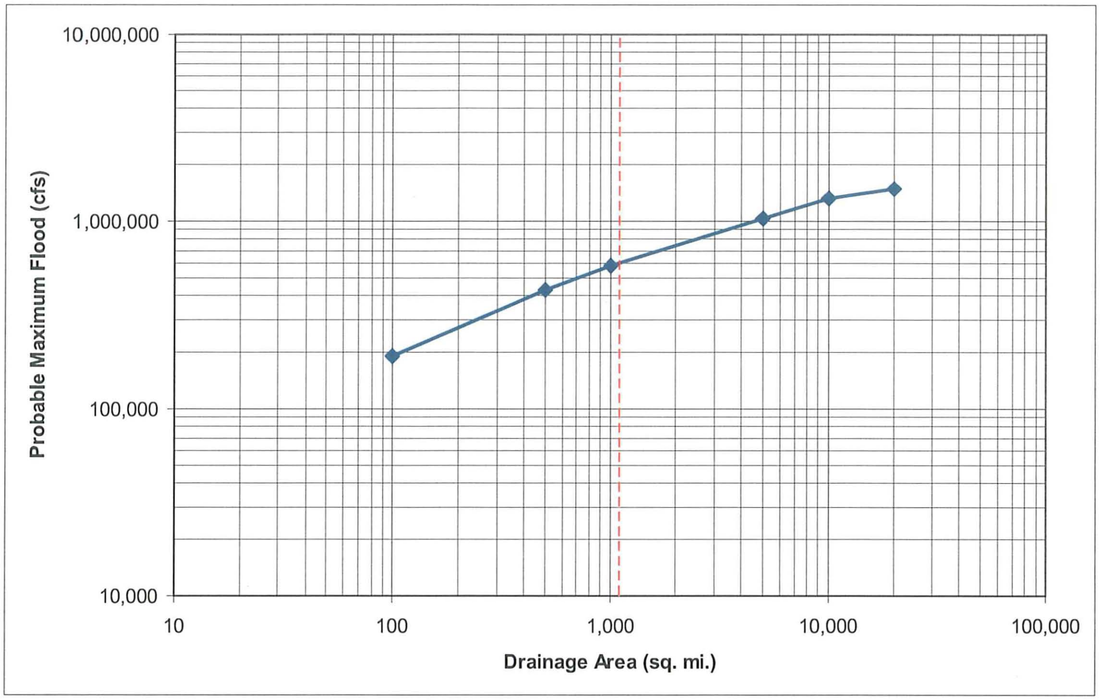

Figure 7-2. PMF Enveloping Isolines Based on Hubbard Creek Dam Location

According to the Texas Water Development Board volumetric survey (Reference 17), the Hubbard Creek Dam drainage area is 1,107 sq. mi. The vertical dashed line in Figure 7-2 is located at the corresponding drainage area value. The resulting PMF flow read from Figure 7-2 is 600,000 cfs.

The PMF is applied to Hubbard Creek Dam to determine the water surface elevation for dam failure analysis. According to the Texas Water Development Board volumetric survey (Reference 17), Hubbard Creek Dam is an earthfill embankment 15,150 ft. in length with a maximum height of 112 ft. or elevation 1,208.0 ft. The service spillway is a circular concrete drop inlet structure that is gate controlled. The crest elevation of the drop inlet is 1,176.5 ft. and the top of the gates is at elevation 1,185.0 ft. All water that enters the drop inlet is discharged through the embankment and exits downstream via a 22 ft. diameter conduit. The normal pool elevation is 1,183.0 ft. The emergency spillway is an excavated broad crested weir located near the left end of the dam. The 2,000 ft. long weir is at elevation 1,194.0 ft. Also, incorporated in the emergency spillway is a 4,000 ft. long fuse plug with a crest elevation of 1,197.0 ft.

According to the USGS gauge 08086400 Water-Data Report 2009 (Reference 31), the service spillway is designed to discharge 30,000 cfs. The maximum recorded elevation for the reservoir is 1,190.22 ft.

As the PMF is applied, it is assumed that the Hubbard Creek Reservoir is full up to the emergency spillway elevation of 1,194.0 ft., which exceeds the maximum recorded reservoir elevation. It is also assumed that the service spillway capacity is unavailable to accommodate any portion of the PMF, but is discharging the full 30,000 cfs to increase downstream flooding effects. Therefore, the water surface elevation is determined based on the PMF spilling over the emergency spillway, the fuse plug, and the dam crest as applicable. Overtopping is modeled using the standard broad crested weir flow equation defined by the HEC-RAS reference manual (Reference 6, Equation 6-14).

$Q = C * L * H^{1.5}$

As previously discussed in Section 6.0, this calculation utilizes the HEC-RAS reference manual recommended weir flow coefficient of 2.6.

Tailwater is determined based on the combined PMF and service spillway discharge flow of 630,000 cfs. The 7.5 minute USGS quadrangles (Reference 32) for Breckenridge, TX and Crystal Falls, TX are inserted into AutoCAD (Reference 2) to determine channel distances, slope, and cross section elevations. Figure 7-3 identifies the selected cross section in relationship to the dam and the channel distances used to determine the slope and elevations.

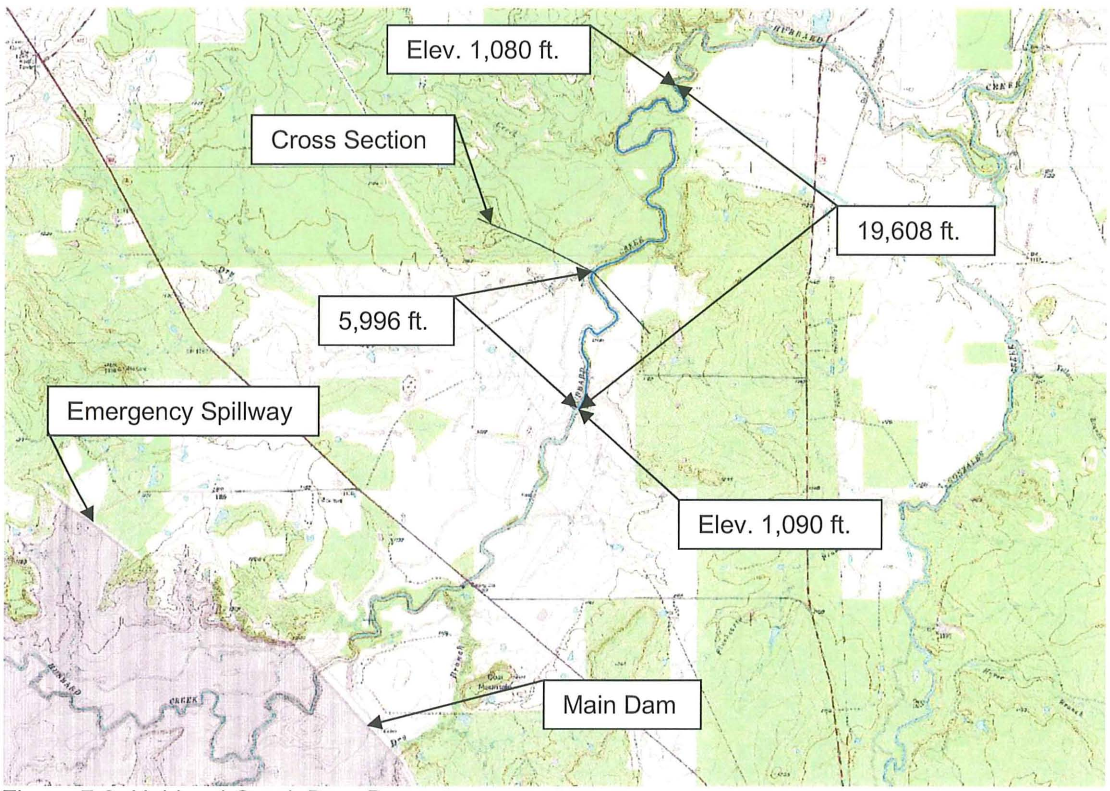

Figure 7-3. Hubbard Creek Dam Downstream

As shown in Figure 7-3, the channel drops 10 ft. over a distance of 19,608 ft. Therefore, the channel slope is 10 ft. / 19,608 ft. = 0.00051 ft./ft. The cross section is 5,996 ft. downstream from elevation 1,090 ft. Therefore, the cross section bottom is 5,996 ft. / 19,608 ft. * 10 ft. = 3 ft. lower than elevation 1,090 ft. The cross section station and elevations are provided in Table 7-2.

Table 7-2. Hubbard Creek Dam Tailwater Section Coordinates

Station (ft.)	Elevation (ft.)	Station (ft.)	Elevation (ft.)
-4,272	1,188	-75	1,090
-3,867	1,180	-34	1,087
-3,631	1,170	34	1,087
-3,465	1,160	59	1,090
-3,285	1,150	95	1,100
-3,078	1,140	140	1,110
-2,354	1,130	2,504	1,120
-1,898	1,120	2,535	1,130
-1,640	1,110	2,611	1,140
-114	1,110	2,656	1,150
-95	1,100	2,766	1,160

Stationing from left to right when looking downstream

Tailwater depth is determined using FlowMaster (Reference 3) and the Manning friction formula. From above the flow is 630,000 cfs and the slope is 0.00051 ft./ft. As previously discussed in Section 6.0, the Manning coefficient of 0.025 is applied to the channel and overbank areas.

The flow depth for the cross section is determined to be 41.7 ft. The FlowMaster results are provided in Appendix B. Therefore, the tailwater elevation at the downstream cross section is 1,087 ft. + 41.7 ft. = 1,128.7 ft. Level pool from the cross section upstream to the dam is assumed. This assumption neglects any increase to the tailwater elevation based on backwater effects. The tailwater elevation is well below the spillway elevation of 1,194 ft. Therefore, the spillway discharge is determined to be independent of tailwater. The cross section and tailwater elevation are shown on Figure 7-4.

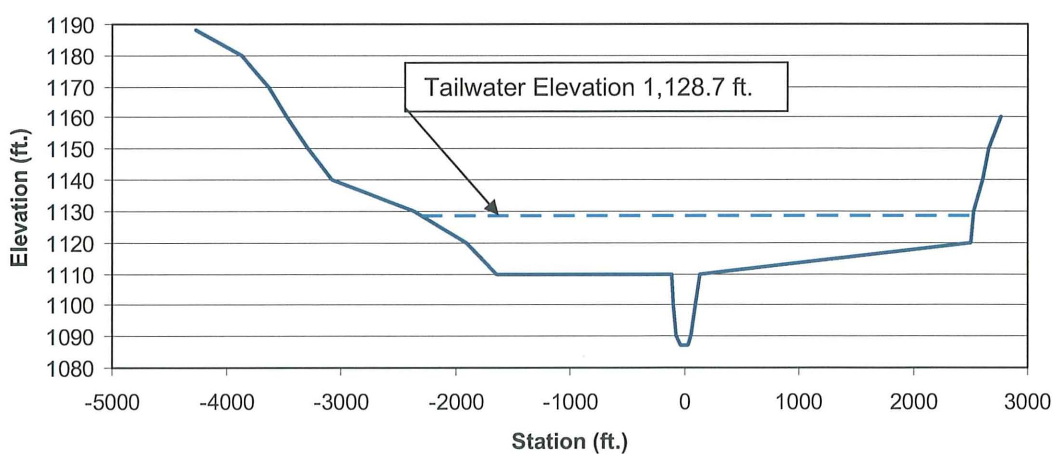

Figure 7-4. Hubbard Creek Dam Tailwater

As previously discussed in Section 6.0, the overtopping elevation is determined using the broad crested weir flow equation with a 2.6 weir flow coefficient. Based on the 2,000 ft. long emergency spillway at elevation 1,194 ft. and the 4,000 ft. long fuse plug at elevation 1,197 ft., the overtopping elevation is determined for the PMF of 600,000 cfs.

Q = 2.6 * emergency spillway length * (overtopping elevation – emergency spillway elevation)$^{1.5}$ + 2.6 * fuse plug length * (overtopping elevation – fuse plug elevation)$^{1.5}$

600,000 cfs = 2.6 * 2,000 ft. * (Z – 1,194 ft.)$^{1.5}$ + 2.6 * 4,000 ft. * (Z – 1,197 ft.)$^{1.5}$

Solving for overtopping elevation, Z = 1,207.36 ft. = 1,207.4 ft.

For the purpose of dam failure evaluation, it is conservative to round up because it results in a higher headwater elevation. A higher headwater elevation will maximize the water height component of the dam failure equation and the resulting dam failure flow. The determined headwater does not exceed the dam crest of 1,208 ft. Therefore, no additional consideration for overtopping at the main dam is necessary.

According to USACE EM 1110-2-1420 (Reference 22) wind setup can be reasonably estimated for lakes and reservoirs using the following equation:

$S = U^2 * F / (1,400 * D)$

USACE EM 1110-2-1420 (Reference 22) indicates that the fetch distance is usually satisfactorily assumed to be two times the effective fetch distance. A straight line fetch is used to define the wind setup and is more conservative than an effective fetch.

As referred to by regulatory guidance, ANSI/ANS-2.8-1992 (Reference 1, Section 9.2.1.1) indicates use of the 2-yr. wind speed applied in the critical direction for a combined probable maximum precipitation and coincident wind event or seismic dam failure and coincident wind event. ANSI/ANS-2.8-1992 (Reference 1, Section 9.1.4) permits the use of the Figure 7-5 to determine the 2-yr. wind speed in lieu of site specific studies.

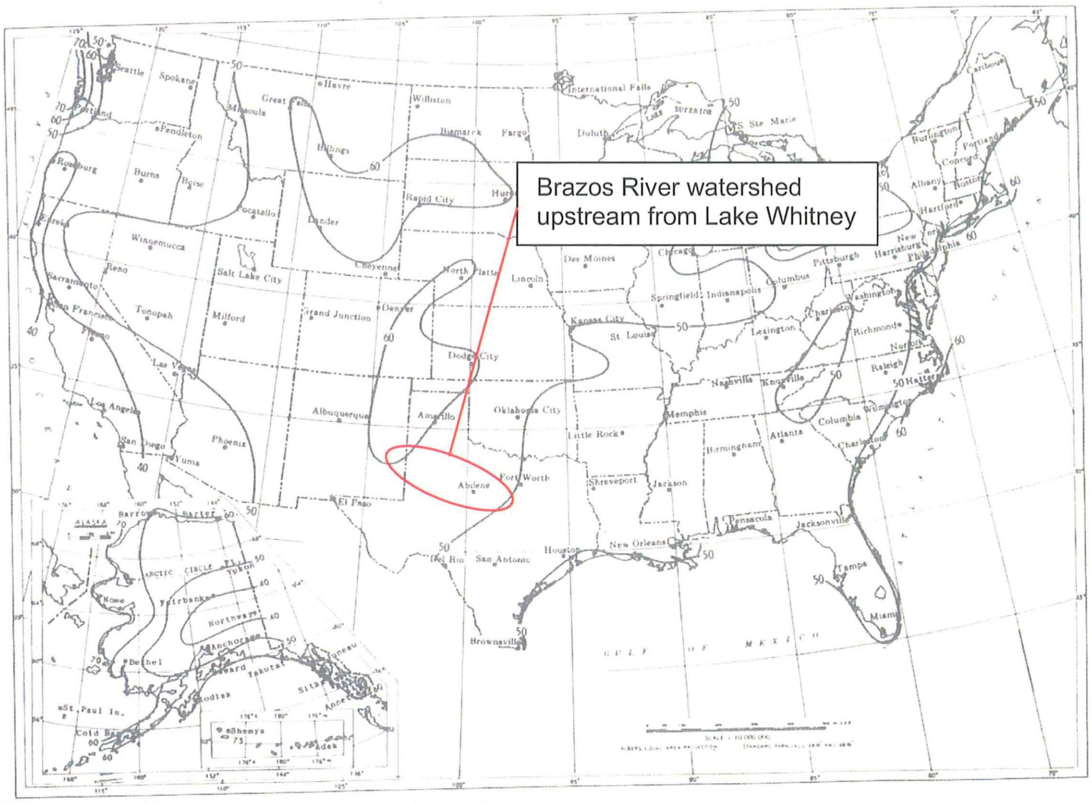

Figure 7-5. Wind Speed (Reference 1, pg. 31, Figure 1)

From Figure 7-5, the Annual Extreme-Mile, 30 ft. Above Ground, 2-yr. Mean Recurrence Interval is between 50 mph and 60 mph for the Brazos River watershed upstream from Whitney Lake. The more conservative wind speed of 60 mph is used to generate wind setup.

The overtopping elevation at Hubbard Creek Reservoir is determined to be 1,207.4 ft. The fetch length is determined from the reservoir surface area at the overtopping elevation. The 7.5 minute USGS quadrangles (Reference 32) for Breckenridge, TX, Buck Mountain, TX, Edward Branch, TX, Eolian, TX, McCatherine Mountain, TX, and Murphy Creek, TX are inserted into AutoCAD (Reference 2) and because only contours with 10 ft. intervals are identified on the quadrangles, the 1,210 ft. elevation is used to determine the surface area. As shown on Figure 7-6, the longest straight line fetch distance is determined to be 60,017 ft. (11.4 mi.).

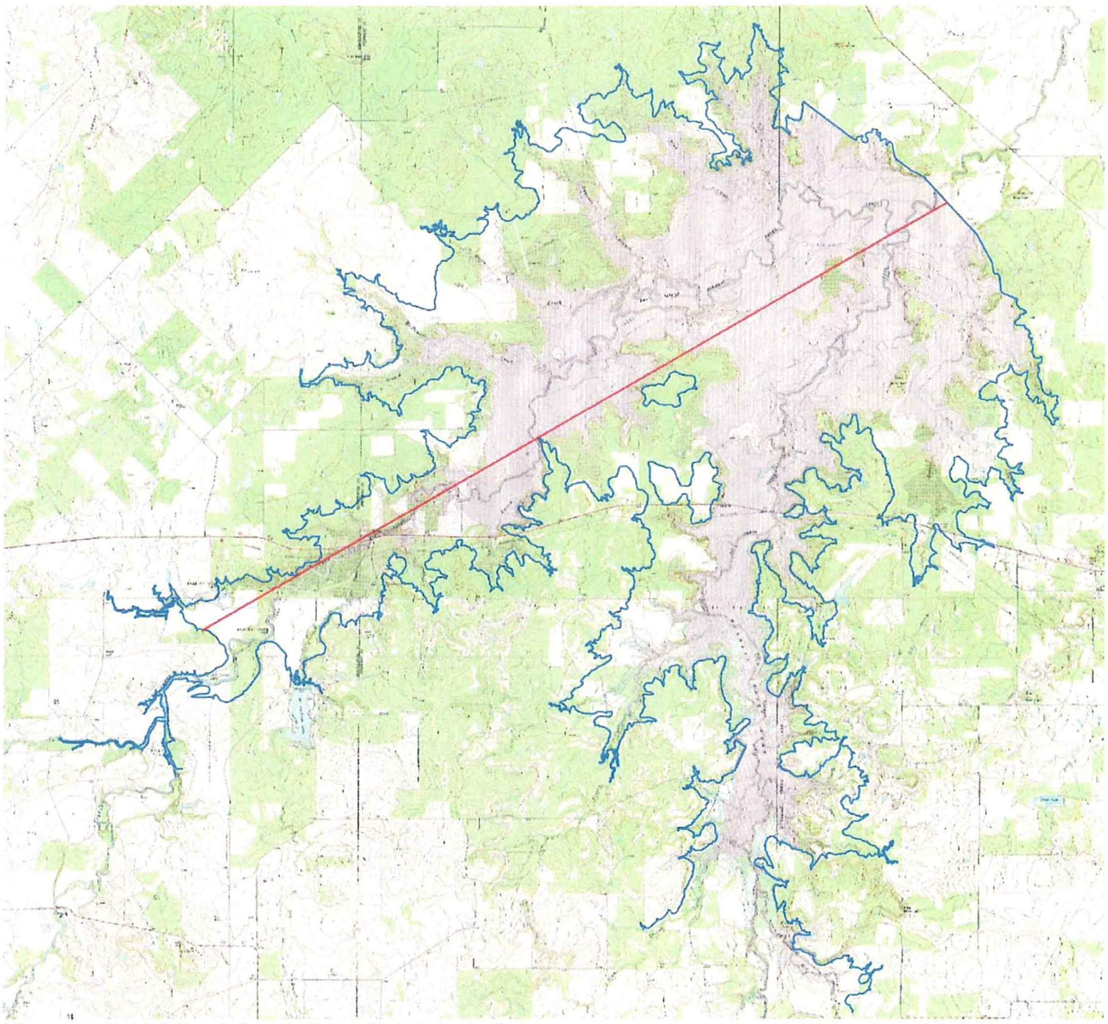

Figure 7-6. Hubbard Creek Dam Fetch Length

A bottom surface profile along the fetch distance is created using the USGS quadrangles (Reference 32) and is provided in Figure 7-7. The data for the distance and elevations are tabulated in Table 7-X. An average depth along the fetch distance is determined using the data in Table 7-3 and the following formula for hydraulic depth:

$$E = \frac{\left(\frac{Y_1 + Y_2}{2}\right) * (X_2 - X_1) + \ldots + \left(\frac{Y_{n-1} + Y_n}{2}\right) * (X_n - X_{n-1})}{X_n - X_1}$$

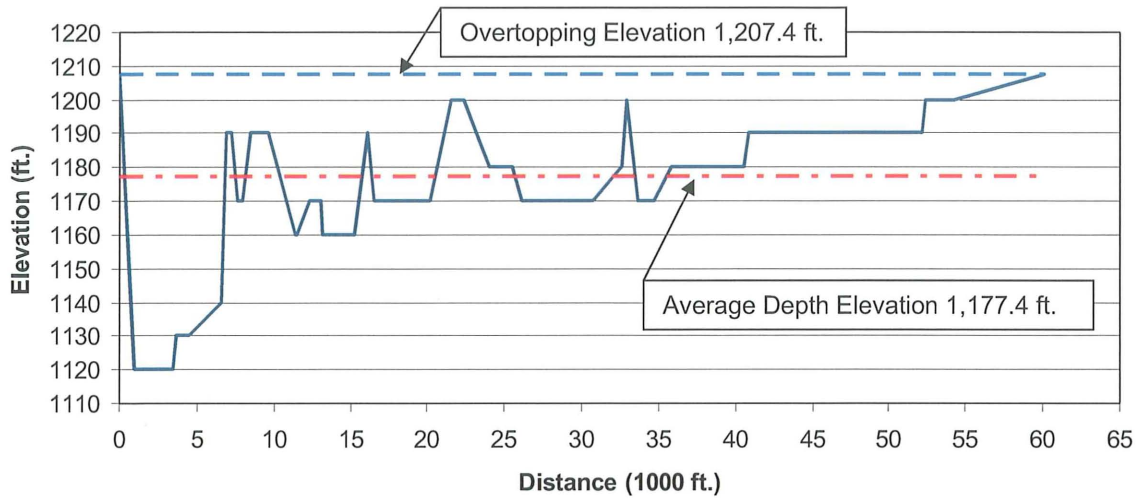

Figure 7-7. Hubbard Creek Dam Bottom Surface Profile

Table 7-3. Hubbard Creek Dam Bottom Surface Profile Section Coordinates

Distance (ft.)	Elevation (ft.)	Distance (ft.)	Elevation (ft.)	Distance (ft.)	Elevation (ft.)
0	1,207.4	11,538	1,160	26,087	1,170
942	1,120	12,377	1,170	30,677	1,170
3,491	1,120	13,099	1,170	32,651	1,180
3,699	1,130	13,209	1,160	32,931	1,200
4,480	1,130	15,296	1,160	33,619	1,170
6,586	1,140	16,076	1,190	34,702	1,170
6,871	1,190	16,122	1,190	35,846	1,180
7,237	1,190	16,511	1,170	40,590	1,180
7,656	1,170	20,176	1,170	40,873	1,190
7,928	1,170	21,496	1,200	52,170	1,190
8,476	1,190	22,362	1,200	52,363	1,200
9,565	1,190	24,048	1,180	54,195	1,200
11,423	1,160	25,515	1,180	60,117	1,207.4

Note: Distance 0 ft. is at the dam.

The average depth bottom surface elevation is calculated to be 1,177.4 ft. The overtopping water surface elevation is 1,207.4 ft. Therefore, the average depth along the fetch distance is calculated to be 1,207.4 ft. – 1,177.4 ft. = 30 ft. From above, the wind speed is 60 mph and the fetch distance is 11.4 mi. Wind setup is calculated as follows:

$$S = (60 \text{ mph})^2 * (11.4 \text{ mi.}) / (1{,}400 * 30 \text{ ft.}) = 0.98 \text{ ft.}$$

Setup is conservatively rounded up to 1.0 ft. For the purpose of dam failure evaluation, it is conservative to round up because it results in a higher headwater elevation. A higher headwater elevation will maximize the water height component of the dam failure equation and the resulting dam failure flow. The PMF headwater elevation at Hubbard Creek Dam including wind setup is 1,207.4 ft. + 1.0 ft. = 1,208.4 ft. and is based on a starting elevation exceeding the maximum historical elevation.

Dam Failure Hubbard Creek Dam

Dam failure is evaluated based on two methods. As identified in ANSI/ANS-2.8-1992 (Reference 1, Section 5.1.3.2) the breach wave height is computed as h = 4 * (headwater – tailwater) / 9 and transposed downstream without attenuation. Alternatively, dam failure flow is calculated using a USACE dam breach equation (Reference 24) and USACE RD-13 breach parameters (Reference 23).

Two failure scenarios are examined, embankment failure of the main dam using the full height of the dam and embankment failure of the fuse plug using the full height of the fuse plug. As identified above, the main dam is 112 ft. tall with a crest elevation at 1,208.0 ft. The headwater is determined to be 1,208.4 ft. and the tailwater is determined to be 1,128.7 ft. The breach wave height for the main dam is calculated as follows:

h = 4 * (1,208.4 ft. – 1,128.7 ft.) / 9 = 35.42 ft., rounded up to 35.5 ft.

Breach parameters for an earth fill dam are determined using USACE RD-13 (Reference 23, Table 1, Page 17). The greatest breach width and side slopes maximize the resulting breach flow. The breach width, W_b, is three times the dam height of 112 ft., and the side slopes of the breach are 1:1 (horizontal:vertical).

Therefore, W_b = 3 * height of dam = 3 * 112 ft. = 336 ft.

HEC-HMS version 2.2.0 release notes (Reference 24, Page 8) are used for the dam break equation including side slopes.

$Q = 1.7 * W_b * h^{1.5} + 1.35 * S * h^{2.5}$

As previously discussed in Section 6.0, the water height is equal to the smaller of the head difference between headwater and tailwater or the head difference between headwater and the breach bottom invert elevation. The difference between the headwater and tailwater is 1,208.4 ft. – 1,128.7 ft. = 79.7 ft. The difference between the headwater and breach bottom is greater than the full height of the dam, 112 ft. Therefore, the breach flow is calculated using the difference between the headwater and tailwater as follows:

$Q = 1.7 * 336 \text{ ft.} * (79.7 \text{ ft.})^{1.5} + 1.35 * 1 * (79.7 \text{ ft.})^{2.5}$ = 482,977 cfs, rounded up to 490,000 cfs.

The total flow is assumed to be the sum of the overtopping and spillway flow previously determined added to the breach flow. There is no reduction in overtopping flow to account for the breached section of the dam. Therefore, the total flow from the Hubbard Creek Dam failure is: Q = 630,000 cfs + 490,000 cfs = 1,120,000 cfs.

Based on the USGS quadrangles (Reference 32), the bottom of the fuse plug section is elevation 1,170 ft. The tailwater is well below the bottom and has no effect on the dam failure. As above, the headwater is determined to be 1,208.4 ft. The breach wave height for the main dam is calculated as follows:

h = 4 * (1208.4 ft. – 1170 ft.) / 9 = 17.06 ft., rounded up to 17.1 ft.

Breach parameters for the fuse plug are determined using USACE RD-13 (Reference 23, Table 1, Page 17). The greatest breach width and side slopes maximize the resulting breach flow. As identified above, the fuse plug is 4,000 ft. wide. The entire fuse plug is used as the breach width, W_b, and the side slopes of the breach are 1:1 (horizontal:vertical).

Because the tailwater is well below the bottom elevation of the fuse plug section, the water height is equal to the head difference between headwater and the breach bottom invert elevation. The difference between the headwater and the breach bottom invert elevation is 1,208.4 ft. – 1,170 ft. = 38.4 ft. Therefore, the breach flow is calculated using the difference between the headwater and the breach bottom as follows:

$Q = 1.7 * 4{,}000 \text{ ft.} * (38.4 \text{ ft.})^{1.5} + 1.35 * 1 * (38.4 \text{ ft.})^{2.5} = 1{,}630{,}437$ cfs, rounded up to 1,640,000 cfs.

The total flow is assumed to be the sum of the overtopping and spillway flow previously determined added to the breach flow. There is no reduction in overtopping flow to account for the breached section of the dam. Therefore, the total flow from the Hubbard Creek Dam failure is: Q = 630,000 cfs + 1,640,000 cfs = 2,270,000 cfs.

In summary, the potential scenarios for the Hubbard Creek Dam failure effects transposed downstream without attenuation to Morris Sheppard Dam are determined to be a main dam breach wave height of 35.5 ft., main dam breach flow of 1,120,000 cfs, fuse plug breach wave height of 17.1 ft., or fuse plug breach flow of 2,270,000 cfs. The main dam breach wave height and the fuse plug breach flow are the controlling failure scenarios for Hubbard Creek Dam.

Future Conditions - Cedar Ridge Reservoir Dam Failure Scenario

PMF Lake Stamford Dam Watershed

Based on the NID database (Reference 25), Lake Stamford Dam is located at the coordinates, latitude 33.0717° and longitude -99.56°.

The NRC RG 1.59 Appendix B (Reference 34) method is used to determine the PMF for Lake Stamford Dam. This method is based on the location of the site and utilizes charts of enveloping PMF isolines for various watershed drainage areas. Figure 7-8 is a typical chart showing the location of Lake Stamford Dam. Table 7-4 presents the results for each chart corresponding to the drainage area provided. Straight line interpolation is used between isolines.

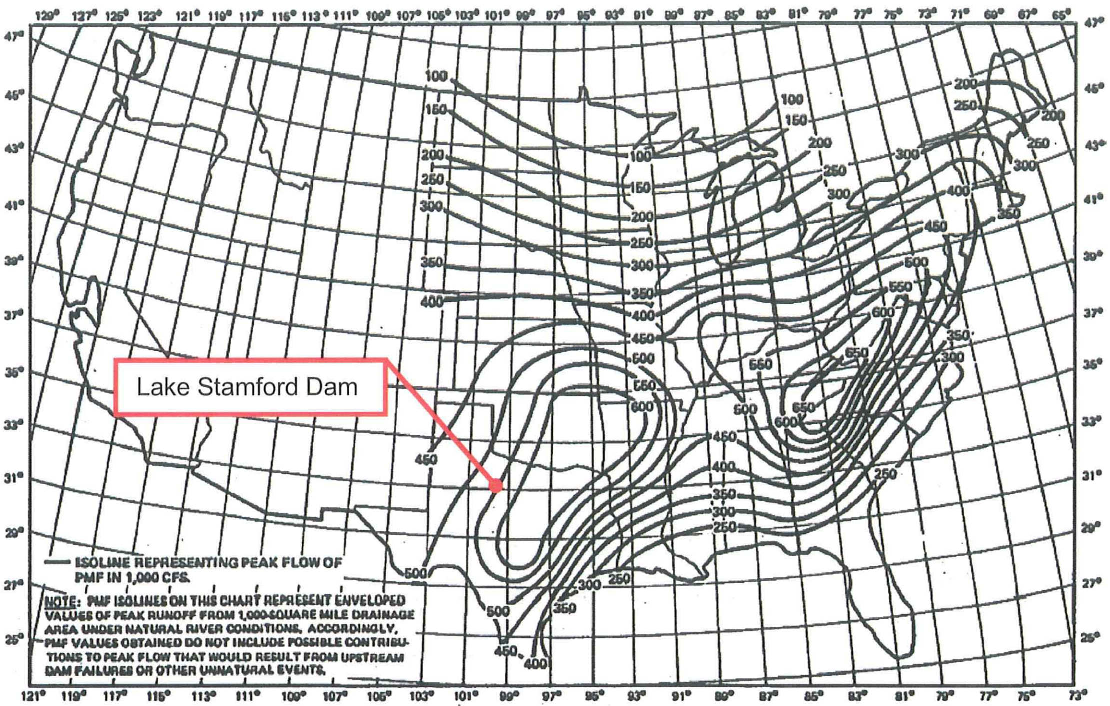

FIGURE B.4 PROBABLE MAXIMUM FLOOD (ENVELOPING PMF ISOLINES) FOR 1,000 SQUARE MILES

Figure 7-8. RG 1.59 1,000 sq. mi. Enveloping PMF Isolines Lake Stamford Dam

Table 7-4. RG 1.59 Appendix B PMF Results for Lake Stamford Dam

RG 1.59 chart	Drainage Area (sq. mi.)	PMF (cfs)
B2	100	180,000
B3	500	410,000
B4	1,000	550,000
B5	5,000	1,000,000
B6	10,000	1,290,000
B7	20,000	1,490,000

The results are plotted on a log-log scale and a smooth curve is fitted to the points, as shown in Figure 7-9. The drainage area for Lake Stamford Dam is then used to determine the PMF for Lake Stamford Dam.

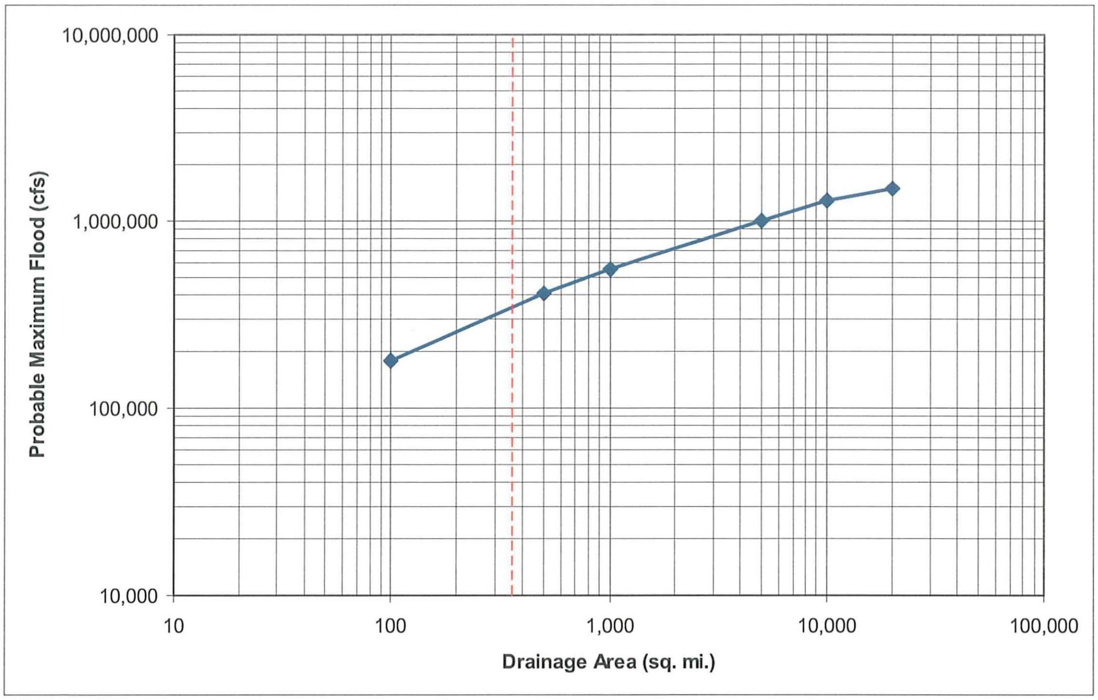

Figure 7-9. PMF Enveloping Isolines Based on Lake Stamford Dam Location

According to the Texas Water Development Board volumetric survey (Reference 20), the Lake Stamford Dam drainage area is 360 sq. mi. The vertical dashed line in Figure 7-9 is located at the corresponding drainage area value. The resulting PMF flow read from Figure 7-9 is 350,000 cfs.

The PMF is applied to Lake Stamford Dam to determine the water surface elevation for dam failure analysis. According to the Texas Water Development Board volumetric survey (Reference 20), Lake Stamford Dam is an earthfill embankment 3,600 ft. in length with a maximum height of 78 ft. or crest elevation 1,436.8 ft. The service spillway is an excavated channel at the left end of the dam with an uncontrolled spillway crest 100 ft. in length at elevation 1,416.8 ft. The normal pool elevation is 1,416.8 ft. The emergency spillway is a natural channel located at the right end of the embankment with a spillway crest elevation of 1,425.8 ft.

According to the USGS gauge 08084500 Water-Data Report 2009 (Reference 30), the maximum recorded elevation for the reservoir is 1,426.18 ft.

As the PMF is applied, it is assumed that Lake Stamford is full up to the crest elevation of 1,436.8 ft., which exceeds the maximum recorded reservoir elevation. It is also assumed that the service and emergency spillway capacities are unavailable to accommodate any portion of the PMF. Therefore, the water surface elevation is determined based on the PMF spilling over only the dam crest. Overtopping is modeled using the standard broad crested weir flow equation defined by the HEC-RAS reference manual (Reference 6, Equation 6-14).

$Q = C * L * H^{1.5}$

As previously discussed in Section 6.0, this calculation utilizes the HEC-RAS reference manual recommended weir flow coefficient of 2.6.

Tailwater is determined based on the PMF of 350,000 cfs. The 7.5 minute USGS quadrangle (Reference 32) for Lake Stamford East, TX was inserted into AutoCAD (Reference 2) to determine channel distances, slope, and cross section elevations. Figure 7-10 identifies the selected cross section in relationship to the dam and the channel distances used to determine the slope and elevations.

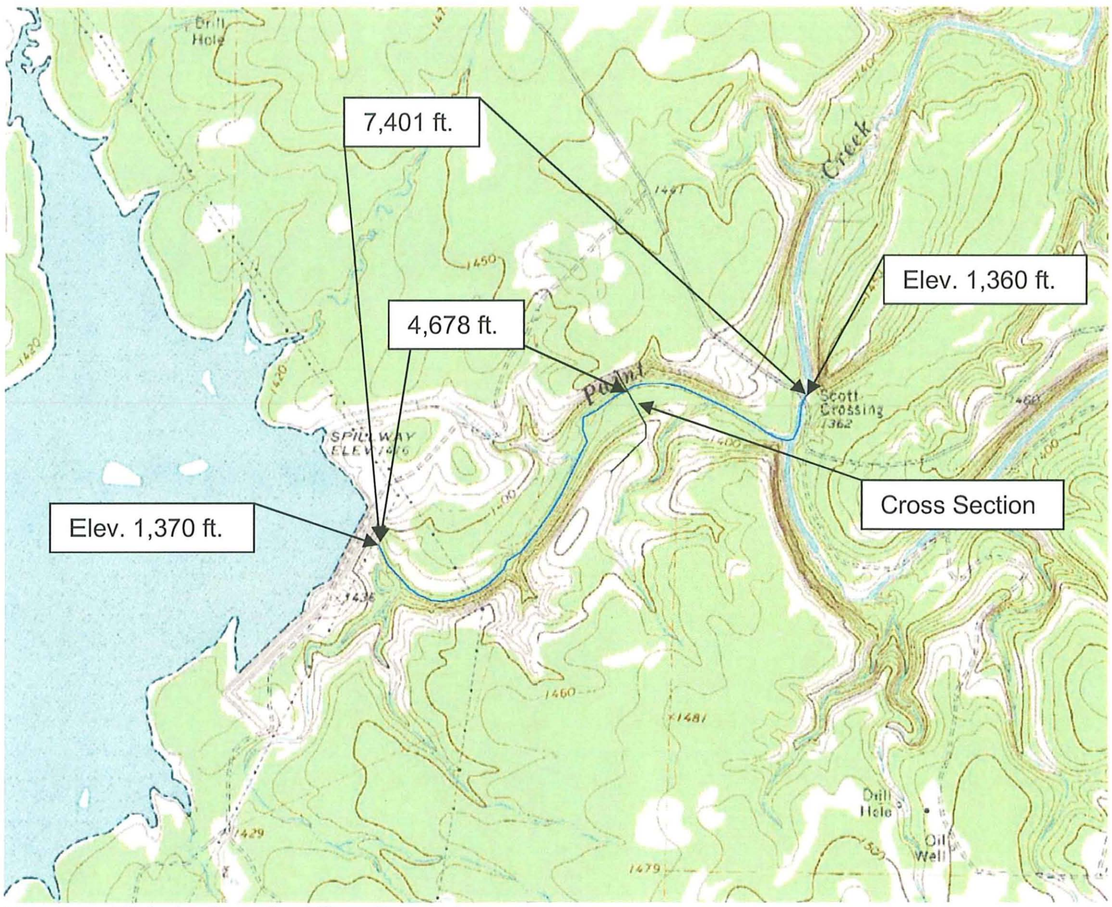

Figure 7-10. Lake Stamford Dam Downstream

As shown in Figure 7-10, the channel drops 10 ft. over a distance of 7,401 ft. Therefore, the channel slope is 10 ft. / 7,401 ft. = 0.0014 ft./ft. The cross section is 4,678 ft. downstream from elevation 1,370 ft. Therefore, the cross section bottom is 4,678 ft. / 7,401 ft. * 10 ft. = 6 ft. lower than elevation 1,370 ft. The cross section station and elevations are provided in Table 7-5.

Table 7-5. Lake Stamford Dam Tailwater Section Coordinates

Station (ft.)	Elevation (ft.)	Station (ft.)	Elevation (ft.)
-254	1,450	385	1,390
-134	1,400	448	1,400
-43	1,370	568	1,410
0	1,364	812	1,420
31	1,370	1,013	1,430
325	1,380	1,211	1,440

Stationing from left to right when looking downstream

Tailwater depth is determined using FlowMaster (Reference 3) and the Manning friction formula. From above the flow is 350,000 cfs and the slope is 0.0014 ft./ft. As previously discussed in Section 6.0, the Manning coefficient of 0.025 was applied to the channel and overbank areas.

The flow depth for the cross section is determined to be 45.1 ft. The FlowMaster results are provided in Appendix C. Therefore, the tailwater elevation at the downstream cross section is 1,364 ft. + 45.1 ft. = 1,409.1 ft. Level pool from the cross section upstream to the dam is assumed. This assumption neglects any increase to the tailwater elevation based on backwater effects. The tailwater elevation is well below the crest elevation of 1,436.8 ft. Therefore, the overtopping discharge is determined to be independent of tailwater. The cross section and tailwater elevation are shown on Figure 7-11.

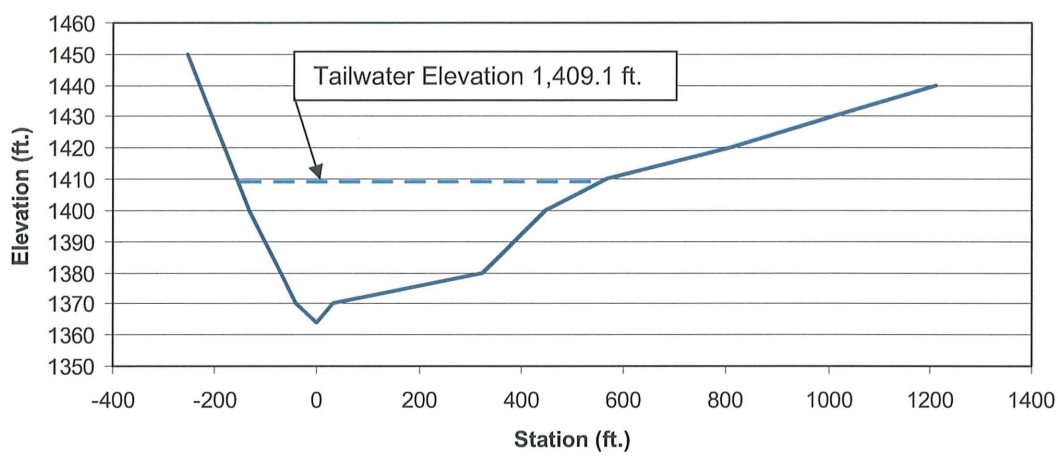

Figure 7-11. Lake Stamford Dam Tailwater

As previously discussed in Section 6.0, the overtopping elevation is determined using the broad crested weir flow equation with a 2.6 weir flow coefficient. Based on the 3,600 ft. dam crest at elevation 1,436.8 ft., the overtopping elevation is determined for the PMF of 350,000 cfs.

$Q = 2.6 * \text{crest length} * (\text{overtopping elevation} - \text{crest elevation})^{1.5}$

$350{,}000 \text{ cfs} = 2.6 * 3{,}600 \text{ ft.} * (Z - 1{,}436.8 \text{ ft.})^{1.5}$

Solving for overtopping elevation, Z = 1,447.98 ft. = 1,448.0 ft.

For the purpose of dam failure evaluation, it is conservative to round up because it results in a higher headwater elevation. A higher headwater elevation will maximize the water height component of the dam failure equation and the resulting dam failure flow.

According to USACE EM 1110-2-1420 (Reference 22) wind setup can be reasonably estimated for lakes and reservoirs using the following equation:

$S = U^2 * F / (1,400 * D)$

USACE EM 1110-2-1420 (Reference 22) indicates that the fetch distance is usually satisfactorily assumed to be two times the effective fetch distance. A straight line fetch is used to define the wind setup and is more conservative than an effective fetch.

As previously discussed, ANSI/ANS-2.8-1992 (Reference 1) is used to define the coincident wind speed. From Figure 7-5, the Annual Extreme-Mile, 30 ft. Above Ground, 2-yr. Mean Recurrence Interval is between 50 mph and 60 mph for the Brazos River watershed upstream from Whitney Lake. The more conservative wind speed of 60 mph is used to generate wind setup.

The overtopping elevation at Lake Stamford Dam is determined to be 1,448.0 ft. The fetch length is determined from the reservoir surface area at the overtopping elevation. The 7.5 minute USGS quadrangles (Reference 32) for Lake Stamford East, TX and Lake Stamford West, TX are inserted into AutoCAD (Reference 2) and because only contours with 10 ft. intervals are identified on the quadrangles, the 1,450 ft. elevation is used to determine the surface area. As shown on Figure 7-12, the longest straight line fetch distance is determined to be 56,087 ft. (rounded up to 10.7 mi.).

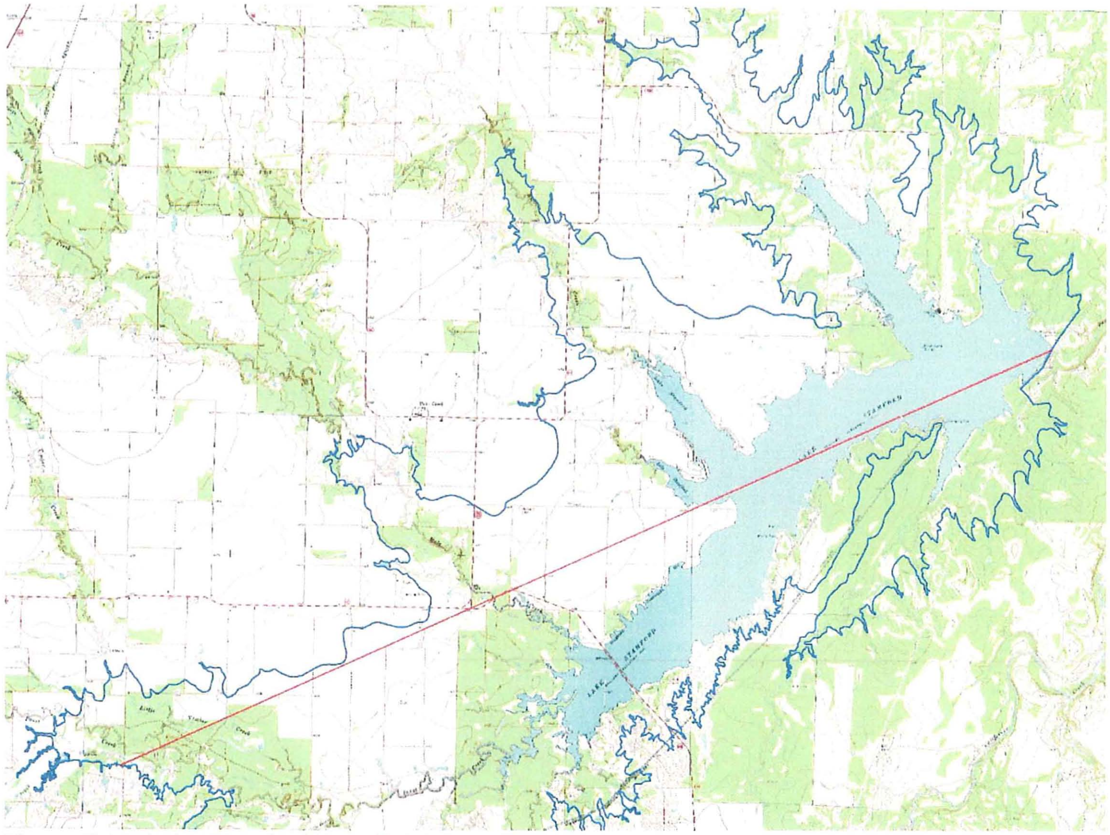

Figure 7-12. Lake Stamford Dam Fetch Length

A bottom surface profile along the fetch distance is created using the USGS quadrangles (Reference 32). For elevations below the water surface, bathymetry from the Texas Water Development Board volumetric survey (Reference 20) is inserted into AutoCAD (Reference 2), as shown on Figure 7-13. The bottom surface profile is shown on Figure 7-14.

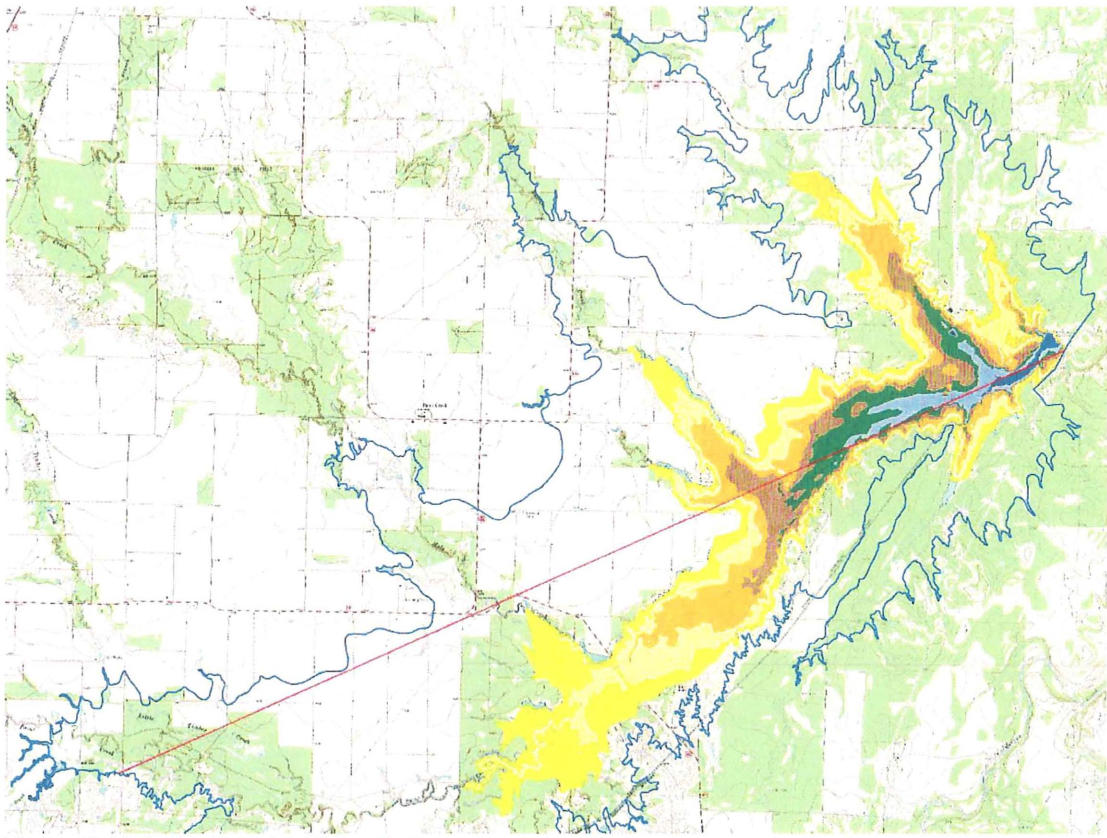

Figure 7-13. Lake Stamford Dam Bathymetry

The data for the distance and elevations are tabulated in Table 7-6. An average depth along the fetch distance is determined using the data in Table 7-6 and the following formula for hydraulic depth:

$$E = \frac{\left(\frac{Y_1 + Y_2}{2}\right) * (X_2 - X_1) + \ldots + \left(\frac{Y_{n-1} + Y_n}{2}\right) * (X_n - X_{n-1})}{X_n - X_1}$$

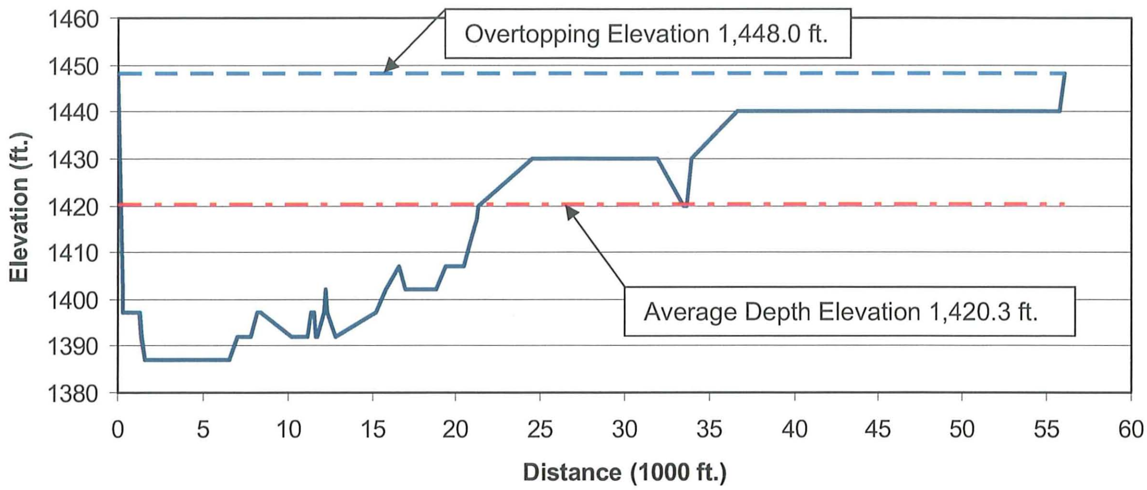

Figure 7-14. Lake Stamford Dam Bottom Surface Profile

Table 7-6. Lake Stamford Dam Bottom Surface Profile Section Coordinates

Distance (ft.)	Elevation (ft.)	Distance (ft.)	Elevation (ft.)	Distance (ft.)	Elevation (ft.)
0	1,448.0	11,004	1,392	19,418	1,407
244	1,402	11,231	1,392	20,410	1,407
317	1,397	11,381	1,397	20,849	1,412
641	1,397	11,555	1,397	21,188	1,417
1,229	1,397	11,658	1,392	21,354	1,420
1,379	1,392	11,737	1,392	24,534	1,430
1,513	1,387	12,126	1,397	31,883	1,430
2,484	1,387	12,242	1,402	33,515	1,420
4,121	1,387	12,357	1,397	33,637	1,420
6,593	1,387	12,797	1,392	33,970	1,430
7,080	1,392	15,282	1,397	36,642	1,440
7,781	1,392	15,845	1,402	55,768	1,440
8,202	1,397	16,630	1,407	56,087	1,448.0
8,419	1,397	16,957	1,402		
10,202	1,392	18,793	1,402		

Note: Distance 0 ft. is at the dam.

The average depth bottom surface elevation is calculated to be 1,420.3 ft. The overtopping water surface elevation is 1,448.0 ft. Therefore, the average depth along the fetch distance is calculated to be 1,448.0 ft. – 1,420.3 ft. = 27.7 ft. From above, the wind speed is 60 mph and the fetch distance is 10.7 mi. Wind setup is calculated as follows:

$S = (60 \text{ mph})^2 * (10.7 \text{ mi.}) / (1{,}400 * 27.7 \text{ ft.}) = 0.99 \text{ ft.}$

Setup is conservatively rounded up to 1.0 ft. For the purpose of dam failure evaluation, it is conservative to round up because it results in a higher headwater elevation. A higher headwater elevation will maximize the water height component of the dam failure equation and the resulting dam failure flow. The PMF headwater elevation at Lake Stamford Dam including wind setup is 1,448.0 ft. + 1.0 ft. = 1,449.0 ft. and is based on a starting elevation exceeding the maximum historical elevation.

Dam Failure Lake Stamford Dam

As previously discussed, dam failure is evaluated based on two methods. As identified in ANSI/ANS-2.8-1992 (Reference 1, Section 5.1.3.2) the breach wave height is computed as h = 4 * (headwater – tailwater) / 9 and transposed downstream without attenuation. Alternatively, dam failure flow is calculated using a USACE dam breach equation (Reference 24) and USACE RD-13 breach parameters (Reference 23).

As identified above, the dam is 78 ft. tall with a crest elevation at 1,436.8 ft. The headwater is determined to be 1,449.0 ft. and the tailwater is determined to be 1,409.1 ft. The breach wave height for the main dam is calculated as follows:

h = 4 * (1,449.0 ft. – 1,409.1 ft.) / 9 = 17.73 ft., rounded up to 17.8 ft.

Breach parameters for an earth fill dam are determined using USACE RD-13 (Reference 23, Table 1, Page 17). The greatest breach width and side slopes maximize the resulting breach flow. The breach width, W_b, is three times the dam height of 78 ft., and the side slopes of the breach are 1:1 (horizontal:vertical).

Therefore, W_b = 3 * height of dam = 3 * 78 ft. = 234 ft.

HEC-HMS version 2.2.0 release notes (Reference 24, Page 8) are used for the dam break equation including side slopes.

$$Q = 1.7 * W_b * h^{1.5} + 1.35 * S * h^{2.5}$$

The water height is equal to the smaller of the head difference between headwater and tailwater or the head difference between headwater and the breach bottom invert elevation. The difference between the headwater and tailwater is 1,449.0 ft. – 1,409.1 ft. = 39.9 ft. The difference between the headwater and breach bottom is greater than the full height of the dam, 78 ft. Therefore, the breach flow is calculated using the difference between the headwater and tailwater as follows:

$Q = 1.7 * 234\text{ ft.} * (39.9\text{ ft.})^{1.5} + 1.35 * 1 * (39.9\text{ ft.})^{2.5}$ = 113,835 cfs, rounded up to 120,000 cfs.

The total flow is assumed to be the sum of the overtopping and spillway flow previously determined added to the breach flow. There is no reduction in overtopping flow to account for the breached section of the dam. Therefore, the total flow from the Lake Stamford Dam failure is: Q = 350,000 cfs + 120,000 cfs = 470,000 cfs.

PMF Fort Phantom Hill Dam Watershed

Based on the NID database (Reference 25), Fort Phantom Hill Dam is located at the coordinates, latitude 32.6167° and longitude -99.6683°.

The NRC RG 1.59 Appendix B (Reference 34) method is used to determine the PMF for Fort Phantom Hill Dam. This method is based on the location of the site and utilizes charts of enveloping

PMF isolines for various watershed drainage areas. Figure 7-15 is a typical chart showing the location of Fort Phantom Hill Dam. Table 7-7 presents the results for each chart corresponding to the drainage area provided. Straight line interpolation is used between isolines.

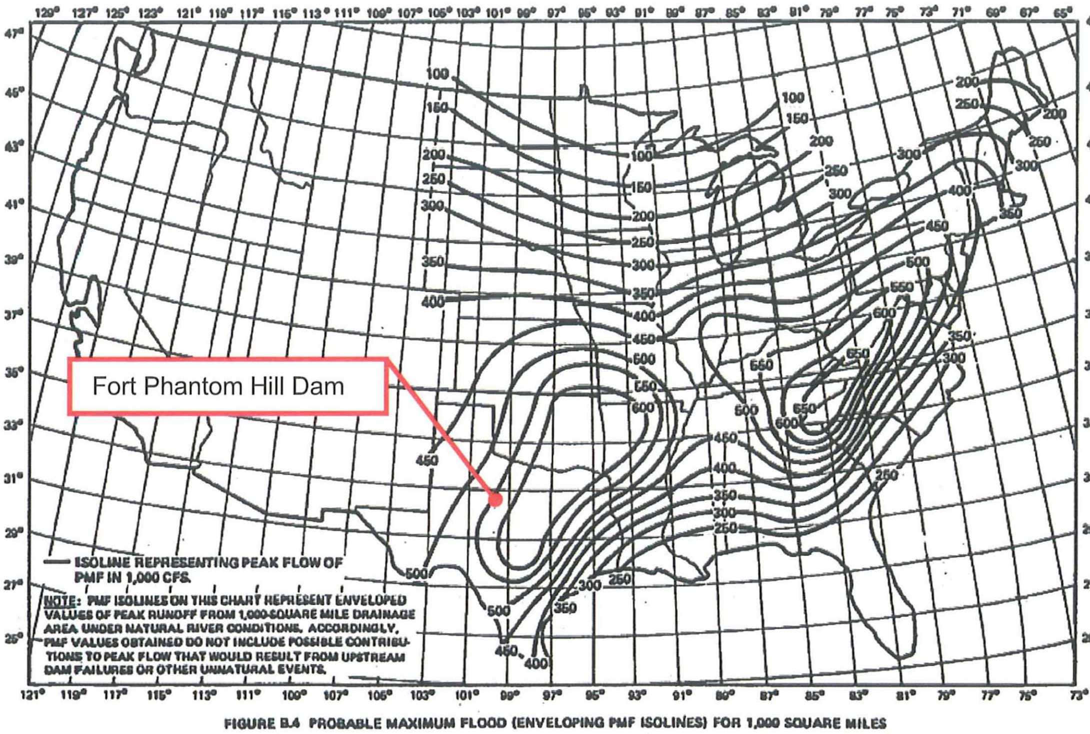

Figure 7-15. RG 1.59 1,000 sq. mi. Enveloping PMF Isolines Fort Phantom Hill Dam

Table 7-7. RG 1.59 Appendix B PMF Results for Fort Phantom Hill Dam

RG 1.59 chart	Drainage Area (sq. mi.)	PMF (cfs)
B2	100	180,000
B3	500	420,000
B4	1,000	550,000
B5	5,000	1,000,000
B6	10,000	1,300,000
B7	20,000	1,500,000

The results are plotted on a log-log scale and a smooth curve is fitted to the points, as shown on Figure 7-16. The drainage area for Fort Phantom Hill Dam is then used to determine the PMF for Fort Phantom Hill Dam.

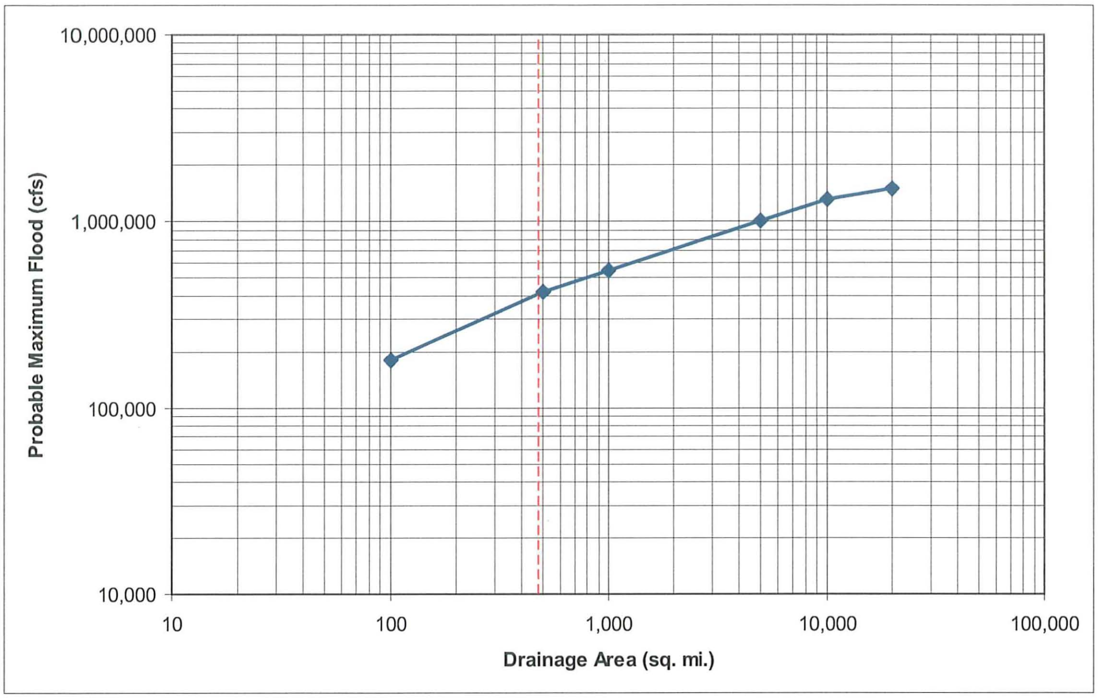

Figure 7-16. PMF Enveloping Isolines Based on Fort Phantom Hill Dam Location

According to the Texas Water Development Board volumetric survey (Reference 16), the Fort Phantom Hill Dam drainage area is 478 sq. mi. The vertical dashed line in Figure 7-16 is located at the corresponding drainage area value. The resulting PMF flow read from Figure 7-16 is 410,000 cfs.

The PMF is applied to Fort Phantom Hill Dam to determine the water surface elevation for dam failure analysis. According to the Texas Water Development Board volumetric survey (Reference 16), Fort Phantom Hill Dam is an earthfill embankment 3,740 ft. in length with a maximum height of 84 ft. The spillway is a natural ground channel with an uncontrolled ogee crest 800 ft. in length at elevation 1,635.9 ft. The normal pool elevation is 1,635.9 ft.

According to the USGS gauge 08083500 Water-Data Report 2009 (Reference 29), the crest of the dam is 1,650.0 ft. and the maximum recorded elevation for the reservoir is 1,639.50 ft.

Based on the USGS quadrangles (Reference 32) for Lake Fort Phantom Hill, there is a levee along the west side of the lake at elevation 1,643 ft. The 7.5 minute USGS quadrangle (Reference 32) for Hamby, TX is inserted into AutoCAD (Reference 2) to determine the levee distance. Figure 7-17 identifies the 6,765 ft. long levee in relationship to the dam.

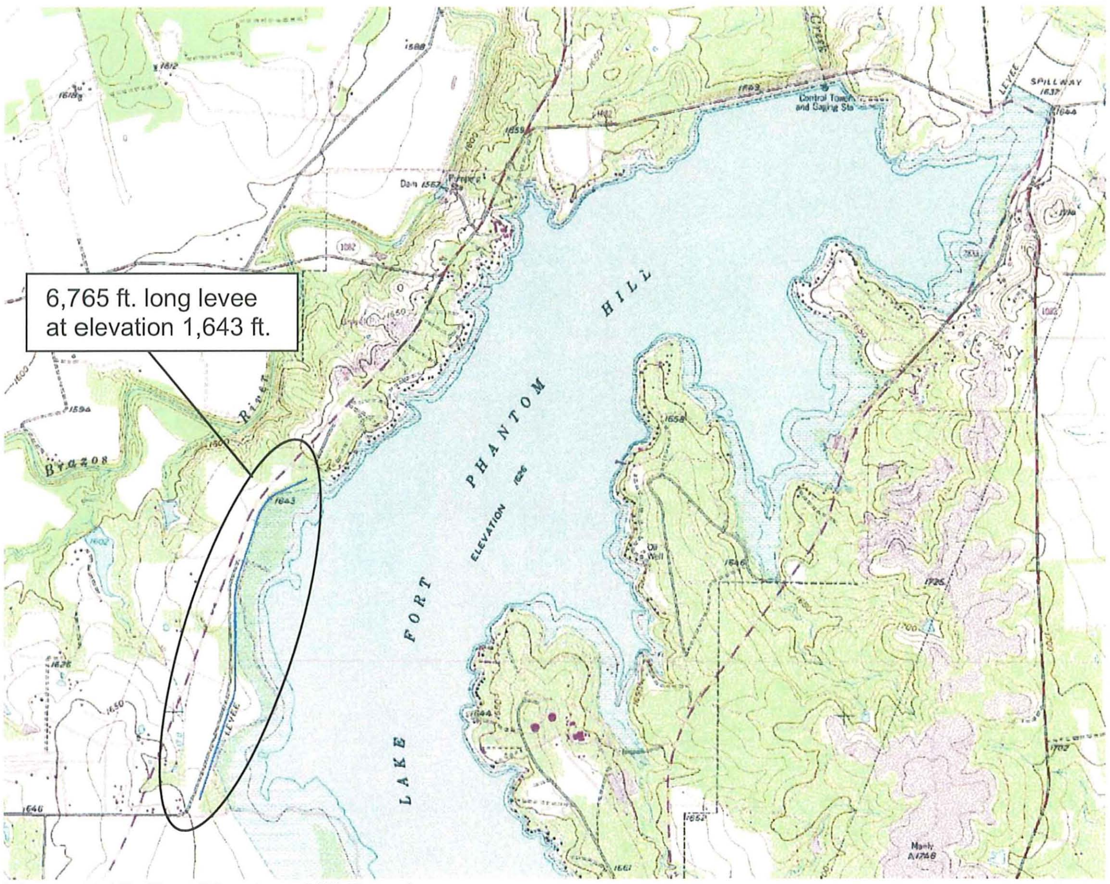

Figure 7-17. Fort Phantom Hill Dam Levee

As the PMF is applied, it is assumed that Lake Fort Phantom Hill is full up to the levee elevation of 1,643.0 ft., which exceeds the maximum recorded reservoir elevation. It is also assumed that the outlet works and spillway capacities are unavailable to accommodate any portion of the PMF. Therefore, the water surface elevation is determined based on the PMF spilling over the levee and the dam crest. Overtopping is modeled using the standard broad crested weir flow equation defined by the HEC-RAS reference manual (Reference 6, Equation 6-14).

$$Q = C * L * H^{1.5}$$

As previously discussed in Section 6.0, this calculation utilizes the HEC-RAS reference manual recommended weir flow coefficient of 2.6.

Tailwater is determined based on the PMF of 410,000 cfs. Flow overtopping the levee is directed into the Clear Fork of the Brazos and joins Elm Creek just below the dam. Elm Creek is the tributary on which the dam is located. The 7.5 minute USGS quadrangles (Reference 32) for Nugent, TX and Hamby, TX are inserted into AutoCAD to determine channel distances, slope, and cross section elevations. Figure 7-18 identifies the selected cross section in relationship to the dam and the channel distances used to determine the slope and elevations.

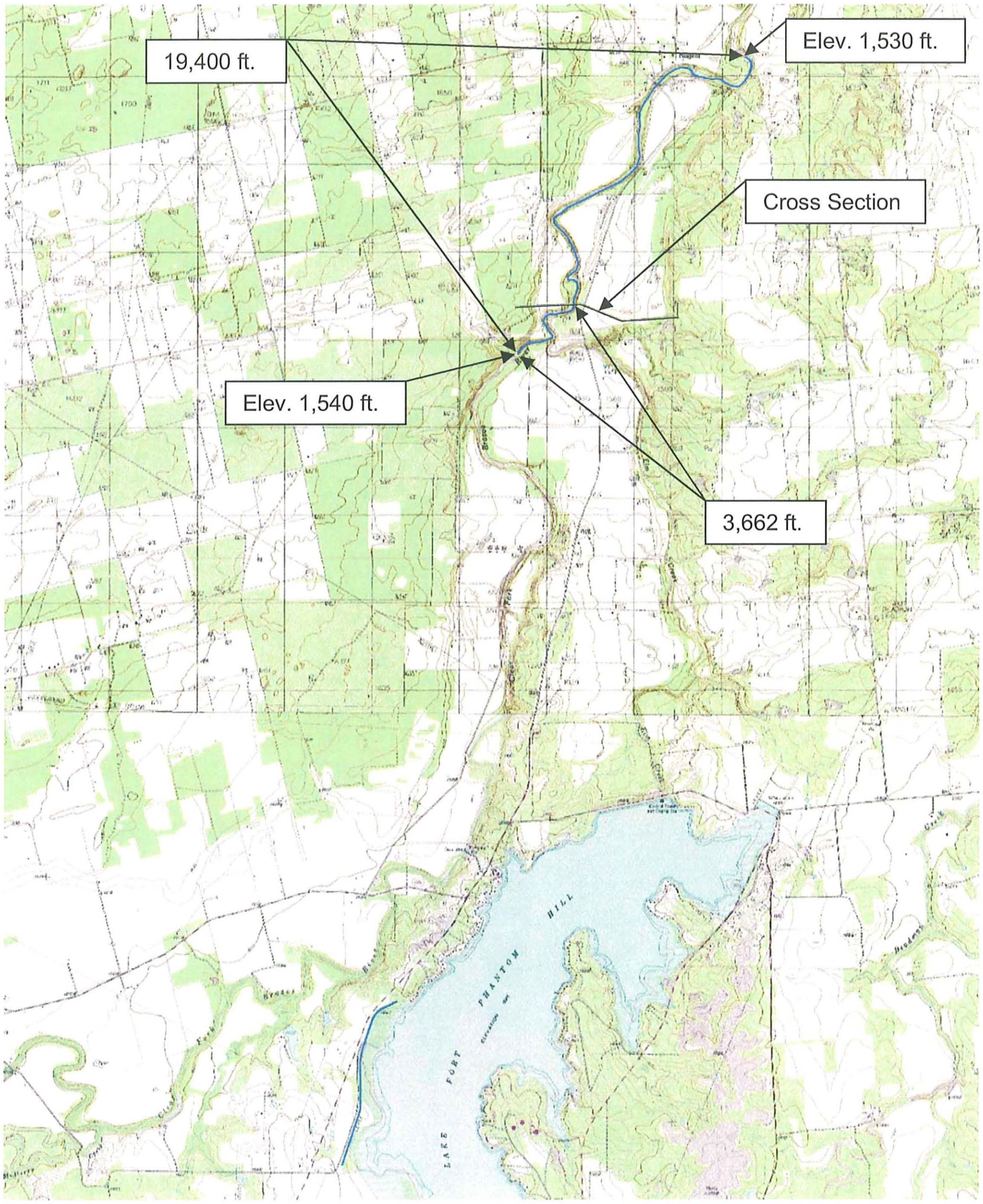

Figure 7-18. Fort Phantom Hill Dam Downstream

As shown in Figure 7-18, the channel drops 10 ft. over a distance of 19,400 ft. Therefore, the channel slope is 10 ft. / 19,400 ft. = 0.00052 ft./ft. The cross section is 3,662 ft. downstream from elevation 1,540 ft. Therefore, the cross section bottom is 3,662 ft. / 19,400 ft. * 10 ft. = 2 ft. lower than elevation 1,540 ft. The cross section station and elevations are provided in Table 7-8.

Table 7-8. Fort Phantom Hill Dam Tailwater Section Coordinates

Station (ft.)	Elevation (ft.)	Station (ft.)	Elevation (ft.)
-2,173	1,620	0	1,538
-1,908	1,610	71	1,540
-1,743	1,600	112	1,560
-1,600	1,590	968	1,565
-1,487	1,580	2,904	1,570
-1,361	1,570	3,594	1,580
-272	1,560	3,659	1,590
-210	1,550	4,008	1,600
-53	1,540		

Stationing from left to right when looking downstream

Tailwater depth is determined using FlowMaster (Reference 3) and the Manning friction formula. From above the flow is 410,000 cfs and the slope is 0.00052 ft./ft. As previously discussed in Section 6.0, the Manning coefficient of 0.025 is applied to the channel and overbank areas.

The flow depth for the cross section is determined to be 38.95 ft. and is rounded down to 38.9 ft. This is conservative because it results in a lower tailwater elevation. The FlowMaster results are provided in Appendix D. Therefore, the tailwater elevation at the downstream cross section is 1,538 ft. + 38.9 ft. = 1,576.9 ft. Level pool from the cross section upstream to the dam is assumed. This assumption neglects any increase to the tailwater elevation based on backwater effects. The tailwater elevation is well below the levee crest elevation of 1,643.0 ft. and the dam crest elevation of 1,650.0 ft. Therefore, the overtopping discharge is determined to be independent of tailwater. The cross section and tailwater elevation are shown on Figure 7-19.

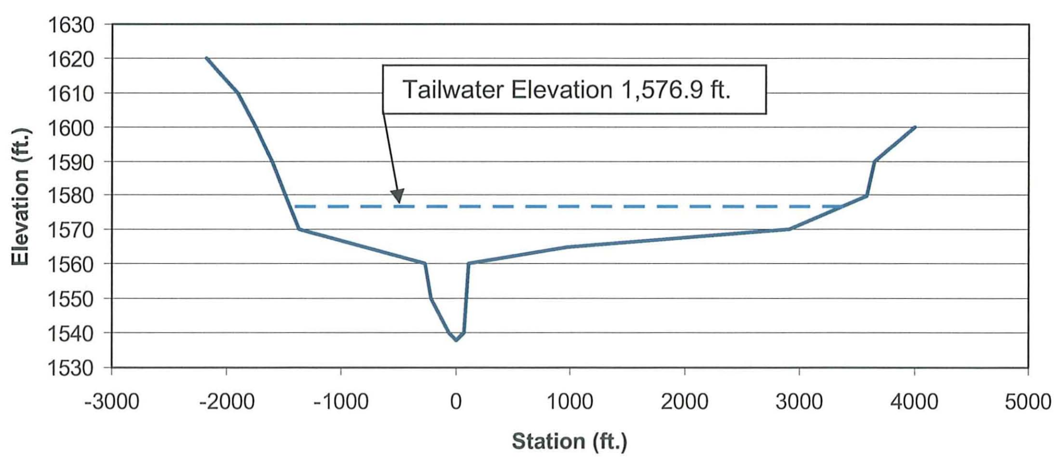

Figure 7-19. Fort Phantom Hill Dam Tailwater

As previously discussed in Section 6.0, the overtopping elevation is determined using the broad crested weir flow equation with a 2.6 weir flow coefficient. Based on the 6,765 ft. levee crest at elevation 1,643.0 ft. and the 3,740 ft. dam crest at elevation 1,650.0 ft., the overtopping elevation is determined for the PMF of 410,000 cfs.

$Q = 2.6 * \text{levee length} * (\text{overtopping elevation} - \text{levee elevation})^{1.5} + 2.6 * \text{dam length} * (\text{overtopping elevation} - \text{dam elevation})^{1.5}$

$410{,}000 \text{ cfs} = 2.6 * 6{,}765 \text{ ft.} * (Z - 1{,}643.0 \text{ ft.})^{1.5} + 2.6 * 3{,}740 \text{ ft.} * (Z - 1{,}650.0 \text{ ft.})^{1.5}$

Solving for overtopping elevation, Z = 1,651.03 ft. = 1,651.1 ft.

For the purpose of dam failure evaluation, it is conservative to round up because it results in a higher headwater elevation. A higher headwater elevation will maximize the water height component of the dam failure equation and the resulting dam failure flow.

According to USACE EM 1110-2-1420 (Reference 22) wind setup can be reasonably estimated for lakes and reservoirs using the following equation:

$S = U^2 * F / (1{,}400 * D)$

USACE EM 1110-2-1420 (Reference 22) indicates that the fetch distance is usually satisfactorily assumed to be two times the effective fetch distance. A straight line fetch is used to define the wind setup and is more conservative than an effective fetch.

As previously discussed, ANSI/ANS-2.8-1992 (Reference 1) is used to define the coincident wind speed. From Figure 7-5, the Annual Extreme-Mile, 30 ft. Above Ground, 2-yr. Mean Recurrence Interval is between 50 mph and 60 mph for the Brazos River watershed upstream from Whitney Lake. The more conservative wind speed of 60 mph is used to generate wind setup.

The overtopping elevation at Fort Phantom Dam is determined to be 1,651.1 ft. The fetch length is determined from the reservoir surface area at the overtopping elevation. The 7.5 minute USGS quadrangle (Reference 32) for Hamby, TX was inserted into AutoCAD (Reference 2) and because only contours with 10 ft. intervals are identified on the quadrangles, midway between the 1,650 ft. and 1,660 ft. contours is used to determine the surface area at the overtopping elevation. As shown on Figure 7-20, the longest straight line fetch distance is determined to be 41,522 ft. (rounded up to 7.9 mi.).

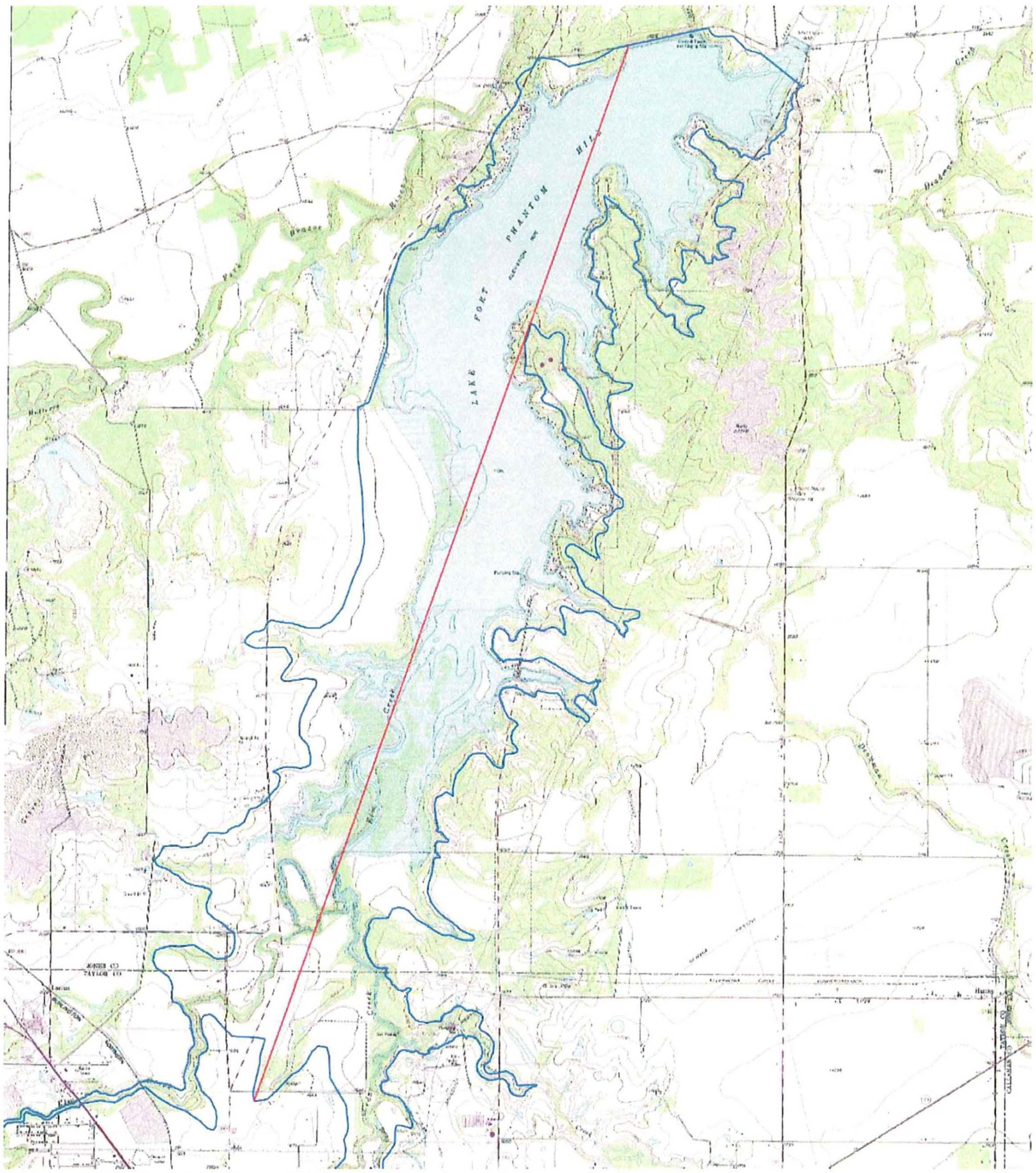

Figure 7-20. Fort Phantom Hill Dam Fetch Length

A bottom surface profile along the fetch distance is created using the USGS quadrangles (Reference 32). For elevations below the water surface, bathymetry from the Texas Water Development Board volumetric survey (Reference 16) is inserted into AutoCAD (Reference 2), as shown on Figure 7-21. The bottom surface profile is shown on Figure 7-22.

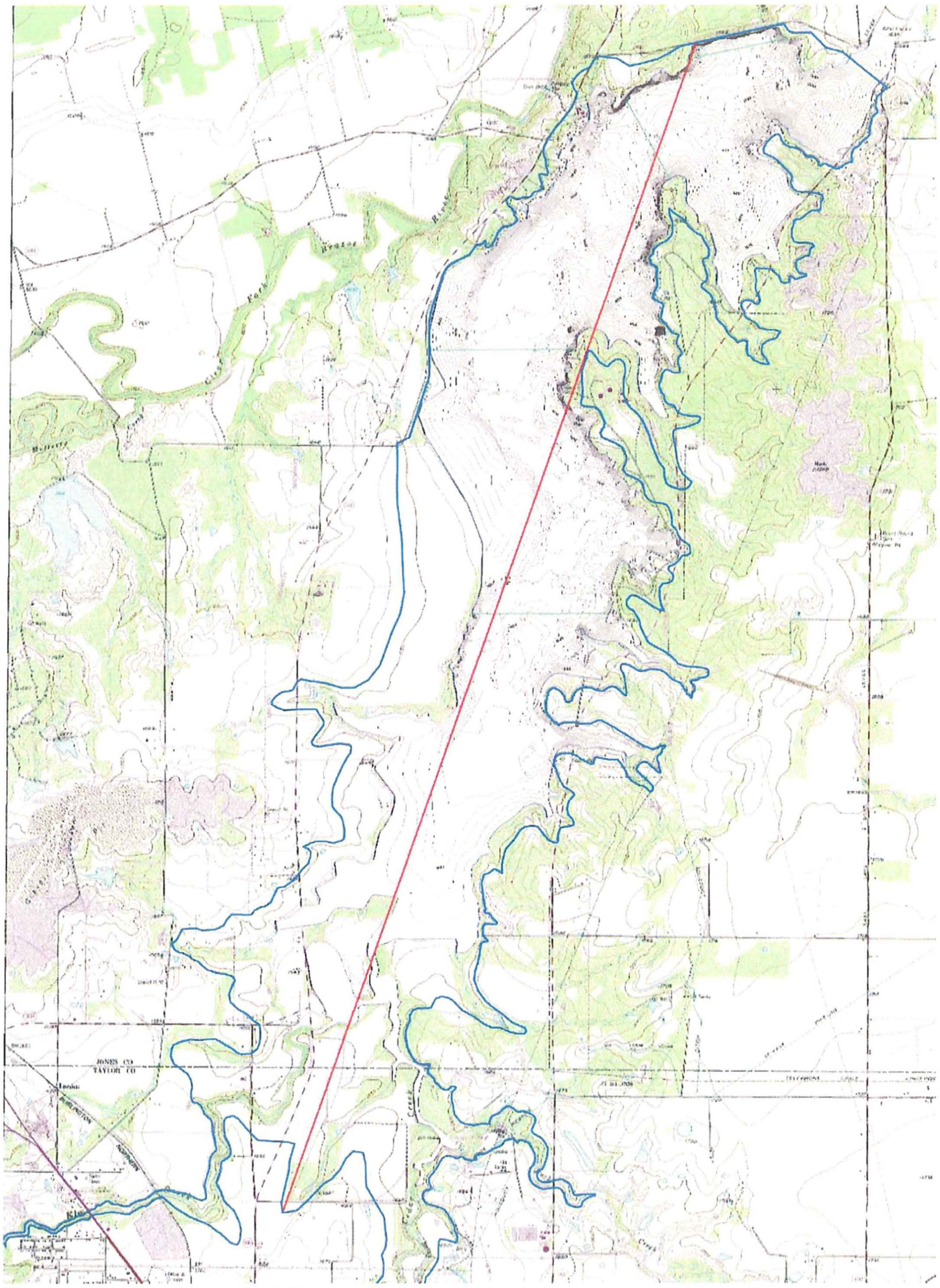

Figure 7-21. Fort Phantom Hill Dam Bathymetry

The data for the distance and elevations are tabulated in Table 7-9. An average depth along the fetch distance is determined using the data in Table 7-9 and the following formula for hydraulic depth:

$$E = \frac{\left(\frac{Y_1 + Y_2}{2}\right) * (X_2 - X_1) + \ldots + \left(\frac{Y_{n-1} + Y_n}{2}\right) * (X_n - X_{n-1})}{X_n - X_1}$$

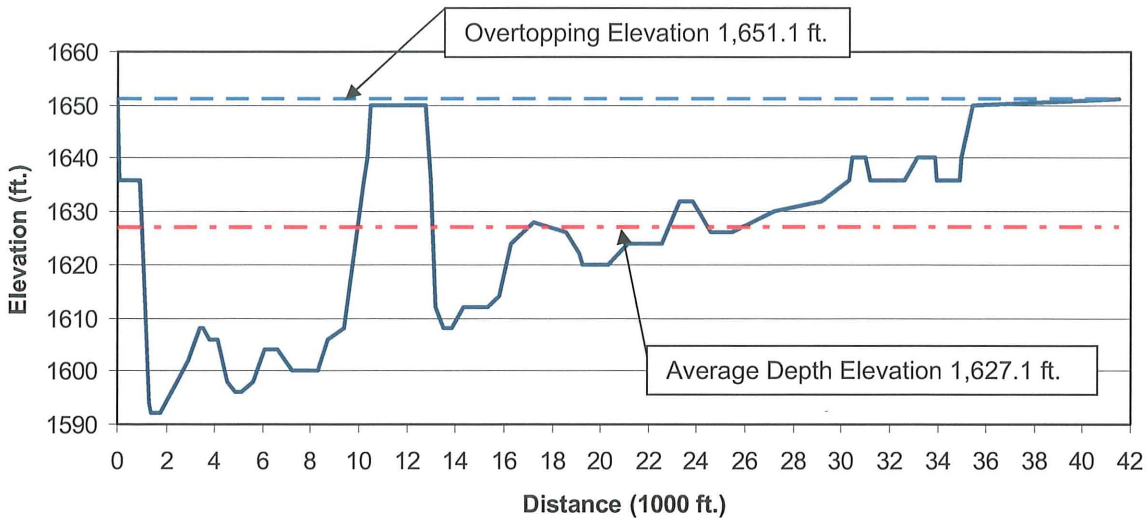

Figure 7-22. Fort Phantom Hill Dam Bottom Surface Profile

Table 7-9. Fort Phantom Hill Dam Bottom Surface Profile Section Coordinates

Distance (ft.)	Elevation (ft.)	Distance (ft.)	Elevation (ft.)	Distance (ft.)	Elevation (ft.)
0	1,651.1	9,364	1,608	23,291	1,632
100	1,636	10,190	1,636	23,856	1,632
863	1,636	10,302	1,640	24,581	1,626
1,303	1,594	10,464	1,650	25,440	1,626
1,377	1,592	12,783	1,650	27,199	1,630
1,752	1,592	12,964	1,636	29,168	1,632
2,924	1,602	13,166	1,612	30,345	1,636
3,354	1,608	13,493	1,608	30,451	1,640
3,536	1,608	13,824	1,608	30,972	1,640
3,806	1,606	14,348	1,612	31,180	1,636
4,119	1,606	15,336	1,612	32,636	1,636
4,502	1,598	15,824	1,614	33,159	1,640
4,861	1,596	16,268	1,624	33,922	1,640
5,141	1,596	17,227	1,628	33,942	1,636
5,636	1,598	18,566	1,626	34,888	1,636
6,089	1,604	19,129	1,622	34,956	1,640
6,639	1,604	19,248	1,620	35,417	1,650
7,220	1,600	20,298	1,620	41,522	1,651.1
8,286	1,600	21,110	1,624		
8,743	1,606	22,579	1,624		

Note: Distance 0 ft. is at the dam.

The average depth bottom surface elevation is calculated to be 1,627.1 ft. The overtopping water surface elevation is 1,651.1 ft. Therefore, the average depth along the fetch distance is calculated to be 1,651.1 ft. – 1,627.1 ft. = 24.0 ft. From above, the wind speed is 60 mph and the fetch distance is 7.9 mi. Wind setup is calculated as follows:

$S = (60\text{ mph})^2 * (7.9\text{ mi.}) / (1{,}400 * 24.0\text{ ft.}) = 0.85\text{ ft.}$

Setup is conservatively rounded up to 0.9 ft. For the purpose of dam failure evaluation, it is conservative to round up because it results in a higher headwater elevation. A higher headwater elevation will maximize the water height component of the dam failure equation and the resulting dam failure flow. The PMF headwater elevation at Fort Phantom Hill Dam including wind setup is 1,651.1 ft. + 0.9 ft. = 1,652.0 ft. and is based on a starting elevation exceeding the maximum historical elevation.

Dam Failure Fort Phantom Hill Dam

As previously discussed, dam failure is evaluated based on two methods. As identified in ANSI/ANS-2.8-1992 (Reference 1, Section 5.1.3.2) the breach wave height is computed as h = 4 * (headwater – tailwater) / 9 and transposed downstream without attenuation. Alternatively, dam failure flow is calculated using a USACE dam breach equation (Reference 24) and USACE RD-13 breach parameters (Reference 23).

As identified above, the dam is 84 ft. tall with a crest elevation at 1,650.0 ft. The headwater is determined to be 1,652.0 ft. and the tailwater is determined to be 1,576.9 ft. The breach wave height for the main dam is calculated as follows:

h = 4 * (1,652.0 ft. – 1,576.9 ft.) / 9 = 33.38 ft., rounded to 33.4 ft.

Breach parameters for an earth fill dam are determined using USACE RD-13 (Reference 23, Table 1, Page 17). The greatest breach width and side slopes maximize the resulting breach flow. The breach width, W_b, is three times the dam height of 84 ft., and the side slopes of the breach are 1:1 (horizontal:vertical).

Therefore, W_b = 3 * height of dam = 3 * 84 ft. = 252 ft.

HEC-HMS version 2.2.0 release notes (Reference 24, Page 8) are used for the dam break equation including side slopes.

$$Q = 1.7 * W_b * h^{1.5} + 1.35 * S * h^{2.5}$$

The water height is equal to the smaller of the head difference between headwater and tailwater or the head difference between headwater and the breach bottom invert elevation. The difference between the headwater and tailwater is 1,652.0 ft. – 1,576.9 ft. = 75.1 ft. The difference between the headwater and breach bottom is greater than the full height of the dam, 84 ft. Therefore, the breach flow is calculated using the difference between the headwater and tailwater as follows:

$Q = 1.7 * 252\ ft. * (75.1\ ft.)^{1.5} + 1.35 * 1 * (75.1\ ft.)^{2.5}$ = 344,794 cfs, rounded up to 350,000 cfs.

The total flow is assumed to be the sum of the overtopping and spillway flow previously determined added to the breach flow. There is no reduction in overtopping flow to account for the breached section of the dam. Therefore, the total flow from the Fort Phantom Hill Dam failure is: Q = 410,000 cfs + 350,000 cfs = 760,000 cfs.

<u>PMF Cedar Ridge Reservoir Dam Watershed</u>

According to the Brazos G Regional Water Plan (Reference 4), Cedar Ridge Reservoir Dam would be constructed on the Clear Fork of the Brazos River along the Shackelford county line near the intersection with the Haskell and Throckmorton county line as shown on Figure 7-23. Based on USGS 7.5 minute quadrangle (Reference 32) Antelope Hills, TX, as shown on Figure 7-24, Cedar Ridge Reservoir Dam would be constructed at the coordinates, latitude 32°57'30" and longitude -99°27'30".

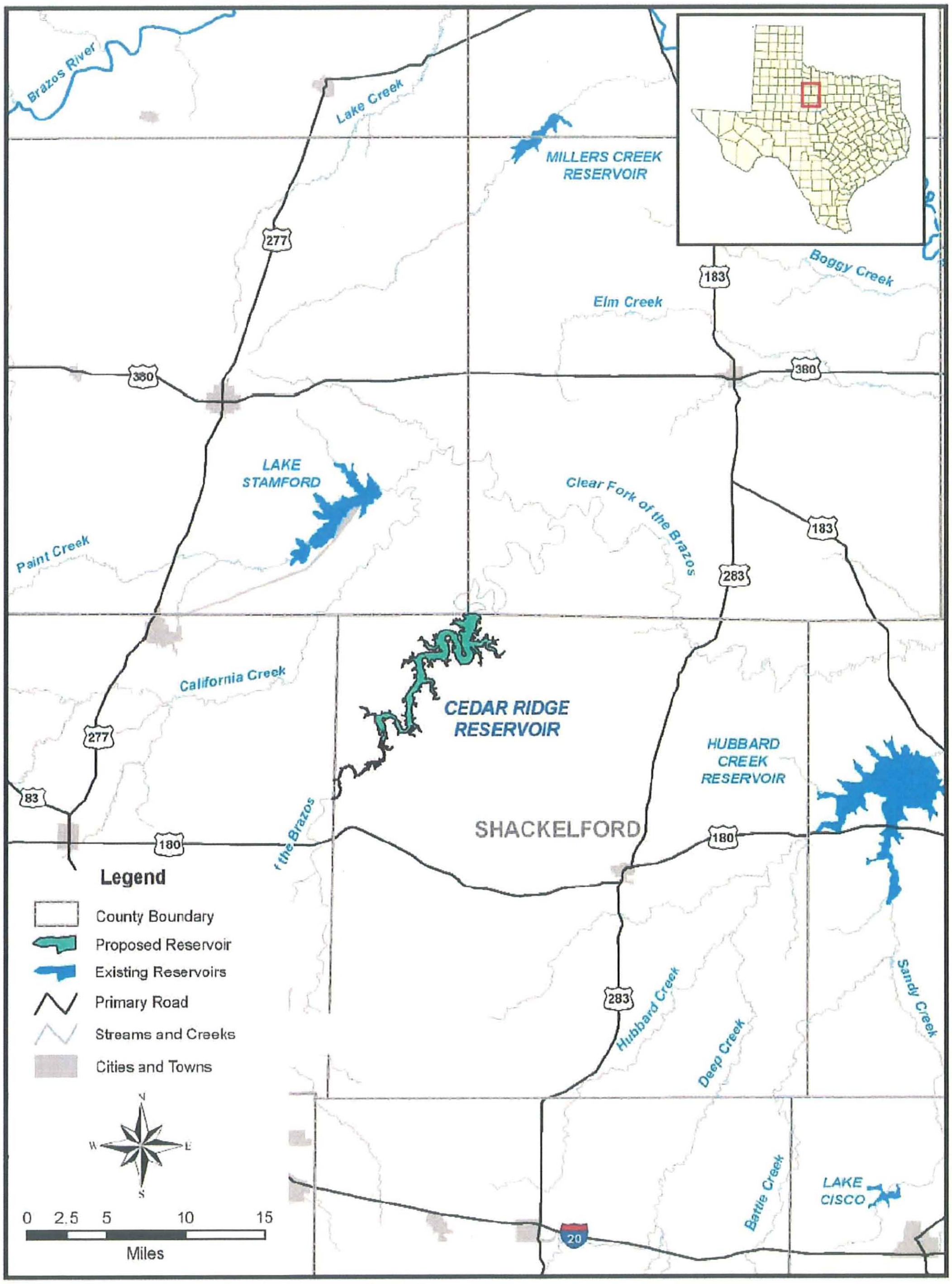

Figure 7-23. Cedar Ridge Reservoir Dam Location

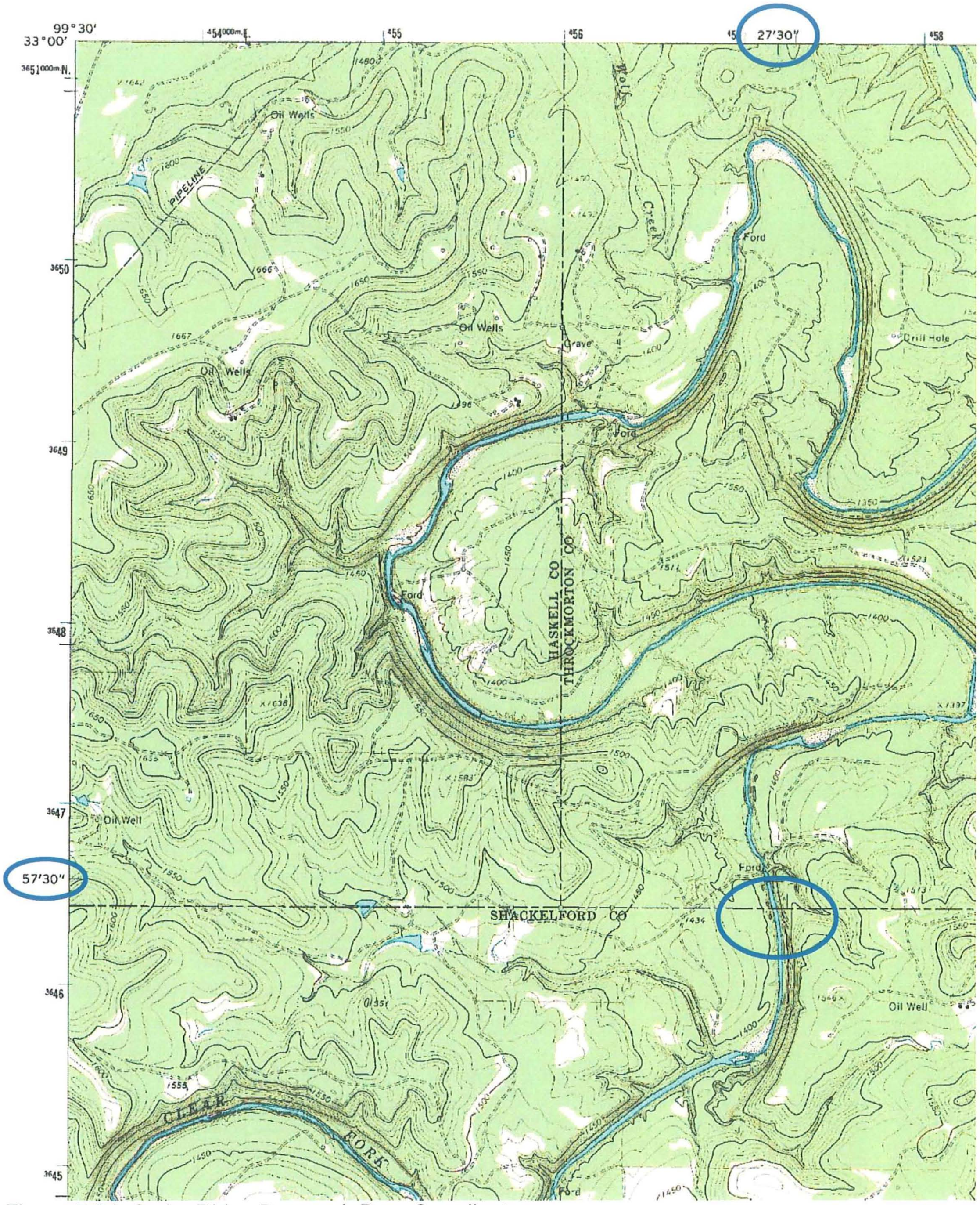

Figure 7-24. Cedar Ridge Reservoir Dam Coordinates

The NRC RG 1.59 Appendix B (Reference 34) method is used to determine the PMF for Cedar Ridge Reservoir Dam. This method is based on the location of the site and utilizes charts of enveloping PMF isolines for various watershed drainage areas. Figure 7-25 is a typical chart showing the location of Cedar Ridge Reservoir Dam. Table 7-10 presents the results for each chart corresponding to the drainage area provided. Straight line interpolation is used between isolines.

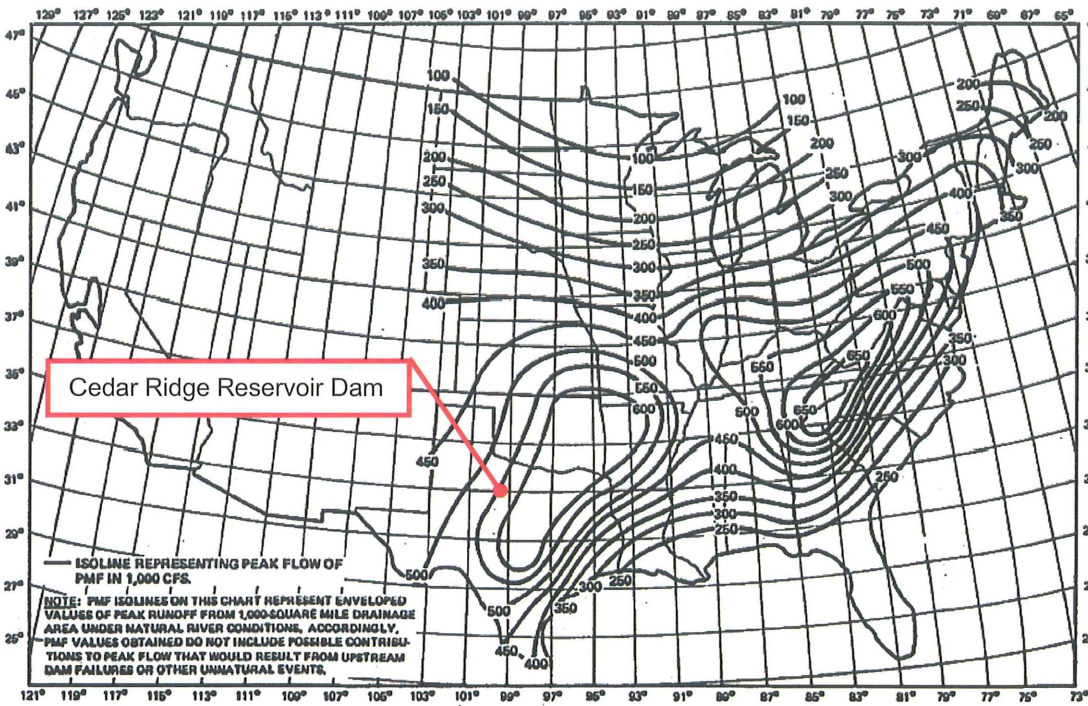

FIGURE B.4 PROBABLE MAXIMUM FLOOD (ENVELOPING PMF ISOLINES) FOR 1,000 SQUARE MILES

Figure 7-25. RG 1.59 1,000 sq. mi. Enveloping PMF Isolines Cedar Ridge Reservoir Dam

Table 7-10. RG 1.59 Appendix B PMF Results for Cedar Ridge Reservoir Dam

RG 1.59 chart	Drainage Area (sq. mi.)	PMF (cfs)
B2	100	180,000
B3	500	415,000
B4	1,000	560,000
B5	5,000	1,015,000
B6	10,000	1,300,000
B7	20,000	1,500,000

The results are plotted on a log-log scale and a smooth curve is fitted to the points, as shown in Figure 7-26. The drainage area for Cedar Ridge Reservoir Dam is then used to determine the PMF for Cedar Ridge Reservoir Dam.

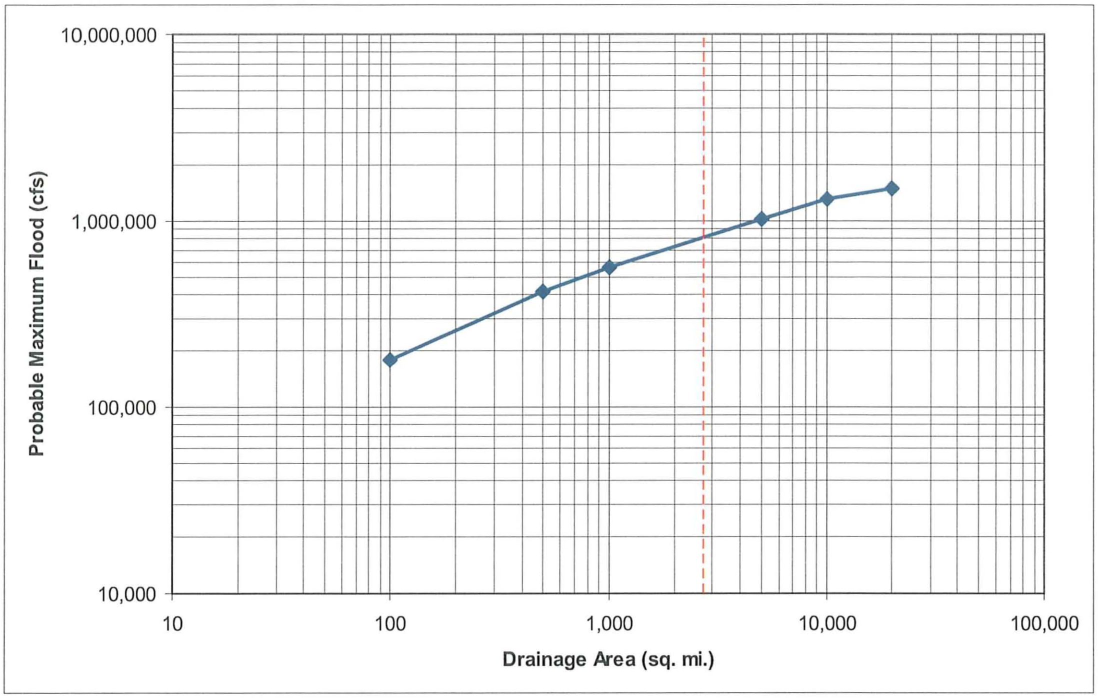

Figure 7-26. PMF Enveloping Isolines Based on Cedar Ridge Reservoir Dam Location

According to the Brazos G Regional Water Plan (Reference 4), the Cedar Ridge Reservoir Dam drainage area is 2,748 sq. mi. The vertical dashed line in Figure 7-26 is located at the corresponding drainage area value. The resulting PMF flow read from Figure 7-26 is 810,000 cfs.

Fort Phantom Hill Dam is located upstream from the Cedar Ridge Reservoir. Resulting flows from Fort Phantom Hill Dam are applied to the Cedar Ridge Reservoir without any attenuation. The Fort Phantom Hill Dam PMF flow of 410,000 cfs is part of the 810,000 cfs PMF determined for the Cedar Ridge Reservoir Dam. Therefore, the added PMF effect at Cedar Ridge Reservoir is 810,000 cfs – 410,000 cfs = 400,000 cfs. The total flow applied to the Cedar Ridge Reservoir Dam to determine the water surface elevation is 760,000 cfs + 400,000 cfs = 1,160,000 cfs. Alternatively, to determine the water surface elevation a breach wave height of 33.4 ft. is applied to the Cedar Ridge Reservoir Dam along with the added PMF effect of 400,000 cfs.

According to the Brazos G Regional Water Plan (Reference 4), Cedar Ridge Reservoir will inundate approximately 6,635 ac. at the normal full pool elevation of 1,489.0 ft. No other specific details for the proposed dam have been developed. The dam would be expected to be higher than the full pool elevation to accommodate flood runoff to some degree. Spillway details have not been developed. Therefore, it is unknown how high above the full pool elevation the dam may be constructed. According to the USGS quadrangle (Reference 32) Lueders East, TX and using the 1,490 ft. contour as a basis for the full pool elevation, the upstream end of the reservoir backs up to a small instream dam that has a spillway elevation of 1,512 ft. The upstream Penick Dam is not included in the National Atlas (Reference 13). Based on the National Inventory of Dams database (Reference 25), Penick Dam is 25 ft. high, 330 ft. long, and has a storage capacity of 375 ac.-ft. The dam and reservoir are small enough to discount for the dam failure analysis.

It would be unlikely that the Cedar Ridge Reservoir Dam would exceed the Penick Dam spillway elevation. Therefore, it is assumed that the Cedar Ridge Reservoir Dam crest is at elevation 1,510.0 ft., which is 1,510.0 ft. – 1,489.0 ft. = 21 ft. above the normal full pool elevation. This is consistent with other dams in the region such as Lake Stamford Dam (1,436.8 ft. – 1,416.8 ft. = 20 ft.), Fort Phantom Hill Dam (1,650.0 ft. – 1,635.9 ft. = 14.1 ft.), and Hubbard Creek Dam (1,208.0 ft. – 1,183.0 ft. = 25 ft.). Figure 7-27 indicates a more likely position for the dam, which minimizes the length of the dam. This is a conservative assumption because a shorter length of dam will maximize the overtopping depth of flow for the dam failure analysis. The crest length is estimated to be 4,965 ft.

Also shown on Figure 7-27 is the channel distance and the elevations used to determine the maximum height of the dam. The channel drops 10 ft. over a distance of 10,099 ft. The dam is 3,436 ft. downstream from elevation 1,370 ft. Therefore, the channel bottom at the dam is 1,370 ft. – 3,436 ft. / 10,099 ft. * 10 ft. = 1,367 ft. The maximum height of the dam is 1,510.0 ft. – 1,367 ft. = 143 ft.

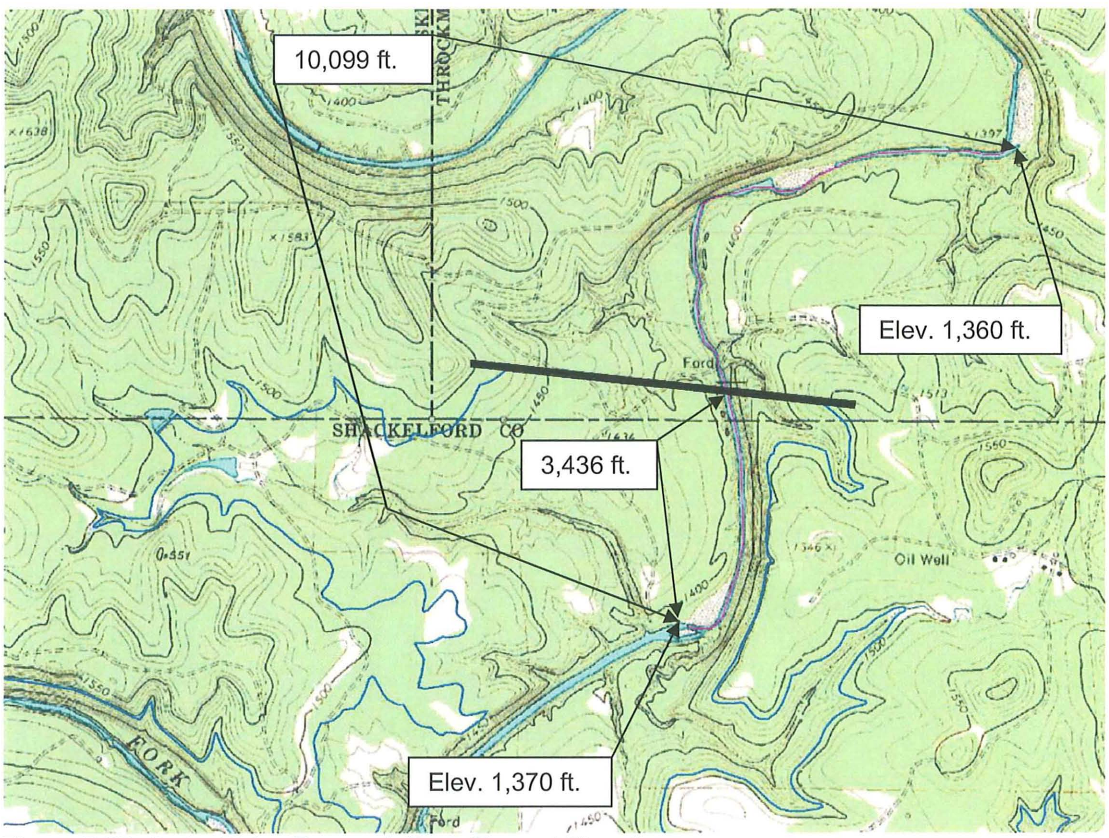

Figure 7-27. Cedar Ridge Reservoir Dam Channel Elevations

The 7.5 minute USGS quadrangles (Reference 32) for Antelope Hills, TX, Lueders NE, TX, Lueders East, TX, and Collins Creek SW, TX are inserted into AutoCAD (Reference 2) to compare the surface area using the 1,490 ft. contour to the surface area approximated by the water plan. At the estimated position for the dam and accounting for island features in the reservoir, the surface area is 6,561 ac. as shown on Figure 7-28. The surface area could be made to exceed the water plan approximation by moving the position of the dam downstream several hundred feet. However, this would also increase the length of the dam. Therefore, by evaluation of the known data and the USGS quadrangles, the estimated position of the dam and the resulting characteristics are accepted for the dam failure analysis.

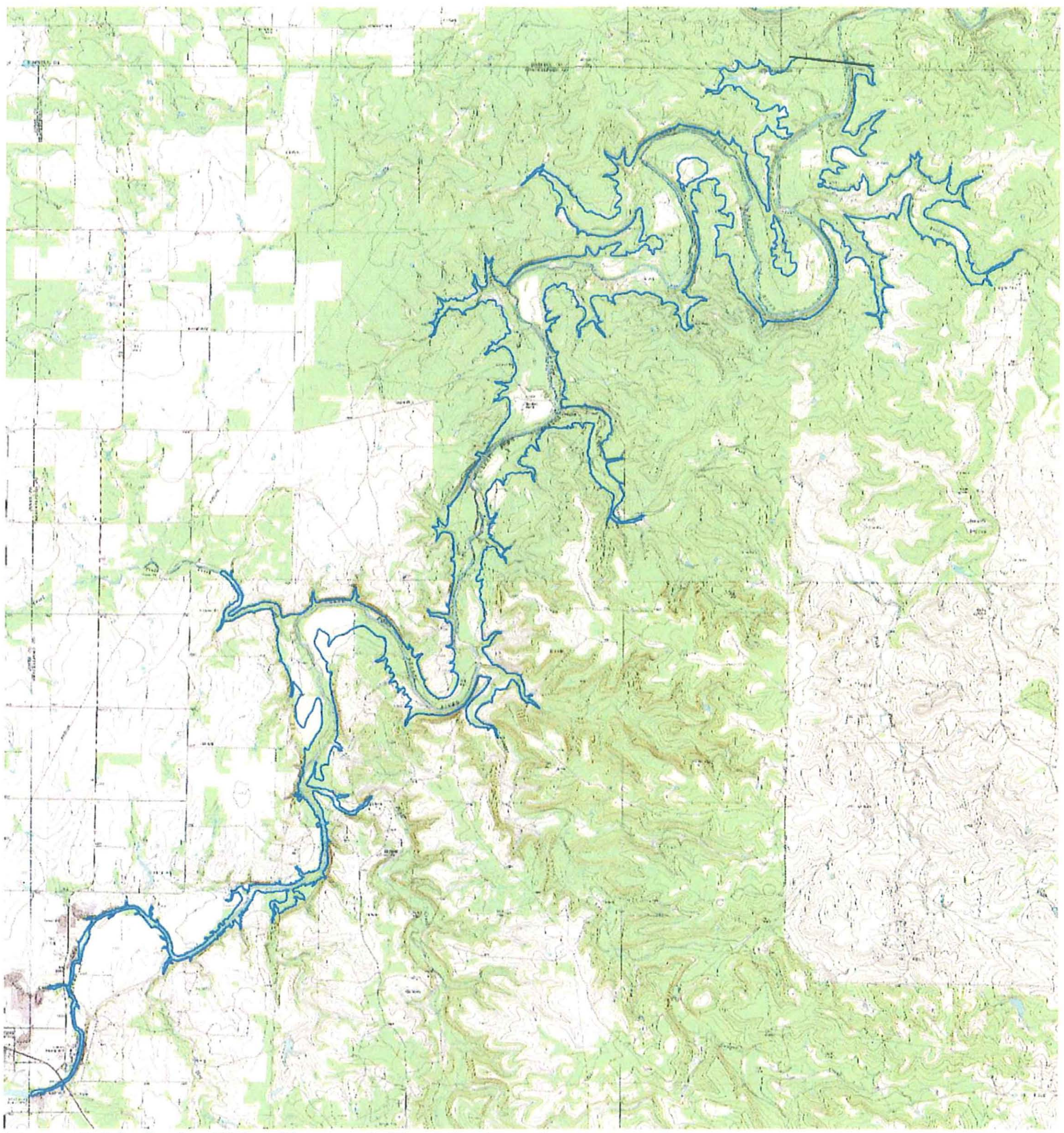

Figure 7-28. Cedar Ridge Reservoir Dam Surface Area

As the PMF including the effects of failure from Fort Phantom Hill Dam is applied, it is assumed that Cedar Ridge Reservoir is full up to the crest elevation of 1,510.0 ft. The water surface elevation is determined based on the PMF spilling over the dam crest. Overtopping is modeled using the standard broad crested weir flow equation defined by the HEC-RAS reference manual (Reference 6, Equation 6-14).

$Q = C * L * H^{1.5}$

As previously discussed in Section 6.0, this calculation utilizes the HEC-RAS reference manual recommended weir flow coefficient of 2.6.

Based on the 4,965 ft. dam crest at elevation 1,510.0 ft., the overtopping elevation is determined for the PMF of 1,160,000 cfs, including the effects of failure from Fort Phantom Hill Dam.

$Q = 2.6 * \text{crest length} * (\text{overtopping elevation} - \text{crest elevation})^{1.5}$

$1{,}160{,}000 \text{ cfs} = 2.6 * 4{,}965 \text{ ft.} * (Z - 1{,}510.0 \text{ ft.})^{1.5}$

Solving for overtopping elevation, Z = 1,530.06 ft. = 1,530.1 ft.

For the purpose of dam failure evaluation, it is conservative to round up because it results in a higher headwater elevation. A higher headwater elevation will maximize the water height component of the dam failure equation and the resulting dam failure flow.

Alternatively, a breach wave height of 33.4 ft. is applied to the Cedar Ridge Reservoir Dam along with the added PMF effect of 400,000 cfs. An overtopping height of 33.4 ft. results in an overtopping elevation of 1,510 ft. + 33.4 ft. = 1,543.4 ft. The overtopping flow is first determined for the breach wave height representing the effects of Fort Phantom Hill Dam failure.

$Q = 2.6 * \text{crest length} * (\text{overtopping elevation} - \text{crest elevation})^{1.5}$

$Q = 2.6 * 4{,}965 \text{ ft.} * (1{,}543.4 - 1{,}510.0 \text{ ft.})^{1.5} = 2{,}491{,}795 \text{ cfs} = 2{,}500{,}000 \text{ cfs}$

To include the added Cedar Ridge Reservoir PMF effect of 400,000 cfs, the overtopping height is determined for a total flow of 2,500,000 cfs + 400,000 cfs = 2,900,000 cfs.

$2{,}900{,}000 \text{ cfs} = 2.6 * 4{,}965 \text{ ft.} * (Z - 1{,}510.0 \text{ ft.})^{1.5}$

Solving for overtopping elevation, Z = 1,546.95 ft. = 1,547.0 ft.

Tailwater is determined for both the transposed breach flow with added PMF effects of 1,160,000 cfs and the transposed breach wave height with added PMF effects corresponding to an overtopping flow of 2,900,000 cfs. The 7.5 minute USGS quadrangle (Reference 32) for Antelope Hills, TX was inserted into AutoCAD (Reference 2) to determine channel distances, slope, and cross section elevations. Figure 7-29 identifies the selected cross section in relationship to the dam and the channel distances used to determine the slope and elevations.

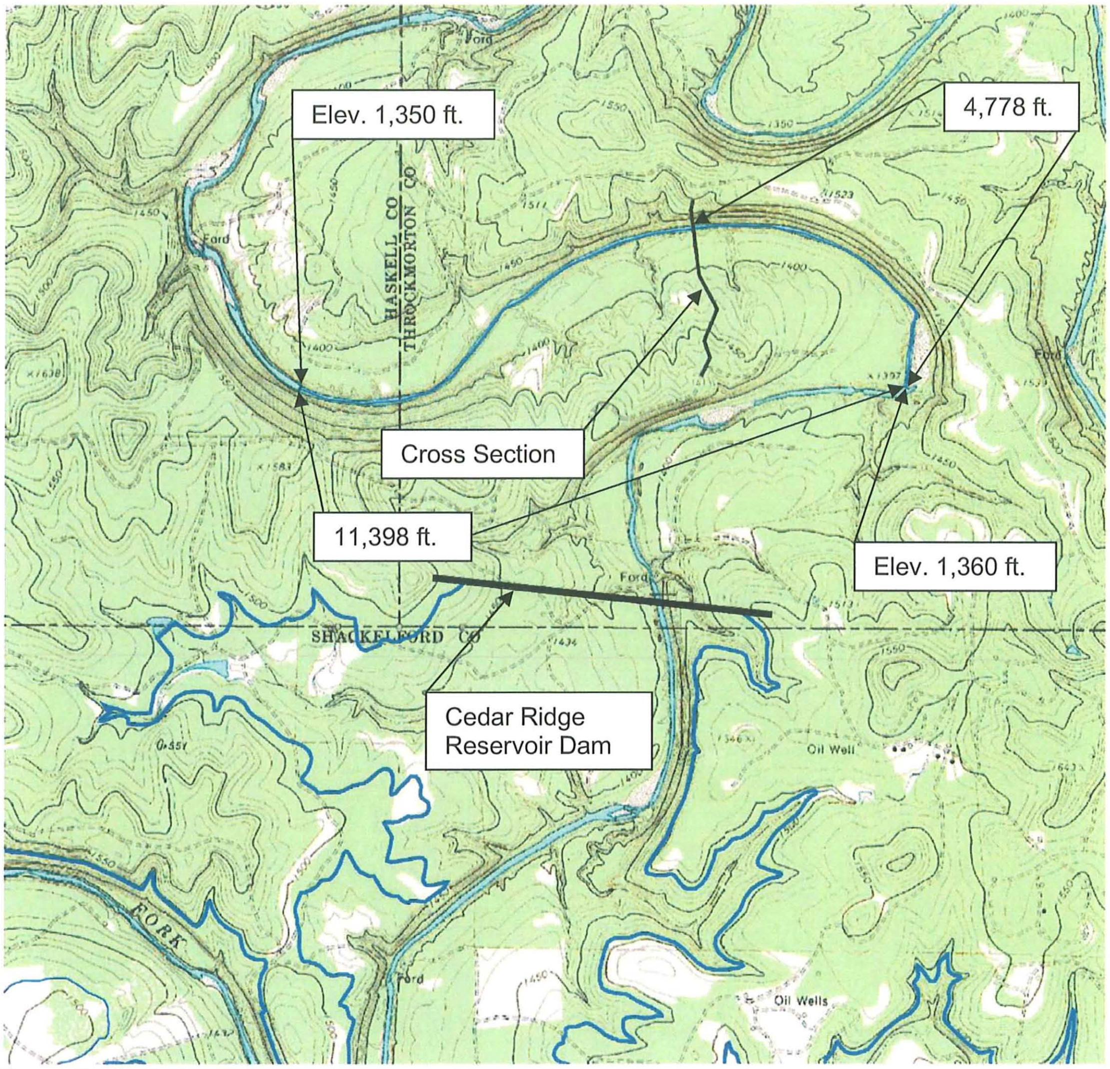

Figure 7-29. Cedar Ridge Reservoir Dam Downstream

As shown in Figure 7-29, the channel drops 10 ft. over a distance of 11,398 ft. Therefore, the channel slope is 10 ft. / 11,398 ft. = 0.0009 ft./ft. The cross section is 4,778 ft. downstream from elevation 1,360 ft. Therefore, the cross section bottom is 4,778 ft. / 11,398 ft. * 10 ft. = 4 ft. lower than elevation 1,360 ft. The cross section station and elevations are provided in Table 7-11.

Table 7-11. Cedar Ridge Reservoir Dam Tailwater Section Coordinates

Station (ft.)	Elevation (ft.)	Station (ft.)	Elevation (ft.)
-2,288	1,480	-84	1,370
-1,863	1,470	-53	1,360
-1,697	1,460	-31	1,356
-1,586	1,450	36	1,356
-1,467	1,440	50	1,360
-1,126	1,430	145	1,400
-978	1,420	214	1,450
-757	1,410	252	1,460
-578	1,400	316	1,470
-249	1,390	429	1,500
-118	1,380		

Stationing from left to right when looking downstream

Tailwater depth is determined using FlowMaster (Reference 3) and the Manning friction formula. From above the two flows of 1,160,000 cfs and 2,900,000 cfs were examined with a slope of 0.0009 ft./ft. As previously discussed in Section 6.0, the Manning coefficient of 0.025 is applied to the channel and overbank areas.

The 1,160,000 cfs flow depth for the cross section is determined to be 85.7 ft. The 2,900,000 cfs flow depth for the cross section is determined to be 115.35 ft. and is rounded down to 115.3 ft. This is conservative because it results in a lower tailwater elevation as discussed above. The FlowMaster results are provided in Appendix E. Therefore, the tailwater elevation at the downstream cross section is 1,356 ft. + 85.7 ft. = 1,441.7 ft. for a flow of 1,160,000 cfs and 1,356 ft. + 115.3 ft. = 1,471.3 ft. for a flow of 2,900,000 cfs. Level pool from the cross section upstream to the dam is assumed. This assumption neglects any increase to the tailwater elevation based on backwater effects. The tailwater elevation in both cases is well below the crest elevation of 1,510.0 ft. Therefore, the overtopping discharge is determined to be independent of tailwater. The cross section and tailwater elevation are shown on Figure 7-30.

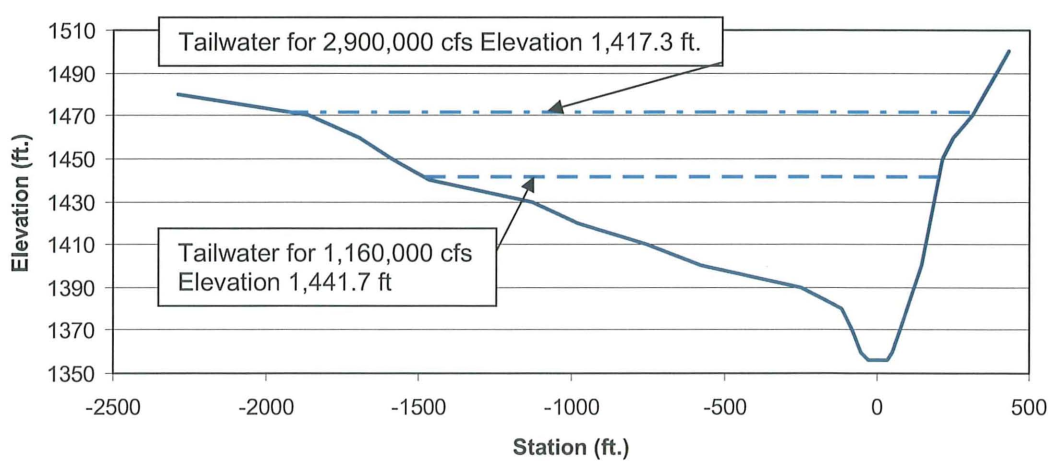

Figure 7-30. Cedar Ridge Reservoir Dam Tailwater

Of the two scenarios examined, the breach wave height resulting in an overtopping flow of 2,900,000 cfs creates the higher headwater elevation. Wind setup for both scenarios is based on the higher headwater elevation which will produce the longer fetch distance.

According to USACE EM 1110-2-1420 (Reference 22) wind setup can be reasonably estimated for lakes and reservoirs using the following equation:

$$S = U^2 * F / (1{,}400 * D)$$

USACE EM 1110-2-1420 (Reference 22) indicates that the fetch distance is usually satisfactorily assumed to be two times the effective fetch distance. A straight line fetch is used to define the wind setup and is more conservative than an effective fetch.

As previously discussed, ANSI/ANS-2.8-1992 (Reference 1) is used to define the coincident wind speed. From Figure 7-5, the Annual Extreme-Mile, 30 ft. Above Ground, 2-yr. Mean Recurrence Interval is between 50 mph and 60 mph for the Brazos River watershed upstream from Whitney Lake. The more conservative wind speed of 60 mph is used to generate wind setup.

The controlling overtopping elevation at Cedar Ridge Reservoir Dam is determined to be 1,547.0 ft. The fetch length is determined from the reservoir surface area at the overtopping elevation. The 7.5 minute USGS quadrangles (Reference 32) for Antelope Hills, TX, Lueders NE, TX, Lueders East, TX, and Collins Creek SW, TX are inserted into AutoCAD (Reference 2) and because only contours with 10 ft. intervals are identified on the quadrangles the 1,550 ft. contour is used to determine the surface area. As shown on Figure 7-31, the longest straight line fetch distance is determined to be 35,957 ft. (rounded up to 6.9 mi.).

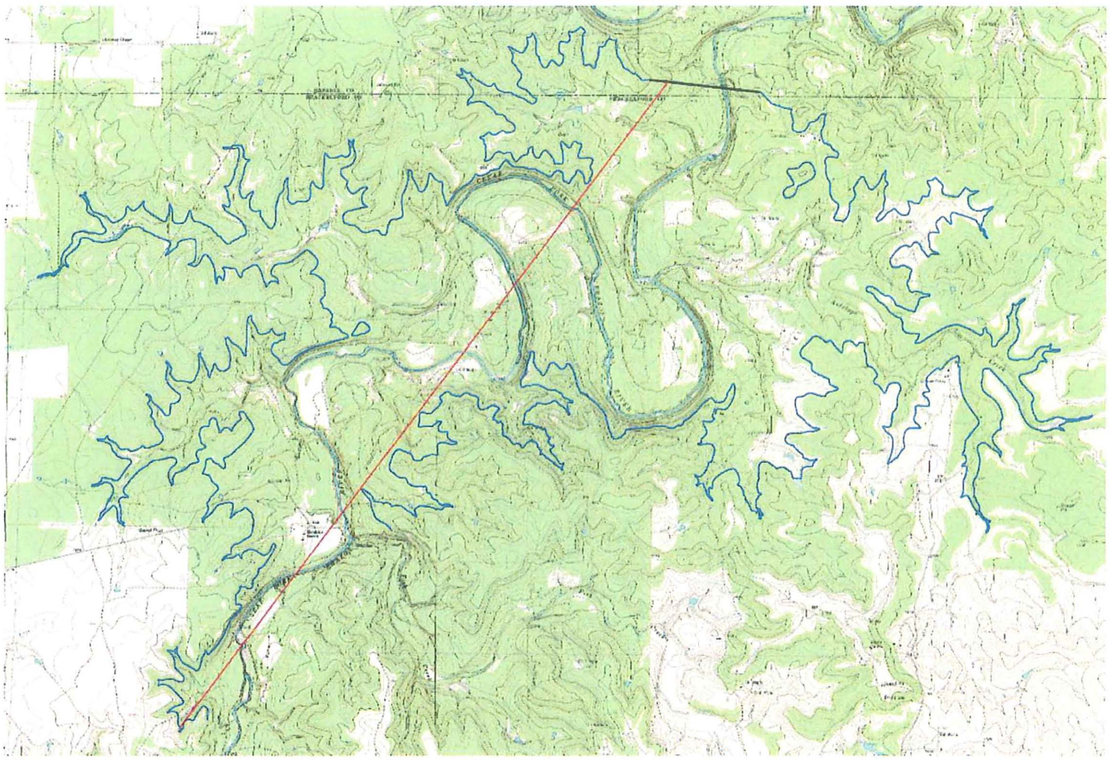

Figure 7-31. Cedar Ridge Reservoir Dam Fetch Length

A bottom surface profile along the fetch distance is created using the USGS quadrangles (Reference 32) and is provided in Figure 7-32. The data for the distance and elevations are tabulated in Table 7-12. An average depth along the fetch distance is determined using the data in Table 7-12 and the following formula for hydraulic depth:

$$E = \frac{\left(\frac{Y_1 + Y_2}{2}\right) * \left(X_2 - X_1\right) + \ldots + \left(\frac{Y_{n-1} + Y_n}{2}\right) * \left(X_n - X_{n-1}\right)}{X_n - X_1}$$

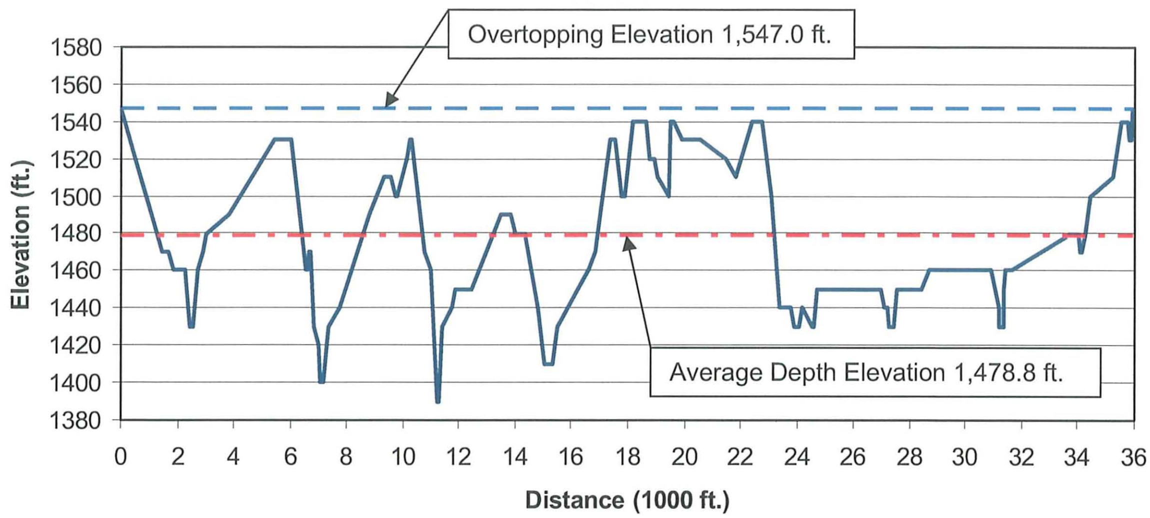

Figure 7-32. Cedar Ridge Reservoir Dam Bottom Surface Profile

Table 7-12. Cedar Ridge Reservoir Dam Bottom Surface Profile Section Coordinates

Distance (ft.)	Elevation (ft.)	Distance (ft.)	Elevation (ft.)	Distance (ft.)	Elevation (ft.)
0	1,547.0	11,421	1,430	23,899	1,430
1,437	1,470	11,735	1,440	24,100	1,430
1,674	1,470	11,849	1,450	24,199	1,440
1,864	1,460	12,460	1,450	24,534	1,430
2,235	1,460	13,476	1,490	24,594	1,430
2,434	1,430	13,826	1,490	24,694	1,450
2,528	1,430	14,023	1,480	26,964	1,450
2,707	1,460	14,337	1,480	27,079	1,440
2,875	1,470	14,844	1,440	27,225	1,440
2,991	1,480	15,043	1,410	27,284	1,430
3,826	1,490	15,323	1,410	27,429	1,430
5,427	1,530	15,485	1,430	27,552	1,450
6,010	1,530	16,596	1,460	28,428	1,450
6,538	1,460	16,861	1,470	28,720	1,460
6,585	1,460	17,381	1,530	30,883	1,460
6,654	1,470	17,557	1,530	31,192	1,440
6,708	1,470	17,792	1,500	31,224	1,430
6,807	1,430	17,895	1,500	31,362	1,430
6,999	1,420	18,174	1,540	31,388	1,450
7,038	1,400	18,654	1,540	31,432	1,460
7,180	1,400	18,781	1,520	31,680	1,460
7,336	1,430	18,927	1,520	33,763	1,480
7,731	1,440	19,045	1,510	34,046	1,480
8,781	1,490	19,474	1,500	34,086	1,470
9,332	1,510	19,533	1,540	34,152	1,470
9,532	1,510	19,647	1,540	34,258	1,480
9,697	1,500	19,916	1,530	34,430	1,500
9,798	1,500	20,565	1,530	35,230	1,510
10,132	1,520	21,479	1,520	35,517	1,540
10,223	1,530	21,793	1,510	35,744	1,540
10,327	1,530	22,379	1,540	35,819	1,530
10,779	1,470	22,745	1,540	35,881	1,530
10,977	1,460	23,090	1,500	35,957	1,547.0
11,202	1,390	23,391	1,440		
11,307	1,390	23,793	1,440		

Note: Distance 0 ft. is at the dam.

The average depth bottom surface elevation is calculated to be 1,478.8 ft. The overtopping water surface elevation is 1,547.0 ft. Therefore, the average depth along the fetch distance is calculated to be 1,547.0 ft. – 1,478.8 ft. = 68.2 ft. From above, the wind speed is 60 mph and the fetch distance is 6.9 mi. Wind setup is calculated as follows:

$S = (60\ \text{mph})^2 * (6.9\ \text{mi.}) / (1{,}400 * 68.2\ \text{ft.}) = 0.26\ \text{ft.}$

Setup is conservatively rounded up to 0.3 ft. For the purpose of dam failure evaluation, it is conservative to round up because it results in a higher headwater elevation. A higher headwater elevation will maximize the water height component of the dam failure equation and the resulting dam failure flow. For the 1,160,000 cfs overtopping scenario, the PMF headwater elevation at Cedar

Ridge Reservoir Dam including wind setup is 1,530.1 ft. + 0.3 ft. = 1,530.4 ft. For the 2,900,000 cfs overtopping scenario, the PMF headwater elevation at Cedar Ridge Reservoir Dam including wind setup is 1,547.0 ft. + 0.3 ft. = 1,547.3 ft.

Dam Failure Cedar Ridge Reservoir Dam

As previously discussed, dam failure is evaluated based on two methods. As identified in ANSI/ANS-2.8-1992 (Reference 1, Section 5.1.3.2) the breach wave height is computed as h = 4 * (headwater – tailwater) / 9 and transposed downstream without attenuation. Alternatively, dam failure flow is calculated using a USACE dam breach equation (Reference 24) and USACE RD-13 breach parameters (Reference 23).

As identified above, the dam is 143 ft. tall with a crest elevation at 1,510.0 ft. For the 1,160,000 cfs overtopping scenario, the headwater is determined to be 1,530.4 ft. and the tailwater is determined to be 1,441.7 ft. The breach wave height is calculated as follows:

h = 4 * (1,530.4 ft. – 1,441.7 ft.) / 9 = 39.42 ft., rounded up to 39.5 ft.

For the 2,900,000 cfs overtopping scenario, the headwater is determined to be 1,547.3 ft. and the tailwater is determined to be 1,471.3 ft. The breach wave height is calculated as follows:

h = 4 * (1,547.3 ft. – 1,471.3 ft.) / 9 = 33.77 ft., rounded to 33.8 ft.

Breach parameters for an earth fill dam are determined using USACE RD-13 (Reference 23, Table 1, Page 17). The greatest breach width and side slopes maximize the resulting breach flow. The breach width, W_b, is three times the dam height of 143 ft., and the side slopes of the breach are 1:1 (horizontal:vertical).

Therefore, W_b = 3 * height of dam = 3 * 143 ft. = 429 ft.

HEC-HMS version 2.2.0 release notes (Reference 24, Page 8) are used for the dam break equation including side slopes.

$Q = 1.7 * W_b * h^{1.5} + 1.35 * S * h^{2.5}$

The water height is equal to the smaller of the head difference between headwater and tailwater or the head difference between headwater and the breach bottom invert elevation. For the 1,160,000 cfs overtopping scenario, the difference between the headwater and tailwater is 1,530.4 ft. – 1,441.7 ft. = 88.7 ft. The difference between the headwater and breach bottom is greater than the full height of the dam, 143 ft. Therefore, the breach flow is calculated using the difference between the headwater and tailwater as follows:

$Q = 1.7 * 429\text{ ft.} * (88.7\text{ ft.})^{1.5} + 1.35 * 1 * (88.7\text{ ft.})^{2.5}$ = 709,277 cfs, rounded up to 710,000 cfs.

The total flow is assumed to be the sum of the overtopping flow previously determined added to the breach flow. There is no reduction in overtopping flow to account for the breached section of the dam. Therefore, the total flow from the failure scenario is: Q = 710,000 cfs + 1,160,000 cfs = 1,870,000 cfs.

For the 2,900,000 cfs overtopping scenario, the difference between the headwater and tailwater is 1,547.3 ft. – 1,471.3 ft. = 76.0 ft. The difference between the headwater and breach bottom is greater than the full height of the dam, 143 ft. Therefore, the breach flow is calculated using the difference between the headwater and tailwater as follows:

$Q = 1.7 * 429 \text{ ft.} * (76.0 \text{ ft.})^{1.5} + 1.35 * 1 * (76.0 \text{ ft.})^{2.5} = 551{,}178$ cfs, rounded up to 560,000 cfs.

The total flow is assumed to be the sum of the overtopping and spillway flow previously determined added to the breach flow. There is no reduction in overtopping flow to account for the breached section of the dam. Therefore, the total flow from the failure scenario is: Q = 560,000 cfs + 2,900,000 cfs = 3,460,000 cfs.

In summary, the critical potential scenarios for the Cedar Ridge Reservoir Dam failure effects, including the domino-type failure from the upstream Fort Phantom Hill Dam, transposed downstream without attenuation are determined to be a breach wave height of 39.5 ft., or a total breach flow of 3,460,000 cfs.

The failure effects from the Lake Stamford Dam failure are added to the Cedar Ridge Reservoir Dam failure effects to account for simultaneous failure of the two dams. Therefore, the total breach wave height is 39.5 ft. + 17.8 ft. = 57.3 ft. or the total breach flow is 3,460,000 cfs + 470,000 cfs = 3,930,000 cfs and transposed downstream without attenuation to Morris Sheppard Dam.

<u>PMF Brazos River</u>

Based on the NID database (Reference 25), Morris Sheppard Dam is located at the coordinates, latitude 32.8711° and longitude -98.4261°, and De Cordova Bend Dam is located at the coordinates, latitude 32.3733° and longitude -97.6883°.

Similar to the process for Hubbard Creek Dam, the NRC RG 1.59 (Reference 34) method is used to determine the PMF on the Brazos River. However, the PMF is based on the location of both Morris Sheppard Dam and De Cordova Bend Dam utilizing the charts of enveloping PMF isolines for various watershed drainage areas. Figure 7-33 is a typical chart showing the location of both dams. Table 7-13 presents the results for each chart corresponding to the drainage area provided. Straight line interpolation is used between isolines.

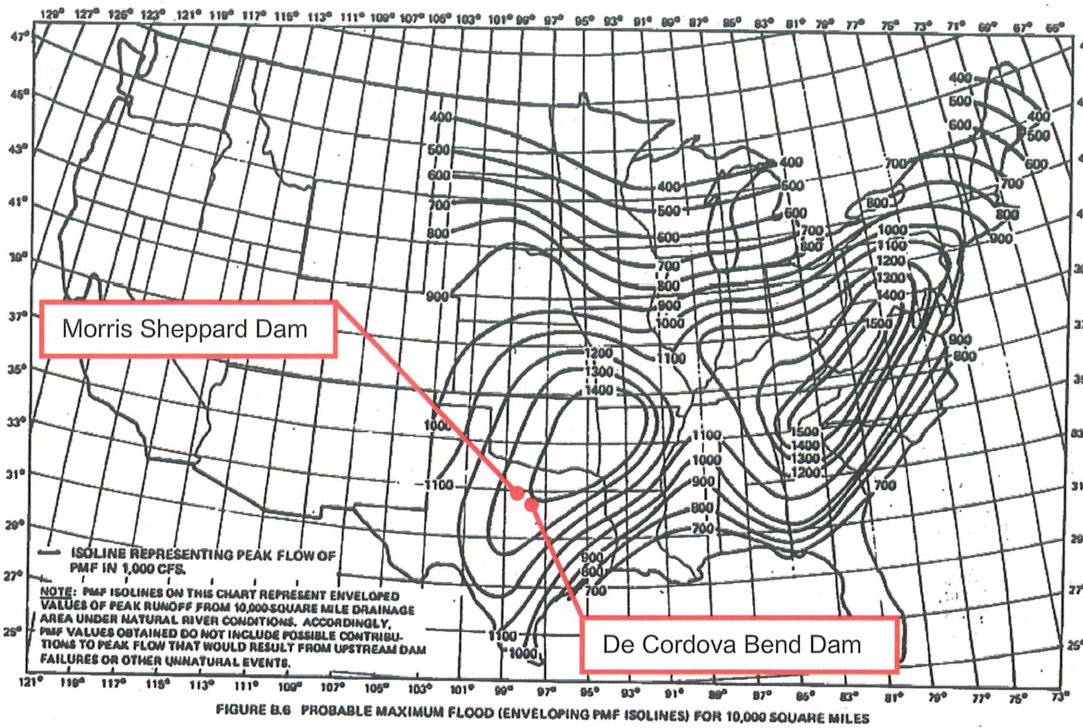

Figure 7-33. RG 1.59 10,000 sq. mi. Enveloping PMF Isolines Brazos River Dams

Table 7-13. RG 1.59 Appendix B PMF Results for the Brazos River

RG 1.59 chart	Drainage Area (sq. mi.)	PMF (cfs) Morris Sheppard Dam location	PMF (cfs) De Cordova Bend Dam location
B2	100	195,000	200,000
B3	500	440,000	450,000
B4	1,000	600,000	600,000
B5	5,000	1,065,000	1,080,000
B6	10,000	1,360,000	1,380,000
B7	20,000	1,500,000	1,500,000

Because the results for both locations were similar, the higher results for De Cordova Bend Dam were plotted on a log-log scale and a smooth curve was fitted to the points, as shown in Figure 7-34.

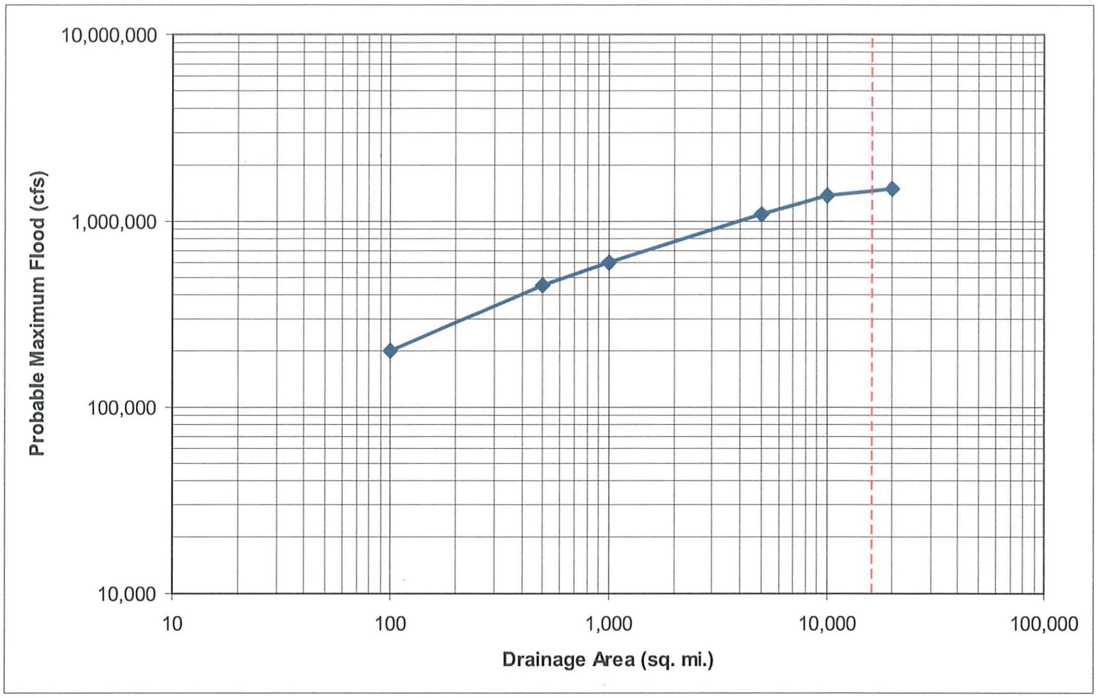

Figure 7-34. PMF Enveloping Isolines Based on Morris Sheppard Dam and De Cordova Bend Dam Location

According to the Texas Water Development Board volumetric survey (Reference 21), the Morris Sheppard Dam contributing drainage area is 13,310 sq. mi. According to the Texas Water Development Board volumetric survey (Reference 19), the De Cordova Bend Dam contributing drainage area is 16,113 sq. mi. Because both areas are close in magnitude, the larger area of De Cordova Bend Dam is used to determine the PMF. The vertical dashed line in Figure 7-34 is located at the corresponding drainage area value. The PMF is determined to be 1.45 million cfs.

Dam Failure Morris Sheppard Dam

Based on the qualitative analysis described above, there are two scenarios for further evaluation. Under existing conditions, the limiting scenario is the domino-type failure of Hubbard Creek Dam, Morris Sheppard Dam, and De Cordova Bend Dam. Under future conditions, the limiting scenario is the domino-type failure of Fort Phantom Hill Dam, Cedar Ridge Reservoir Dam, Morris Sheppard Dam, and De Cordova Bend Dam along with the failure of Lake Stamford Dam simultaneous with the failure of Cedar Ridge Reservoir.

Based on the existing conditions qualitative analysis for Hubbard Creek Dam, the limiting scenario failure effects transposed downstream without attenuation to Morris Sheppard Dam are determined to be a main dam breach wave height of 35.5 ft. or a fuse plug breach flow of 2,270,000 cfs. Based on the future conditions qualitative analyses for Fort Phantom Hill Dam, Cedar Ridge Reservoir Dam, and Lake Stamford Dam, the limiting scenario failure effects transposed downstream without attenuation to Morris Sheppard Dam are determined to be a breach wave height of 57.3 ft. or a breach flow of 3,930,000 cfs.

Because both the existing conditions and future conditions involve upstream failure effects on Morris Sheppard Dam, the more critical breach wave height and breach flow will be evaluated further. The future conditions dam failure scenario including domino-type failure of Fort Phantom Dam and Cedar Ridge Reservoir Dam simultaneous with the failure of Lake Stamford Dam has both a more critical breach wave height and breach flow.

The PMF for the Brazos River is combined with the total breach flows from Cedar Ridge Reservoir Dam, including the effects of Fort Phantom Hill Dam, and Lake Stamford Dam without any attenuation, and applied to Morris Sheppard Dam. Dam breach results are then applied to De Cordova Dam without any attenuation. The total flow applied to Morris Sheppard Dam to determine the water surface elevation is 1,450,000 cfs + 3,930,000 cfs. = 5,380,000 cfs. Alternatively, to determine the water surface elevation a breach wave height of 57.3 ft. is applied to the Morris Sheppard Dam along with the added PMF effect of 1,450,000 cfs.

According to the Texas Water Development Board volumetric survey (Reference 21), Morris Sheppard Dam is a concrete buttress dam with earthen dikes and has a maximum height of 189 ft. or elevation 1,024.0 ft. The service spillway is gate controlled with an ogee crest elevation of 987.0 ft. and the top of gates elevation of 1,000.0 ft. The dam impounds Possum Kingdom Lake at a normal pool elevation of 1,000.0 ft.

According to the Brazos River Authority Morris Sheppard Dam Breach Analysis Report (Reference 10), the total length of the concrete buttress section is 1,640 ft. At the right abutment, the dam continues with a 1,107 ft. long earthen dike with a concrete core wall. In 1991 a 1,400 ft. long emergency spillway at elevation 1,000.0 ft. was completed at the south end of the concrete core wall. Previous PMF studies showed a peak stage of elevation 1031.6 ft. resulting in overtopping of the core wall by 3.6 ft. From this information, it is determined the top elevation of the concrete core wall is 1031.6 ft. – 3.6 ft. = 1,028 ft.

According to the Federal Energy Regulatory Commission Environmental Use and Inspection Report (Reference 9), the spillway length is 707 ft. with 9-73.6 ft. wide gates. Therefore, the length of the top of gates at elevation 1,000 ft. is 9 * 73.6 ft. = 662.4 ft., rounded to 662 ft. The remaining 1,640 ft. – 662 ft. = 978 ft. of the concrete buttress dam has a crest elevation of 1,024.0 ft.

According to the USGS gauge 08088500 Water-Data Report 2008 (Reference 27), the maximum recorded elevation for the reservoir is 1,003.60 ft. and occurred prior to completion of the emergency spillway.

In summary, the lowest portions of the dam are the service spillway with gates closed and the emergency spillway at elevation 1,000.0 ft. totaling 662 ft. + 1,400 ft. = 2,062 ft. in length. The buttress sections at elevation 1,024.0 ft are 978 ft. in length and the earthen embankment section with the concrete core wall at elevation 1,028.0 ft. is 1,107 ft. in length. Morris Sheppard Dam is shown in Figure 7-35. The embankment section extends beyond the view shown in the figure. The emergency spillway section is not shown.

Figure 7-35. Morris Sheppard Dam (Reference 5)

As the PMF including the effects of upsteam dam failures is applied, it is assumed that the spillway gates are closed and that Possum Kingdom Lake is above the emergency spillway elevation at the historical maximum recorded elevation of 1,003.6 ft. Overtopping is modeled using the standard broad crested weir flow equation defined by the HEC-RAS reference manual (Reference 6, Equation 6-14).

$Q = C * L * H^{1.5}$

As previously discussed in Section 6.0, this calculation utilizes the HEC-RAS reference manual recommended weir flow coefficient of 2.6.

The overtopping flow for the maximum historical elevation is first determined using the broad crested weir flow equation. From above, the weir length at the lowest elevation of 1,000 ft. is 2,062 ft.

$Q = 2.6 * 2{,}062 \text{ ft.} * (1{,}003.6 \text{ ft.} - 1{,}000 \text{ ft.})^{1.5}$ = 36,620 cfs, rounded up to 40,000 cfs.

The total flow to consider is the sum of the flow generated by the maximum historical elevation and the flow from the upstream dam failures and combined PMF. Therefore, the total flow is: Q = 5,380,000 cfs + 40,000 cfs = 5,420,000 cfs.

Based on the configuration of the dam crests at different elevations the weir flow equation is modified to include the service spillway crest with gates closed and emergency spillway crest elevation, the dam crest elevation, and the embankment core wall elevation. The overtopping elevation is determined for the upstream dam failures and combined PMF of 5,420,000 cfs.

$Q = C * Ls * (Z - Es)^{1.5} + C * Lc * (Z - Ec)^{1.5} + C * Lw * (Z - Ew)^{1.5}$

Where:
Q = flow (cfs)
C = weir flow coefficient
Ls = length of the service spillway crest and emergency spillway crest (ft.)
Es = elevation of the service spillway crest, gates closed, and emergency spillway crest (ft.)
Lc = length of the buttress dam crest minus the spillway length (ft.)
Ec = elevation of the buttress dam crest (ft.)
Lw = length of the embankment core wall crest (ft.)
Ew = elevation of the embankment core wall crest (ft.)
Z = overtopping water surface elevation (ft.)

From above:
Ls = 2,062 ft.
Es = 1,000.0 ft.
Lc = 978 ft.
Ec = 1,024.0 ft.
Lw = 1,107 ft.
Ew = 1,028.0 ft.

Therefore, the weir flow equation is:

$$5{,}420{,}000 \text{ cfs} = 2.6 * 2{,}062 \text{ ft.} * (Z - 1{,}000.0 \text{ ft.})^{1.5} + 2.6 * 978 \text{ ft.} * (Z - 1{,}024.0 \text{ ft.})^{1.5} + 2.6 * 1{,}107 \text{ ft.} * (Z - 1{,}028.0 \text{ ft.})^{1.5}$$

Solving for overtopping elevation, Z = 1,075.67 ft. = 1,075.7 ft.

For the purpose of dam failure evaluation, it is conservative to round up because it results in a higher headwater elevation. A higher headwater elevation will maximize the water height component of the dam failure equation and the resulting dam failure flow.

Alternatively, a breach wave height of 57.3 ft. is applied to the Morris Sheppard Dam along with the added PMF effect of 1,450,000 cfs. A breach wave height of 57.3 ft. results in an overtopping elevation of 1,003.6 ft. + 57.3 ft. = 1,060.9 ft. The overtopping flow is first determined for the breach wave height representing the effects of the upstream dam failures.

$$Q = C * Ls * (Z - Es)^{1.5} + C * Lc * (Z - Ec)^{1.5} + C * Lw * (Z - Ew)^{1.5}$$

$$Q = 2.6 * 2{,}062 \text{ ft.} * (1{,}060.9 \text{ ft.} - 1{,}000.0 \text{ ft.})^{1.5} + 2.6 * 978 \text{ ft.} * (1{,}060.9 \text{ ft.} - 1{,}024.0 \text{ ft.})^{1.5} + 2.6 * 1{,}107 \text{ ft.} * (1{,}060.9 \text{ ft.} - 1{,}028.0 \text{ ft.})^{1.5} = 3{,}661{,}046 \text{ cfs rounded up to } 3{,}670{,}000 \text{ cfs.}$$

To include the added PMF effect of 1,450,000 cfs, the overtopping height is determined for a total flow of 3,670,000 cfs + 1,450,000 cfs = 5,120,000 cfs.

$$5{,}120{,}000 \text{ cfs} = 2.6 * 2{,}062 \text{ ft.} * (Z - 1{,}000.0 \text{ ft.})^{1.5} + 2.6 * 978 \text{ ft.} * (Z - 1{,}024.0 \text{ ft.})^{1.5} + 2.6 * 1{,}107 \text{ ft.} * (Z - 1{,}028.0 \text{ ft.})^{1.5}$$

Solving for overtopping elevation, Z = 1,073.29 ft. = 1,073.3 ft.

Based on the 7.5 minute USGS quadrangle (Reference 32) for Fortune Bend, TX, as shown in Figure 7-36, it is physically possible to achieve the calculated elevations. A near vertical relief occurs at the left abutment up to at least an elevation of 1,100.0 ft. The right abutment slopes up to 1,067.0 ft., slopes down to about 1,045.0 ft. then back up to above 1,100.0 ft. The calculation considers flow overtops only the buttress, spillway, and embankment structures. The areas at the right abutment below the calculated overtopping elevations would carry some flow, reducing the overtopping water surface elevation. However, the flow capacity of this area would be limited and any reduction to the water surface elevation is conservatively neglected.

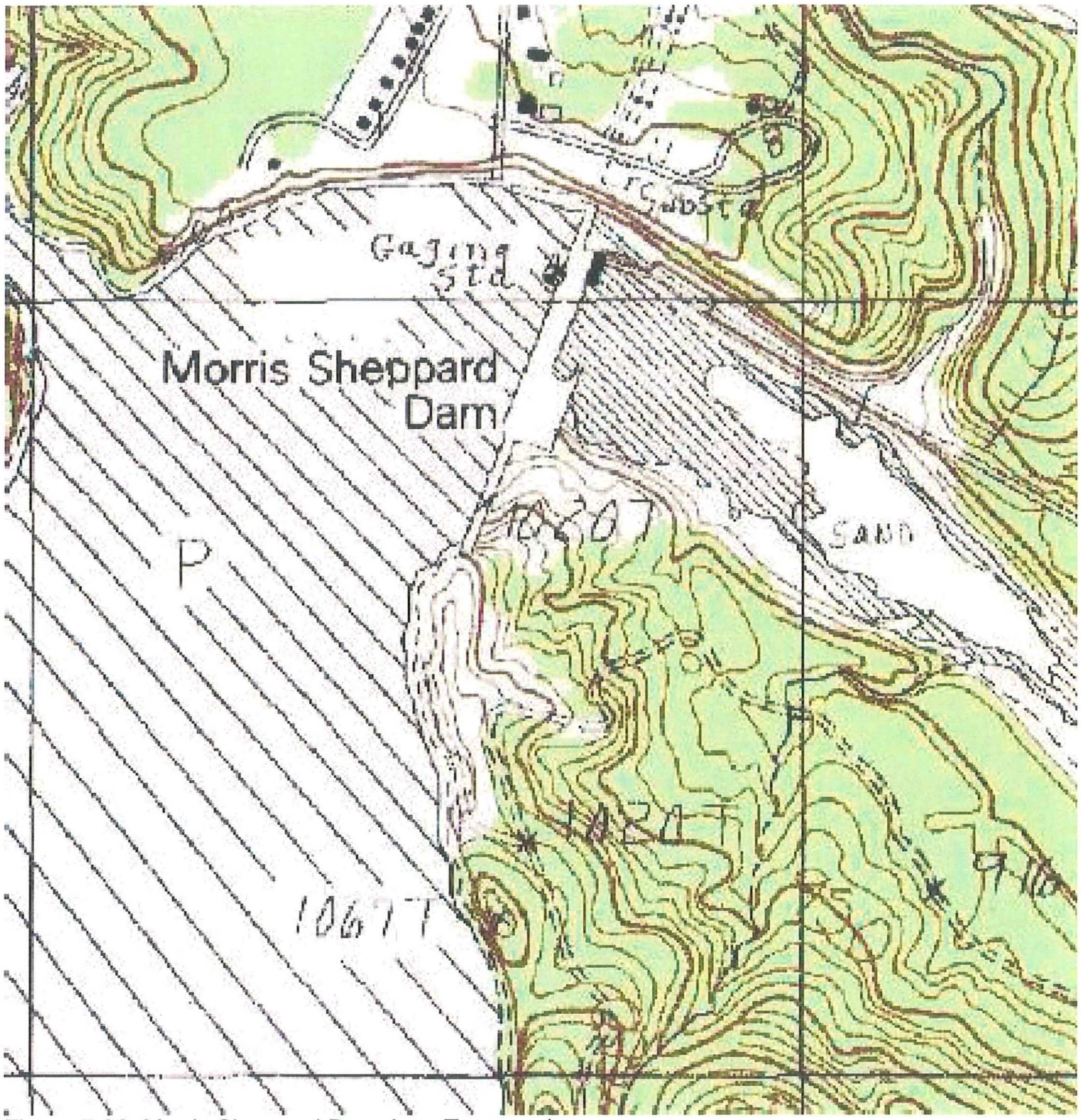

Figure 7-36. Morris Sheppard Dam Area Topography

Tailwater is determined for both the transposed breach flow with added PMF effects of 5,420,000 cfs and the transposed breach wave height with added PMF effects corresponding to an overtopping flow of 5,120,000 cfs. The 7.5 minute USGS quadrangle (Reference 32) for Fortune Bend, TX is inserted into AutoCAD (Reference 2) to determine channel distances, slope, and cross section elevations. Figure 7-37 identifies the selected cross section in relationship to the dam and the channel distances used to determine the slope and elevations.

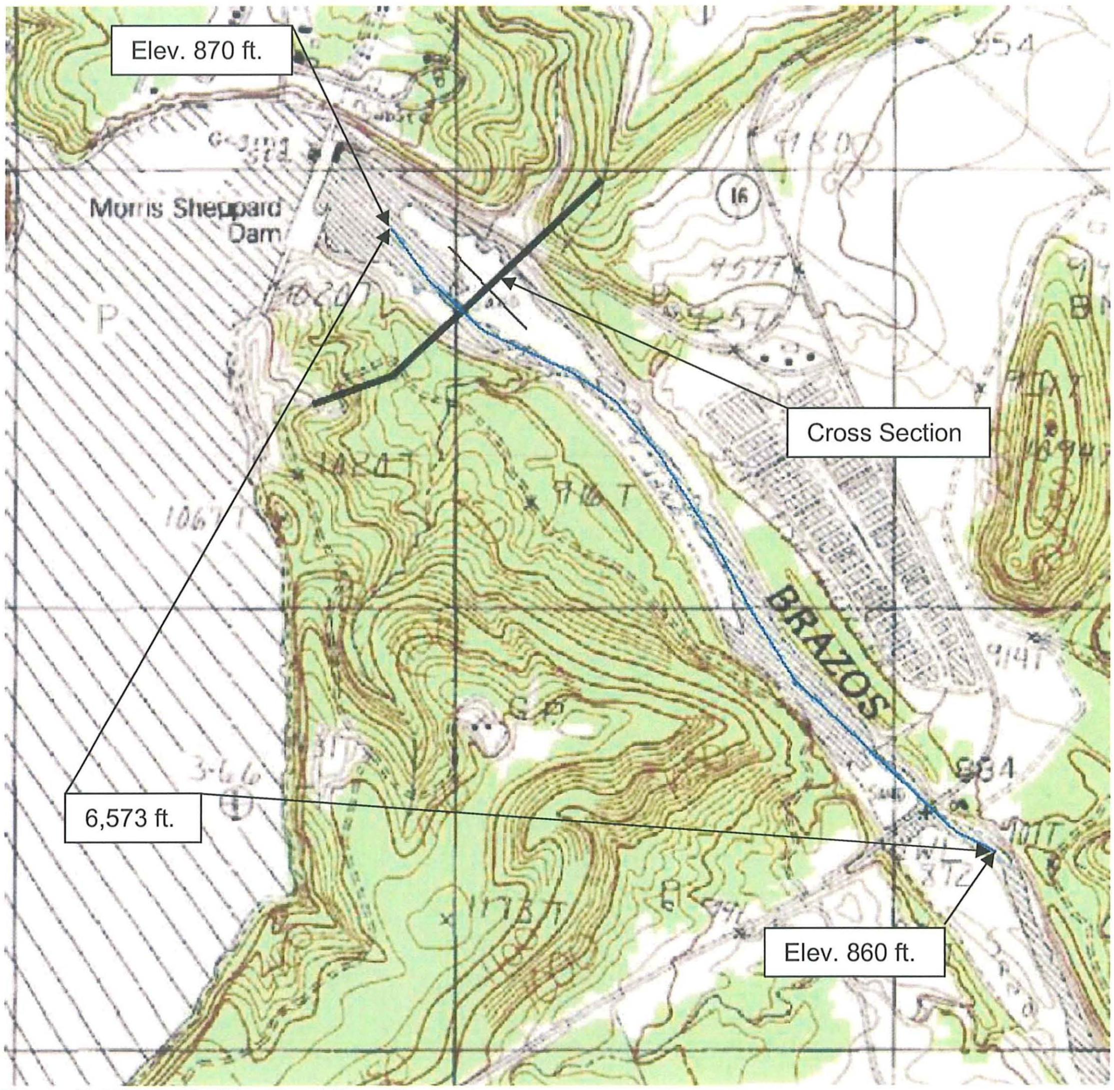

Figure 7-37. Morris Sheppard Dam Downstream

As shown in Figure 7-37, the channel drops 10 ft. over a distance of 6,573 ft. Therefore, the channel slope is 10 ft. / 6,573 ft. = 0.00152 ft./ft., rounded up to 0.0016 ft./ft. Because of the sand bar downstream of the dam, the cross section bottom is set to elevation 870 ft. The cross section station and elevations are provided in Table 7-14.

Table 7-14. Morris Sheppard Dam Section Coordinates

Station (ft.)	Elevation (ft.)	Station (ft.)	Elevation (ft.)
-1,175	1,050	495	890
-744	1,000	547	900
-651	950	656	920
-603	940	1,068	930
-562	930	1,164	940
-518	920	1,215	950
-405	910	1,277	960
-385	900	1,332	970
-333	880	1,425	980
-266	870	1,505	990
310	870	1,589	1,000
440	880		

Stationing from left to right when looking downstream

Tailwater depth is determined using FlowMaster (Reference 3) and the Manning friction formula. From above the two flows of 5,420,000 cfs and 5,120,000 cfs were examined with a slope of 0.0016 ft./ft. As previously discussed in Section 6.0, the Manning coefficient of 0.025 is applied to the channel and overbank areas.

The 5,420,000 cfs flow depth for the cross section is determined to be 103.01 ft. and is rounded down to 103.0 ft. The 5,120,000 cfs flow depth for the cross section is determined to be 100.34 ft. and is rounded down to 100.3 ft. Rounding down is conservative because it results in a lower tailwater elevation as discussed above. The FlowMaster results are provided in Appendix F. Therefore, the tailwater elevation at the downstream cross section is 870 ft. + 103.0 ft. = 973.0 ft. for a flow of 5,420,000 cfs and 870 ft. + 100.3 ft. = 970.3 ft. for a flow of 5,120,000 cfs. Level pool from the cross section upstream to the dam is assumed. This assumption neglects any increase to the tailwater elevation based on backwater effects. The tailwater elevation in both cases is well below the spillway crest elevation of 1,000.0 ft. Therefore, the overtopping discharge is determined to be independent of tailwater. The cross section and tailwater elevation are shown on Figure 7-38.

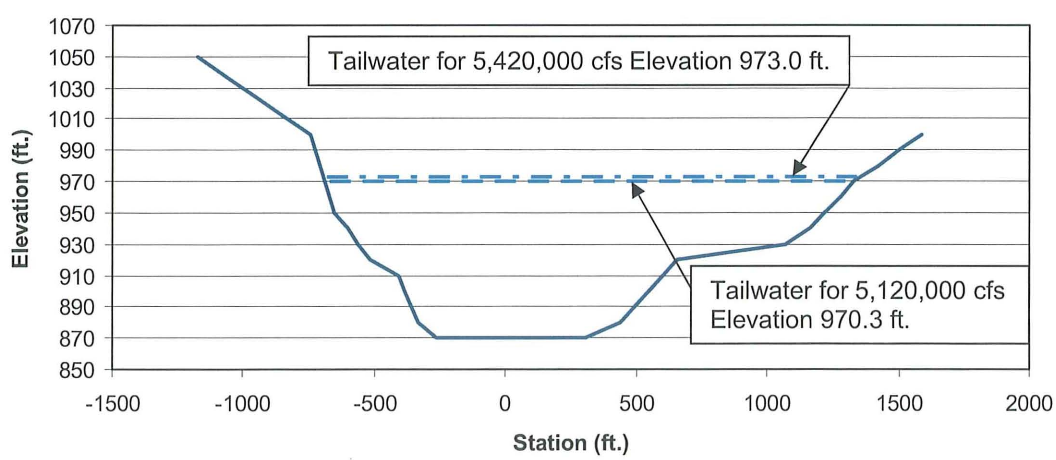

Figure 7-38. Morris Sheppard Dam Tailwater

Of the two scenarios examined, the breach flow resulting in an overtopping flow of 5,420,000 cfs creates the higher headwater elevation. Wind setup for both scenarios is based on the higher headwater elevation which will produce the longer fetch distance.

According to USACE EM 1110-2-1420 (Reference 22) wind setup can be reasonably estimated for lakes and reservoirs using the following equation:

$S = U^2 * F / (1,400 * D)$

USACE EM 1110-2-1420 (Reference 22) indicates that the fetch distance is usually satisfactorily assumed to be two times the effective fetch distance. A straight line fetch is used to define the wind setup and is more conservative than an effective fetch.

As previously discussed, ANSI/ANS-2.8-1992 (Reference 1) is used to define the coincident wind speed. From Figure 7-5, the Annual Extreme-Mile, 30 ft. Above Ground, 2-yr. Mean Recurrence Interval is between 50 mph and 60 mph for the Brazos River watershed upstream from Lake Whitney Lake. The more conservative wind speed of 60 mph is used to generate wind setup.

The controlling overtopping elevation at Morris Sheppard Dam is determined to be 1,075.7 ft. The fetch length is determined from the reservoir surface area at the overtopping elevation. The 7.5 minute USGS quadrangle (Reference 32) for Fortune Bend, TX is inserted into AutoCAD (Reference 2) and because only contours with 10 ft. intervals are identified on the quadrangles the 1,080 ft. contour is used to determine the surface area. As shown on Figure 7-39, the longest straight line fetch distance is determined to be 11,911 ft. (rounded up to 2.3 mi.).

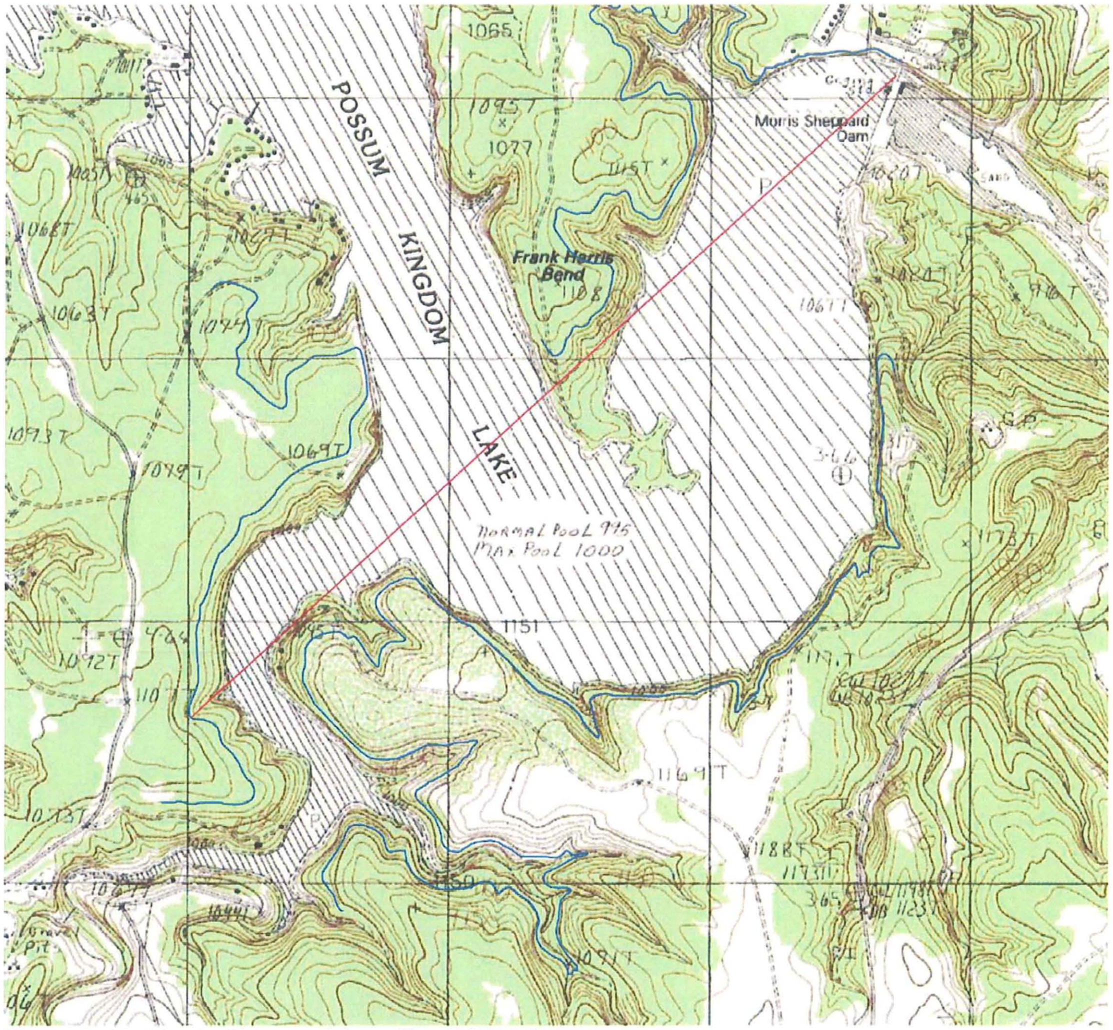

Figure 7-39. Morris Sheppard Dam Fetch Length

A bottom surface profile along the fetch distance is created using the USGS quadrangles (Reference 32). For elevations below the water surface, bathymetry from the Texas Water Development Board volumetric survey (Reference 21) is inserted into AutoCAD (Reference 2), as shown on Figure 7-40. The bottom surface profile is shown on Figure 7-41.