ENERCON	CALCULATION COVER SHEET	CALC. NO. TXUT-001-FSAR-2.4.4-CALC-015
		REV. 0
		PAGE NO. 1 of 36

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 (If YES, identify the superseded calculation.) Superseded Calculation No._____		X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: N/A

Revision Impact on Results: N/A

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

(Print Name and Sign)

Originator: Suraj Balan Anubhav Gaur	Date: 8/25/08
Design Verifier: Randall Kolar Kendra Dresback	Date: 8/25/08
Approver: Mike Laggart	Date: 9/4/08

CALCULATION REVISION STATUS

REVISION	DATE	DESCRIPTION
0	August 25, 2008	Original Calculation

PAGE REVISION STATUS

PAGE NO.	REVISION	PAGE NO.	REVISION
All	0		

APPENDIX REVISION STATUS

APPENDIX NO.	PAGE NO.	REVISION NO.	APPENDIX NO.	PAGE NO.	REVISION NO.
A	All	0			

ENERCON	CALCULATION DESIGN VERIFICATION PLAN AND SUMMARY SHEET	CALC. NO. TXUT-001-FSAR-2.4.4-CALC-015 REV. 0 PAGE NO. 3 of 36

Calculation Design Verification Plan:

Apply CSP Number 3.01 Revision 5 Section 4.5.a. Design Review Method and to include at a minimum that:

- methodology is appropriate and correctly applied
- calculation provides a reasonable estimate of dam failure effects and resulting water surface elevation

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

Approver: Mike Lagport	**Date:** 9/4/08

Calculation Design Verification Summary:

We have reviewed the Brazos River dam failure PMF calculation and have come to the following conclusions:

1. The methodology applied to this set of calculations is appropriate and has been applied correctly;

2. The calculation provides a conservative estimate of the dam failure effects and the resulting water surface elevation;

3. The Originator has followed the recommendations given during the review process;

4. The calculations were independently verified and all errors have been corrected in the final version of the calculation sheets.

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

(Print Name and Sign)

Design Verifier: Randall Kolar Randall Kolar Kendra Dresback Kendra Dresback	**Date:** 8/25/08
Others:	**Date:**

ENERCON	CALCULATION DESIGN VERIFICATION CHECKLIST	CALC. NO. TXUT-001-FSAR-2.4.4-CALC-015 REV. 0 PAGE NO. 4 of 36

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: Randall Kolar Randall Kolar **Kendra Dresback** Kendra Dresback	**Date:** 8/25/08
Others:	**Date:**

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Appendices

Appendix A - FlowMaster Results - 22 Pages

1.0 Purpose And Scope

To determine the peak flow and resulting water surface elevation at the Comanche Peak Nuclear Power Plant (CPNPP), Units 3 and 4 due to the effects of a Probable Maximum Flood (PMF) coincident with assumed hydrologic domino type upstream dam failures.

2.0 Summary Of Results And Conclusions

The peak flow of the PMF coincident with assumed hydrologic domino type dam failure of Hubbard Creek Dam, Morris Sheppard Dam, and De Cordova Bend Dam at the Brazos River – Paluxy River confluence is about 6.7 million cubic feet per second (cfs). The resulting water surface elevation is 754.01 feet (ft.) above mean sea level (msl) at the Brazos River and the Paluxy River confluence cross section. The resulting water surface elevation for the channel slope sensitivity analysis at the Paluxy River and Brazos River confluence cross section was 774.99 ft. msl. Both resultant water surface elevations are less than the CPNPP Units 3 and 4 grade elevation of 822 ft. msl.

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. Bentley Systems, Inc., FlowMaster computer software, Service Pack 3.

3. Brazos River Authority, Website, http://www.brazos.org/, accessed February 2008.

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6. Luminant/Comanche Peak Units 3 & 4 MNES US APWR, Site Plan, Drawing - GAS-05-11-100-002 Rev. B, by Washington Group International, February 27, 2008.

7. Comanche Peak Steam Electric station. Texas Utilities Generation Company (TXU), 2007. "Final Safety Analysis Report (FSAR)" Amended 101. Comanche Peak Steam Electric Station. Glen Rose, Texas (February 1, 2007).

8. Federal Energy Regulatory Commission, "Environmental and Public Use Inspection Report, Morris Sheppard (Possum Kingdom)", August 5, 1999.

9. Fugro-McClelland (Southwest), Inc., "Geotechnical Remedial Work, Morris Sheppard Dam, Palo Pinto County, Texas, Stability Analyses Report," September 24, 1993.

10. Natural Resources Conservation Service, "Technical Release 55: Urban Hydrology for Small Watersheds," United States Department of Agriculture, June 1986.

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

13. U.S. Army Corps of Engineers, "Engineering and Design, River Hydraulics," EM 1110-2-1416, October 15, 1993

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

15. U.S. Army Corps of Engineers, Hydrologic Engineering Center, Hydrologic Modeling System, HEC-HMS Release Notes, Version 2.2.0, August 2002.

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

17. U.S. Geological Survey, Quadrangles, Website, http://www.topozone.com, accessed February 2008.

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

19. 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.

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

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

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

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

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

25. Norman, J.M., R.J. Houghtalen, W.J. Johnston, "Hydraulic Design of Highway Culverts," Federal Highway Administration, FHWA-NHI-01-020 HDS No. 5, Second Edition, May 2005.

4.0 Assumptions

Dam failures are assumed to occur coincident with the peak of the probable maximum flood (PMF). Failure during the PMF exceeds the regulatory guidance for hydrologic events coincident with seismic failure.

PMF estimates were determined by the alternative generalized methods described in Nuclear Regulatory Commission (NRC) Regulatory Guide 1.59 (RG 1.59) (Reference 19). 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 Hubbard Creek Dam coincident with the PMF for its respective watershed. The resultant breach flow was transposed without attenuation to the Morris Sheppard Dam, where it was then combined with the PMF for the Brazos River watershed. The combined dam failure flow and PMF was then transposed without attenuation to De Cordova Bend Dam. Finally, the total resulting flow was transposed to a location on the Brazos River near the site without attenuation.

The approach included the conservative assumptions that the PMF for the Hubbard Creek Dam and Brazos River watersheds occur coincidentally, and that no attenuation of the flood flow was 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 were assumed based on U.S. Army Corps of Engineers (USACE) RD-13 (Reference 14). When a range was provided, parameters were assumed to be at the more conservative end of the range.

As applied in the dam failure equation, given by USACE EM 1110-2-1420 (Reference 12) assumes no tailwater conditions. This is a conservative assumption because it maximizes the head difference for the water surface elevation used in the equation. A greater head difference results in a larger breach flow.

Water surface elevation calculations at Morris Sheppard Dam and De Cordova Bend Dam assumed the spillway gates are closed. This assumption increases the overtopping depth resulting in greater breach flows.

The Manning's roughness coefficient "n" value for the Brazos River is based on published tables (Reference 5). To account for variability and uncertainty of the n value, a sensitivity analysis was performed using an n value increased by 50 percent.

5.0 Design Inputs

- NRC RG 1.59 Appendix B (Reference 19) figures enveloping PMF isolines for drainage area sizes were used to determine PMF.
- The Comanche Peak Nuclear Power Plant Units 3 and 4, plant grade was set at elevation 822 feet (Reference 6).
- USACE National Inventory of Dams (NID) database (Reference 16) information:
  - Hubbard Creek Dam – earthfill; 1107 sq. mi. drainage area; 12,580 ft. long; 109 ft. high; 2000 ft. uncontrolled spillway; spillway capacity 480,387 cfs.
  - Morris Sheppard Dam – buttress; 13,310 sq. mi. drainage area; 2740 ft. long; 154 ft. high; 729 ft. gated spillway; spillway capacity 515,000 cfs.
  - De Cordova Bend Dam – gravity; 15,451 sq. mi. drainage area; 2200 ft. long; 79 ft. high; 656 ft. gated spillway; spillway capacity 635,000 cfs.

The confluence cross section was determined based on U.S. Geological Survey (USGS) quadrangles using 10-ft. contour intervals. The confluence cross section stations and elevation in ft. above msl at the respective stations are shown in Figure 7-17.

6.0 Methodology

The listed design guidelines were 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.
- 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.
- American Nuclear Society, "Determining Design Basis Flooding at Power Reactor Sites," ANSI/ANS-2.8-1992, July 28, 1992.
- U.S. Nuclear Regulatory Commission, "Standard Format and Content of Safety Analysis Reports for Nuclear Power Plants", Regulatory Guide 1.70, November 1978.
- U.S. Nuclear Regulatory Commission, "Flood Protection for Nuclear Power Plants", Regulatory Guide 1.102, September 1976.
- U.S. Nuclear Regulatory Commission, "Combined License Applications for Nuclear Power Plants (LWR Edition)," Regulatory Guide 1.206, June 2007.
- U.S. Nuclear Regulatory Commission, "Early Site Permits; Standard design Certifications; and Combined Licenses for Nuclear Power Plants", 10 CFR Part 52, August 2007.
- U.S. Nuclear Regulatory Commission, "Industry Guidelines for Combined License Applicants under 10 CFR Part 52", NEI 04-05, October 2005.

The potential effects from flooding on the Brazos River were examined based on the PMF coincident with assumed hydrologic domino type dam failure of Hubbard Creek Dam (located on a tributary of the Brazos River), Morris Sheppard Dam, and De Cordova Bend Dam. The PMF for the Hubbard Creek Dam watershed and the Brazos River main river dam watershed was based on the NRC RG 1.59 (Reference 19). RG 1.59 Appendix B methods were used to select the PMF estimates in the dam failures analysis.

Assumed coincident dam failure flows were based on the USACE mathematical expressions provided by EM 1110-2-1420 (Reference 12) and HEC-HMS documentation (Reference 15). Breach characteristics were based on the information included in the USACE RD-13 (Reference 14).

Hubbard Creek Dam is an earthfill structure with an uncontrolled spillway. The PMF for Hubbard Creek Dam was applied to the structure based on information obtained from the USACE NID database (Reference 16). The standard broad crested weir overflow equation (Reference 4) was used to determine the peak water surface elevation. The dam failure flow was determined using the USACE mathematical expressions and was added to the PMF for the Brazos River without attenuation.

The combined dam failure flow added to the PMF for the Brazos River was used to determine the peak water surface elevation at Morris Sheppard Dam. The information from NID (Reference 16)

database and the standard broad crested weir overflow equation were used to determine the peak water surface elevation at the Morris Sheppard Dam. The dam failure flow was determined by the USACE mathematical expression and was then applied to the De Cordova Bend Dam without attenuation. A similar method was used to determine the dam failure flow which was applied to the confluence of the Brazos River and the Paluxy River without attenuation.

The Paluxy River is a tributary of the Brazos River, with a confluence just downstream of the confluence with Squaw Creek. A cross section at the Brazos River – Paluxy River confluence was determined based on USGS quadrangles with 10 feet contour intervals. FlowMaster computer software (Reference 2) in combination with the PMF flow and the effects of upstream dam failure without attenuation was used to determine the water surface elevation at the cross section. FlowMaster (Reference 2) computes open channel flow through the continuity equation, as defined in the USACE EM 1110-2-1416 (Reference 13, Equation 2-4), and Manning's formula, as defined in TR-55 (Reference 10, Equation 3-4). The resulting water surface elevation at the Brazos River and the Paluxy River confluence was compared with the CPNPP, Units 3 & 4 grade elevation of 822 ft. msl.

7.0 Calculations

Introduction

The NRC NUREG-0800 (Reference 20) and Regulatory Guide 1.206 (RG 1.206) (Reference 18) both specifically examine dam failure under hydrologic conditions. If failures are likely and under seismic 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 necessary to examine. Postulated dam failure does not assume that the dams are unsafe or will fail in the manner prescribed.

The critical dam failure event was the assumed domino-type failure of Hubbard Creek Dam, Morris Sheppard Dam and DeCordova Dam coincident with the PMF. The Brazos River watershed, locations for the three dams and CPNPP, Units 3 and 4 are identified in Figure 7-1.

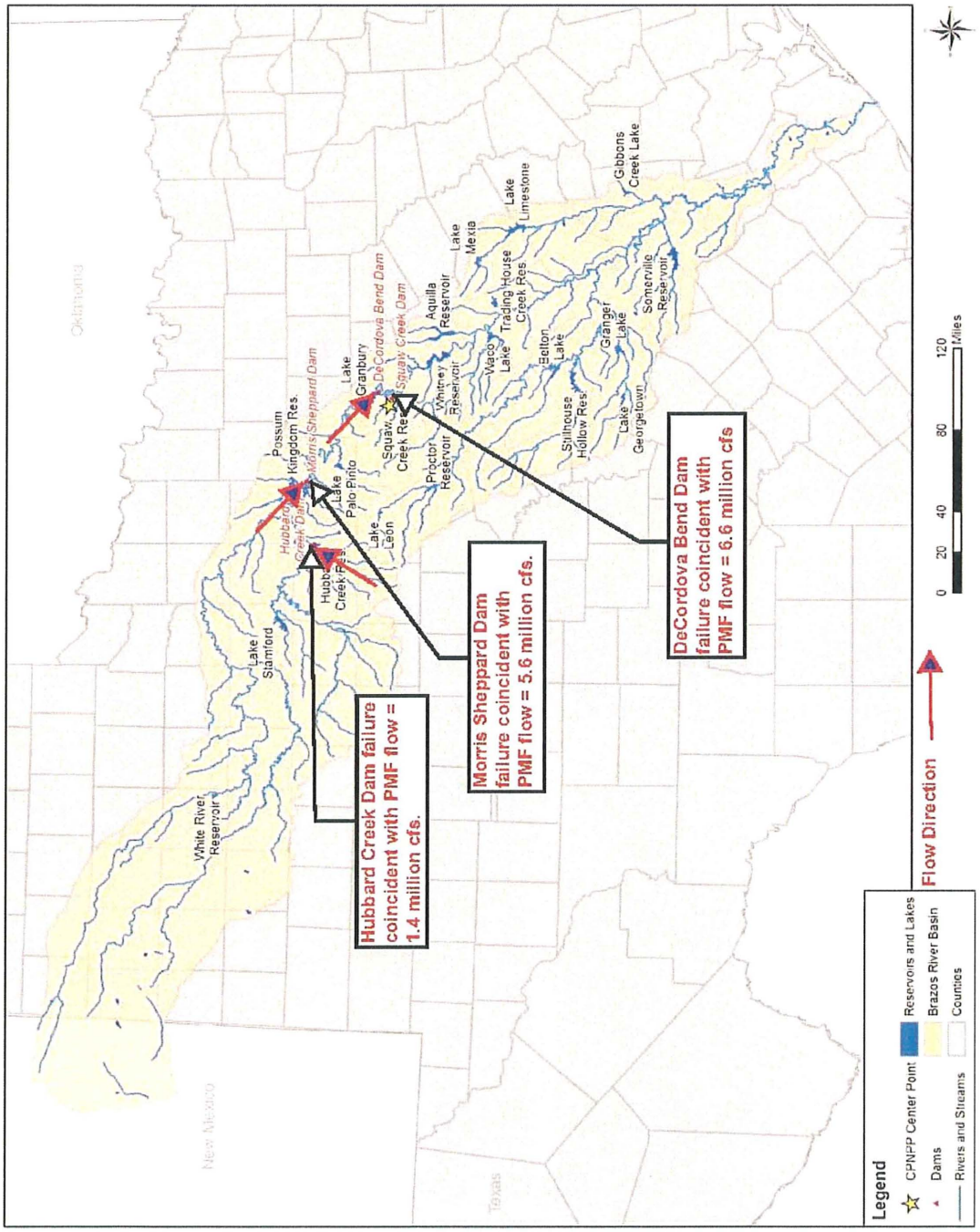


Figure 7-1 Brazos River Watershed

PMF Hubbard Creek Dam Watershed

Hubbard Creek Dam is located at the coordinates shown in Figure 7-2. Coordinates are based on the USGS quadrangles accessed using Topozone (Reference 17).

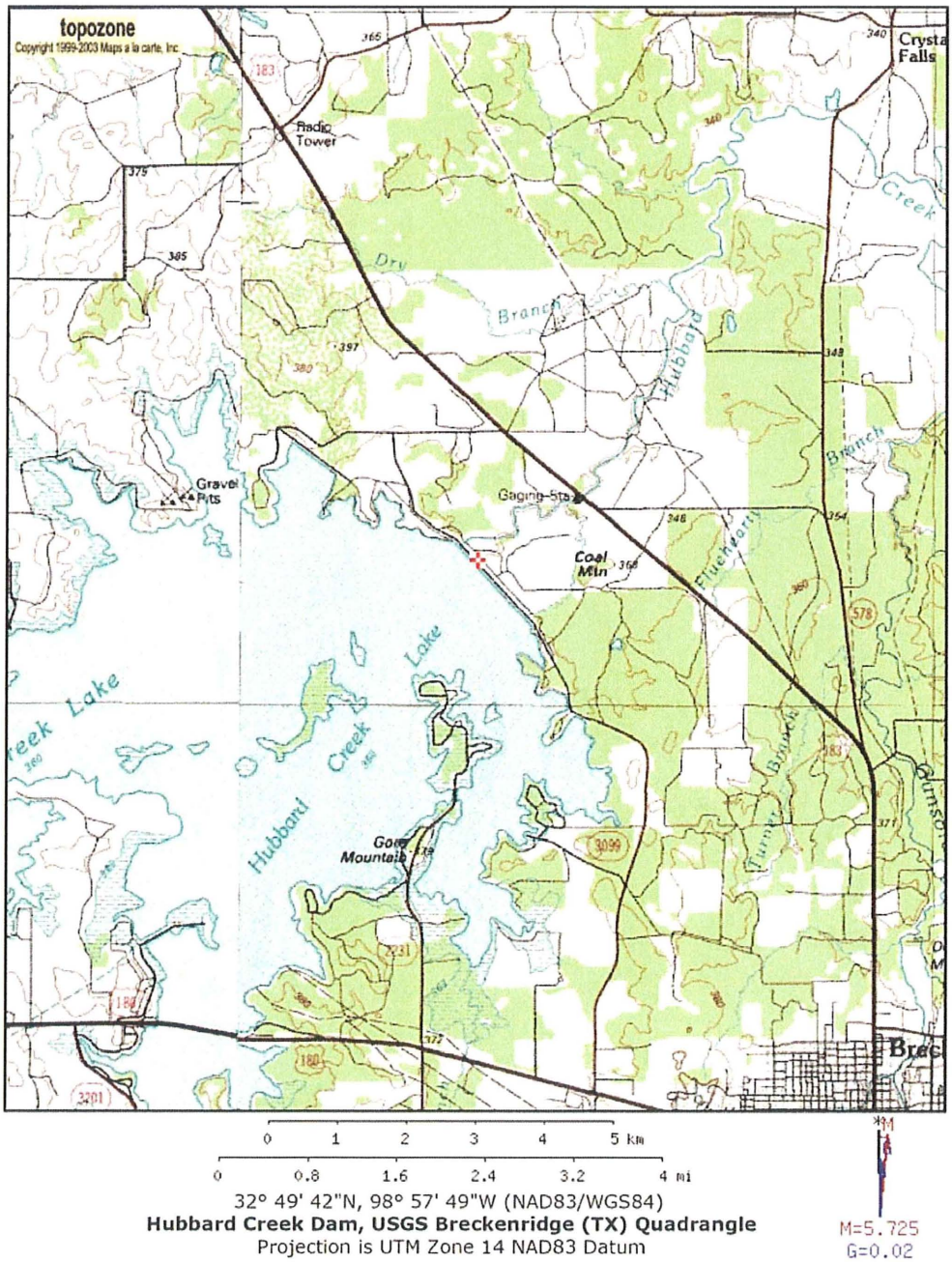


Figure 7-2 Hubbard Creek Dam Location

The NRC RG 1.59 Appendix B (Reference 19) method was 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 7-3 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 was used between isolines.

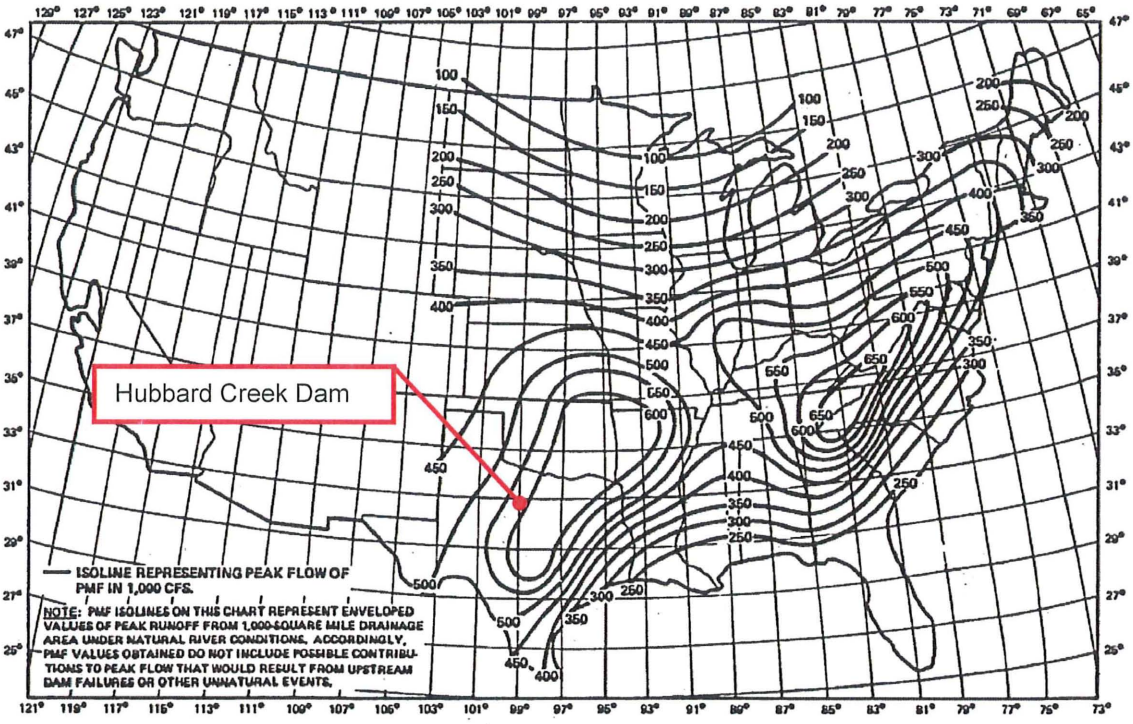


FIGURE B.4 PROBABLE MAXIMUM FLOOD (ENVELOPING PMF ISOLINES) FOR 1,000 SQUARE MILES

Figure 7-3 RG 1.59 1,000 sq. mi. Enveloping PMF Isolines

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 were plotted on a log-log scale and a smooth curve was fitted to the points, as shown in Figure 7-4. The drainage area for Hubbard Creek Dam was then used to determine the PMF for Hubbard Creek Dam.

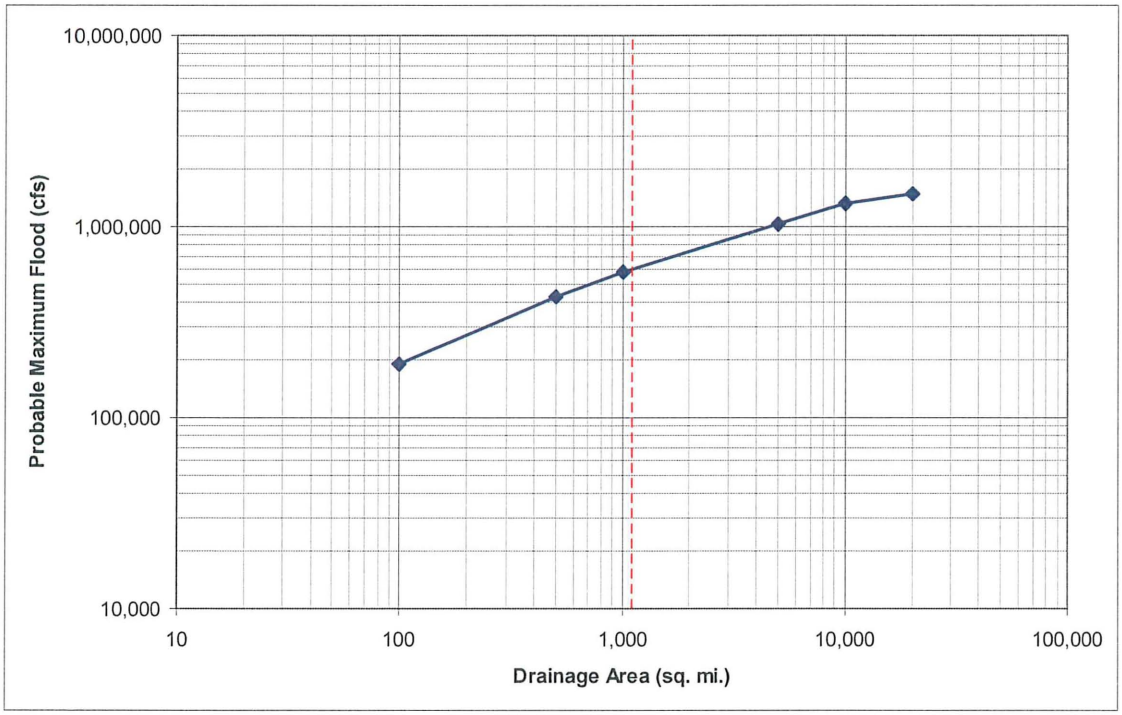


Figure 7-4 PMF Enveloping Isolines Based on Hubbard Creek Dam Location

According to the NID database (Reference 16), the Hubbard Creek Dam drainage area is 1,107 sq. mi. The vertical dashed line in Figure 7-4 is located at the corresponding drainage area value. The resulting PMF flow read from Figure 7-4 is 600,000 cfs.

The PMF was applied to Hubbard Creek Dam to determine the water surface elevation. According to the NID database, Hubbard Creek Dam is an earthfill structure, 109 ft. high, 12,580 ft. long, and has a 2000 ft. long uncontrolled spillway with a capacity of 480,387 cfs.

The PMF exceeds the spillway capacity by: 600,000 cfs – 480,387 cfs = 119,613 cfs. The excess flow spills over the dam crest. Overtopping was modeled through the standard broad crested weir flow equation defined by the HEC-RAS reference manual (Reference 4, 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.)

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. As discussed in Section 4.0, no tailwater was assumed to maximize the breach flow equation result. 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. The lower the weir flow coefficient the higher the head of overtopping will be for a given flow rate. In turn, a higher head of overtopping the greater breach flows will be.

The weir length is the dam crest length, 12,580 ft. The broad crested weir equation was used to solve for H, the overtopping depth was determined to be 2.4 ft.

$H = [Q / (C * L)]^{2/3} = [119{,}613 \text{ cfs} / (2.6 * 12{,}580 \text{ ft.})]^{2/3} = 2.4 \text{ ft.}$

<u>Dam Failure Hubbard Creek Dam</u>
Breach parameters for an earth fill dam were determined through USACE RD-13 (Reference 14, 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 109 ft., and the side slopes of the breach are 1:1 (horizontal:vertical).

Therefore, $W_b = 3 * \text{height of dam} = 3 * 109 \text{ ft.} = 327 \text{ ft.}$

HEC-HMS version 2.2.0 release notes (Reference 15, Page 8) were used for the dam break equation including 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

As discussed in Section 4.0, no tailwater was assumed to maximize the water height component of the equation and subsequent breach flow. The water height is equal to the height of the dam plus the addition of any overtopping flows: h = 109 ft. + 2.4 ft. = 111.4 ft. Therefore, the breach flow is:
$Q = 1.7 * 327 \text{ ft.} * (111.4 \text{ ft.})^{1.5} + 1.35 * 1 * (111.4 \text{ ft.})^{2.5} = 830{,}445 \text{ cfs.}$

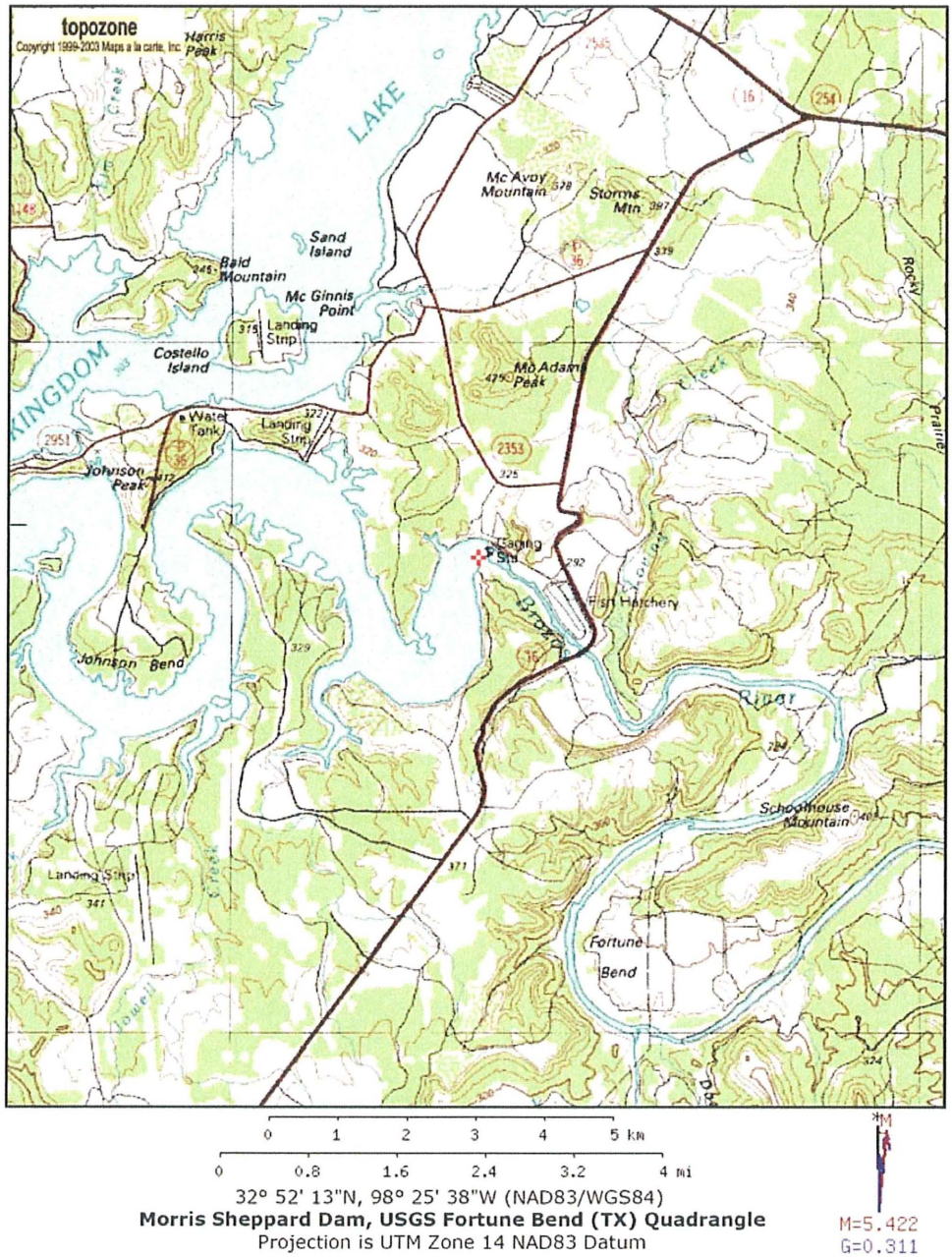
The total flow is the sum of the spillway flow, the breach flow, and the remainder of the overtopping flow not affected by the breach width. The total breach width at the crest is the breach width plus the horizontal component of the side slopes. Therefore, the breach width at the crest is:
$W_b = 327 \text{ ft.} + 109 \text{ ft.} + 109 \text{ ft.} = 545 \text{ ft.}$

The flow not affected by the breach width overtops the remaining crest length: 12,580 ft. – 545 ft. = 12,035 ft. The remaining overtopping flow was determined by multiplying the initial overtopping flow prior to breaching by a ratio of the unaffected crest length divided by the total crest length:
119,613 cfs * 12,035 ft. / 12,580 ft. = 114,431 cfs.

Therefore, the total flow from the Hubbard Creek Dam failure is: Q = 480,387 cfs + 830,445 cfs + 114,431 cfs = 1,425,263 cfs ≈ 1.4 million cfs. This flow was transposed to the Brazos River and combined with the PMF flow on the Brazos River without any attenuation.

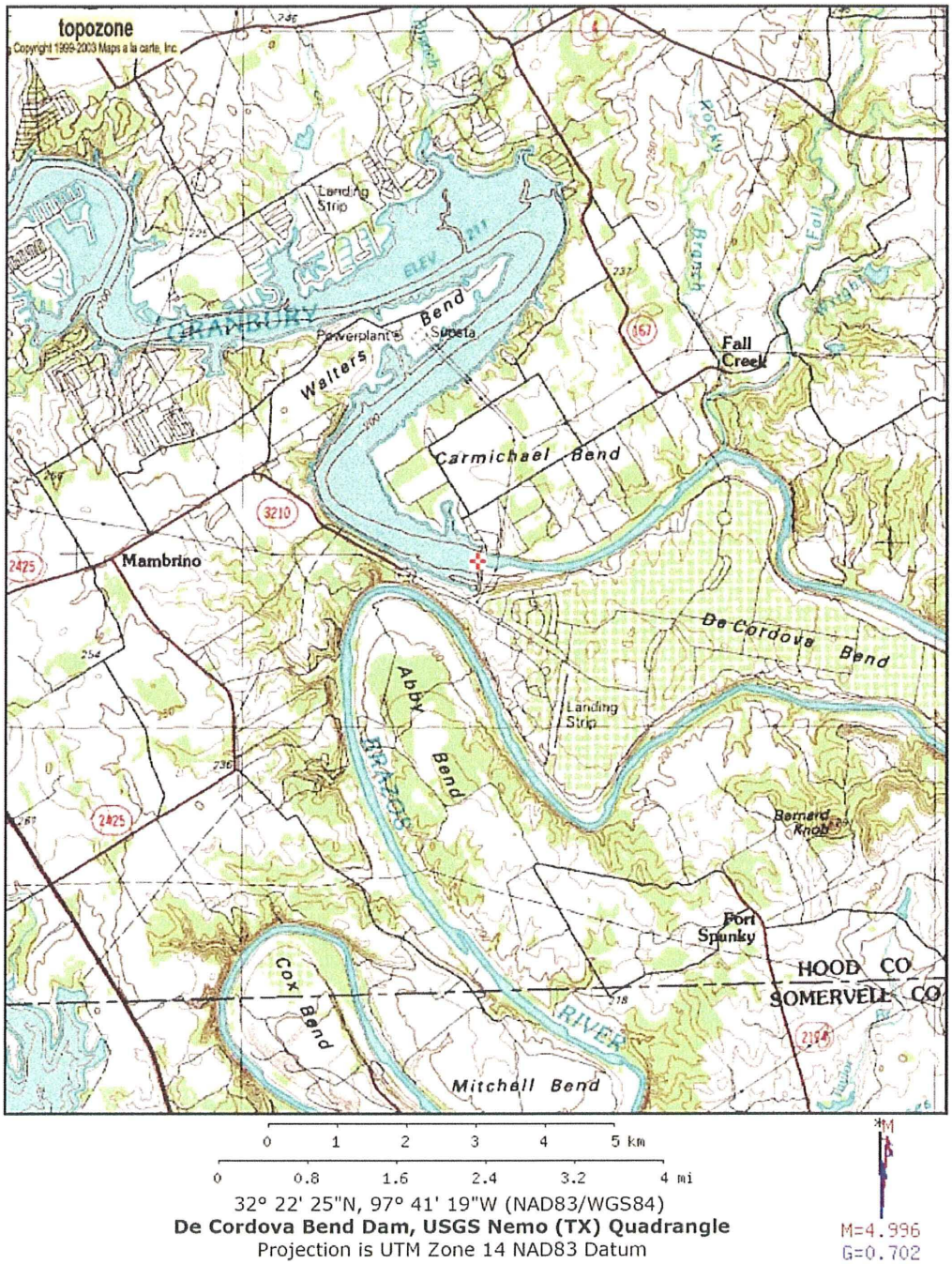
PMF Brazos River

Morris Sheppard Dam is located at the coordinates shown in Figure 7-3, and De Cordova Bend Dam is located at the coordinates shown in Figure 7-4. Coordinates are based on the USGS quadrangles accessed using Topozone (Reference 17).



32° 52' 13"N, 98° 25' 38"W (NAD83/WGS84)
Morris Sheppard Dam, USGS Fortune Bend (TX) Quadrangle
Projection is UTM Zone 14 NAD83 Datum

Figure 7-5 Morris Sheppard Dam Location



32° 22' 25"N, 97° 41' 19"W (NAD83/WGS84)
De Cordova Bend Dam, USGS Nemo (TX) Quadrangle
Projection is UTM Zone 14 NAD83 Datum

Figure 7-6 De Cordova Bend Dam Location

Similar to the process for Hubbard Creek Dam, the NRC RG 1.59 (Reference 19) method was used to determine the PMF on the Brazos River. However, the PMF was 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 7-7 is a typical chart showing the location of both dams. Table 7-2 presents the results for each chart corresponding to the drainage area provided. Straight line interpolation was used between isolines.

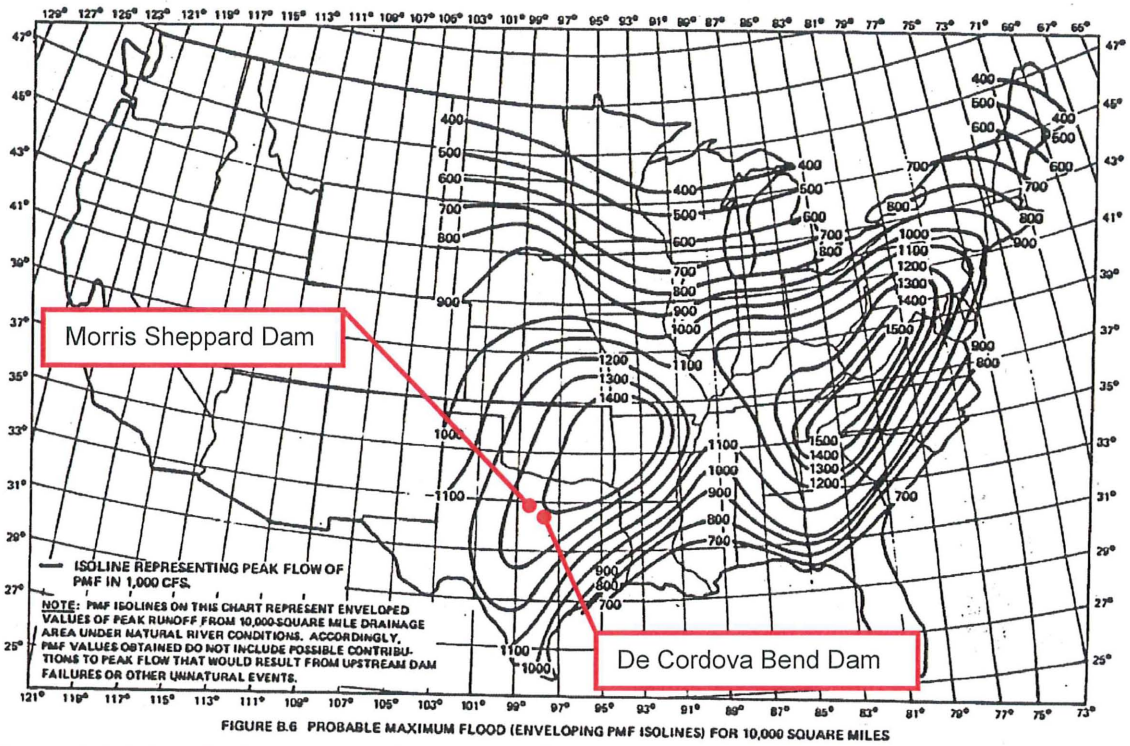


Figure 7-7 RG 1.59 10,000 sq. mi. Enveloping PMF Isolines

Table 7-2 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 7-8.

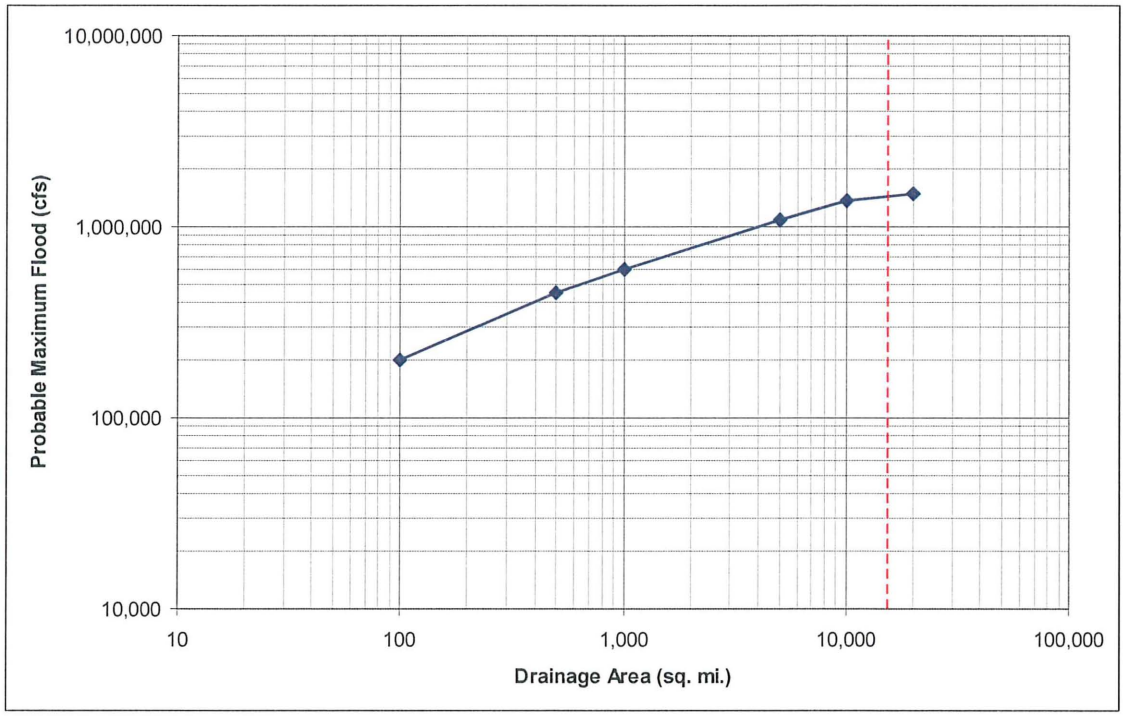


Figure 7-8 PMF Enveloping Isolines Based on Morris Sheppard Dam and De Cordova Bend Dam Location

According to the NID database (Reference 16), the Morris Sheppard Dam drainage area is 13,310 sq. mi. and the De Cordova Bend Dam drainage area is 15,451 sq. mi. Because both areas are close in magnitude, the larger area of De Cordova Bend Dam was used to determine the PMF. The vertical dashed line in Figure 7-8 was located at the corresponding drainage area value. PMF was determined to be 1.45 million cfs.

Dam Failure Morris Sheppard Dam

The PMF for the Brazos River is combined with the breach flow from Hubbard Creek Dam without any attenuation, and applied to Morris Sheppard Dam. Dam breach results were then applied to De Cordova Dam without any attenuation. The total flow applied to Morris Sheppard Dam to determine the water surface elevation was 1,450,000 cfs + 1,425,263 cfs. = 2,875,263 cfs ≈ 2.9 million cfs.

According to the NID database (Reference 16), Morris Sheppard Dam, shown in Figure 7-9, is a concrete buttress structure, 154 ft. high, 2,740 ft. long, and has a 729 ft. long gated spillway with a capacity of 515,000 cfs. It is assumed that the spillway gates are closed and all flow spills over the dam crest and gates. The hydraulic height with spillway gates closed is listed at 130 ft.

According to the Federal Energy Regulatory Commission data (Reference 8), the total length of the concrete buttress section is 1,640 ft. The spillway elevation is 987 ft. with the spillway gates measuring 13 ft. high. This places the normal pool water surface elevation at 1,000 ft. and corresponds to the hydraulic height of 130 ft. The structural height of 154 ft. corresponds to an elevation of 1,024 ft.

According to a geotechnical report for the dam (Reference 9), the remaining crest length (2,740 ft. – 1,640 ft. = 1,110 ft.) is an earth embankment section with a 20 ft. wide crest at elevation 1,024 ft. The embankment section extends beyond the view shown in Figure 7- 9.

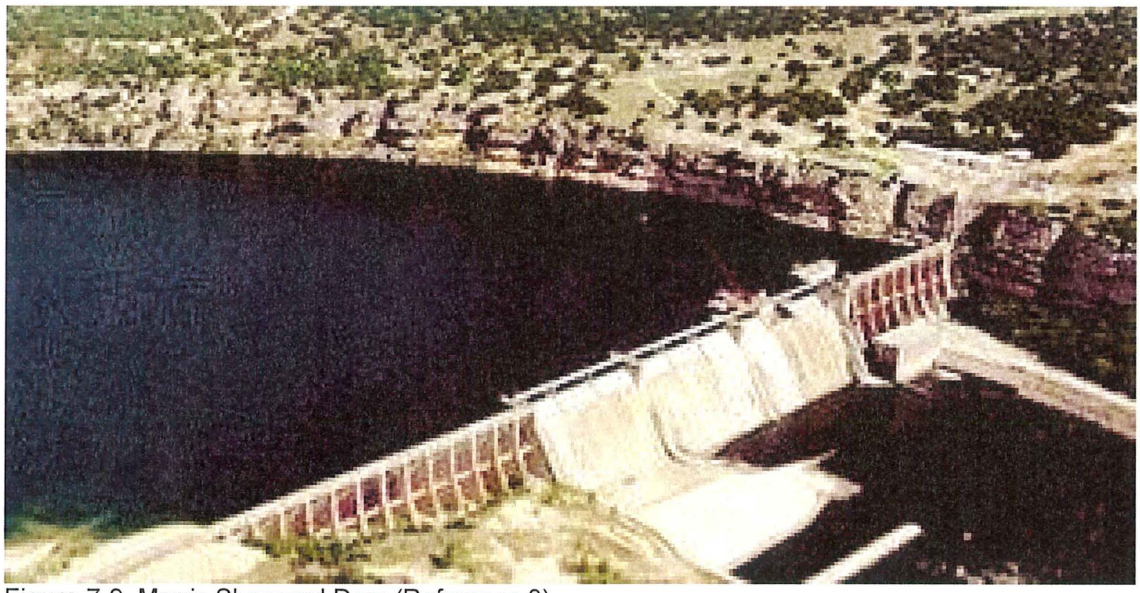


Figure 7-9, Morris Sheppard Dam (Reference 3)

Overtopping was modeled using the standard broad crested weir flow equation defined by the HEC-RAS reference manual (Reference 4, 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.)

As discussed in Section 4.0, no tailwater was assumed in order to maximize the breach flow equation result. As above, this calculation utilized the HEC-RAS reference manual recommended weir flow coefficient of 2.6.

The weir flow equation was modified to include both the spillway crest elevation and the dam crest elevation.

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

where Q = flow (cfs)
C = weir flow coefficient
Ls = length of the spillway crest (ft.)
Es = elevation of the spillway crest, gates closed (ft.)
Lc = total length of the dam crest minus the spillway length (ft.)
Ec = elevation of the dam crest (ft.)
H = overtopping water surface elevation (ft.)

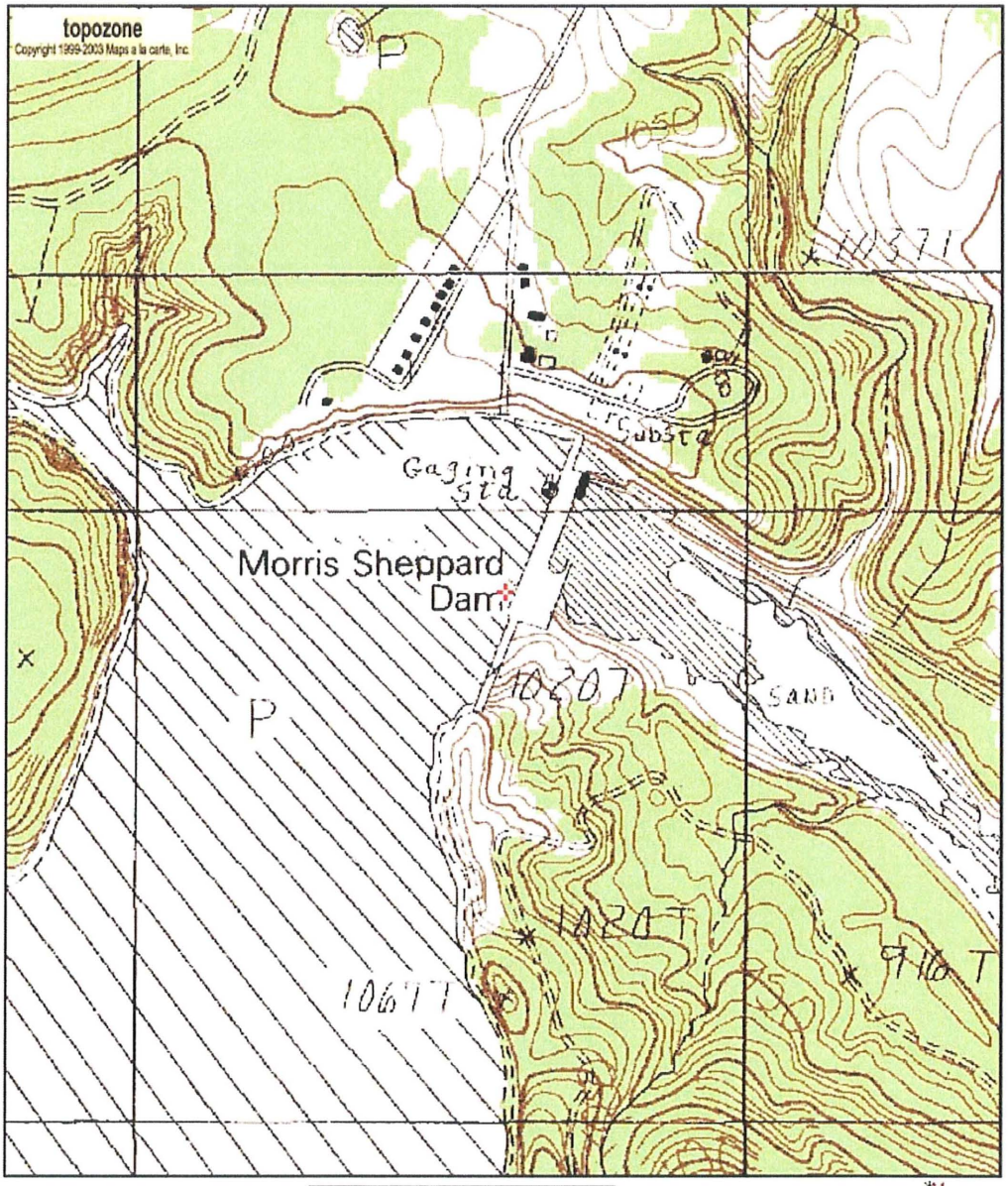
From above Ls = 729 ft.
Es = 1,000 ft.
Lc = 2,740 ft. – 729 ft. = 2,011 ft.; and Ec = 1,024 ft.
The weir flow equation is:

$2{,}875{,}263 \text{ cfs} = Q = 2.6 * 729 \text{ ft.} * (H - 1{,}000 \text{ ft.})^{1.5} + 2.6 * 2{,}011 \text{ ft.} * (H - 1{,}024 \text{ ft.})^{1.5}$

Solving for H iteratively:

H (ft.)	Q (cfs)
1050	1,511,959
1070	2,741,320
1075	3,135,422
1072	2,896,767
1071	2,818,673
1071.7	2,873,261
1071.8	2,881,089
1071.74	2,876,392

Based on iterations, the reservoir maximum elevation is 1,071.7 ft. Referencing USGS topography shown in Figure 7- 10, it is physically possible to achieve the calculated elevation. A near vertical relief occurs at the left abutment up to at least an elevation of 1,100 ft. The right abutment slopes up to 1,067 ft., slopes down to about 1,045 ft. then back up to above 1,100 ft. The calculation considers flow overtops only the buttress, spillway, and embankment structures. The areas at the right abutment below elevation 1,071.7 ft. 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 would be ignored.



UTM 14 553580E 3637065N (NAD83/WGS84)
Morris Sheppard Dam, USGS Fortune Bend (TX) Quadrangle
Projection is UTM Zone 14 NAD83 Datum

Figure 7-10 Morris Sheppard Dam Area Topography

Four failure scenarios are postulated as described below.

1. Overtopping failure of the spillway section
2. Overtopping failure of the embankment section
3. Overtopping failure of the buttress section at the left abutment
4. Overtopping failure of the buttress section between the spillway and embankment sections

The overtopping failures of the buttress sections were eliminated without calculation. In reference to Figure 7- 9, the left abutment buttress section has a shorter crest length than the spillway section. Therefore, failure of the spillway section would result in a greater breach flow. The buttress section between the spillway and embankment sections is approximately the same length as the spillway, but the section depth is about half that of the spillway section. Therefore, failure of the spillway section would result in a greater breach flow.

Figures 7-11, 7-12, and 7-13, show the maximum depth of the embankment section as approximately at elevation 990 ft. According to the geotechnical report (Reference 9), the concrete core wall noted in the figures is 2 ft. wide and extends into the foundation. The core wall was assumed to fail with the embankment section due to overtopping. The LSt and SaSt designations in Figure 7- 13 correspond to limestone and sandstone respectively.

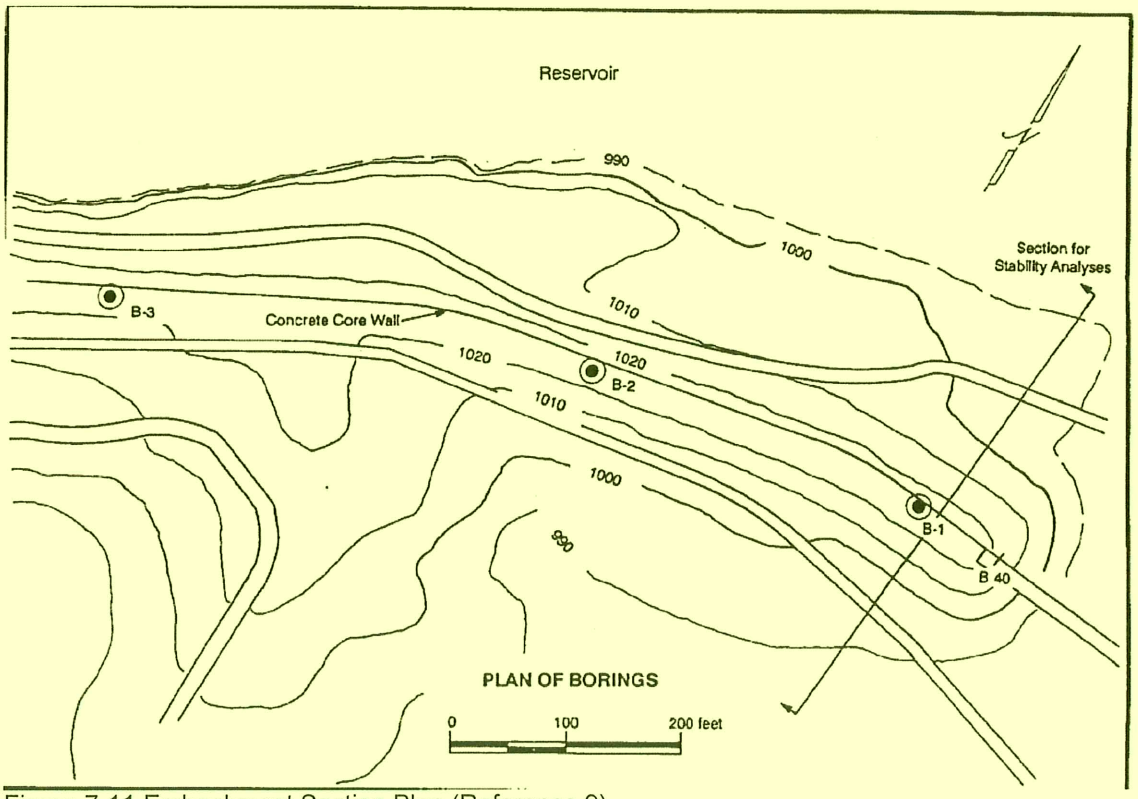
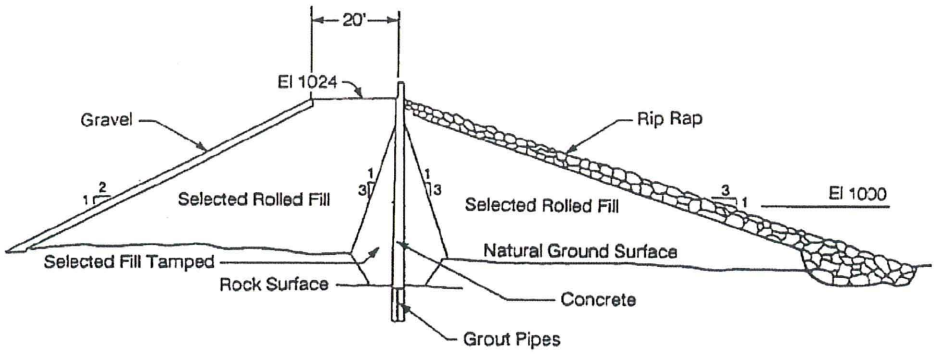


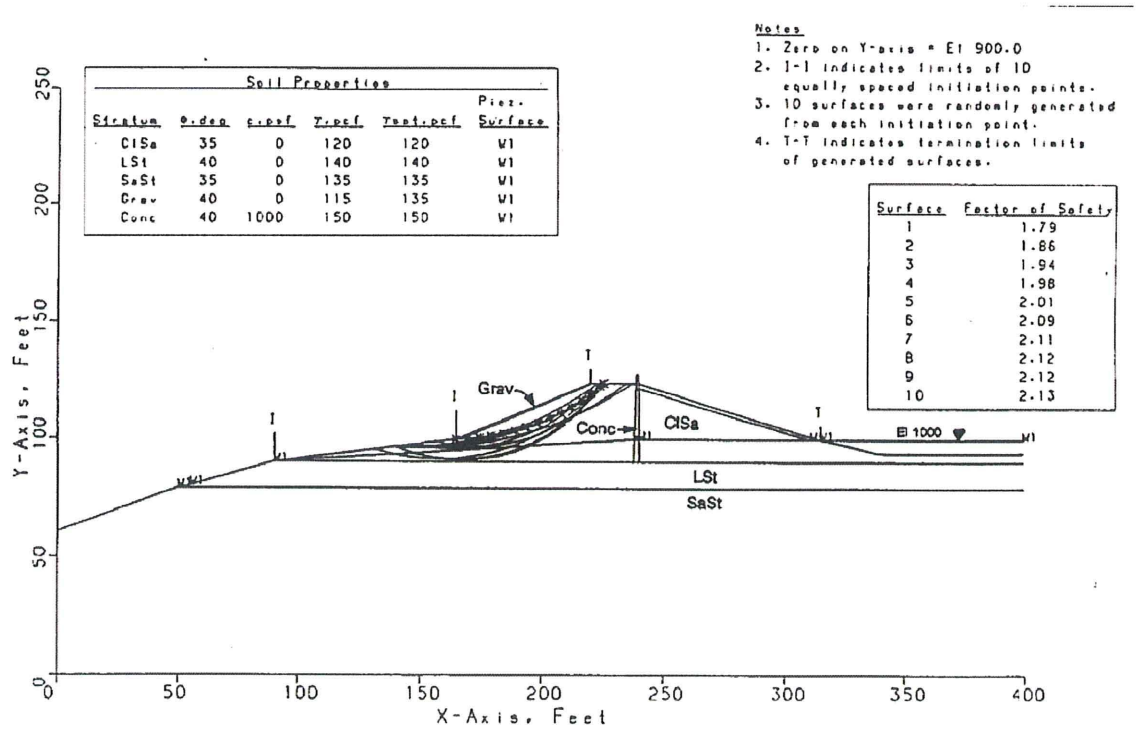
Figure 7-11 Embankment Section Plan (Reference 9)



TYPICAL EMBANKMENT CROSS-SECTION

NOT TO SCALE

Figure 7-12 Embankment Section Typical Cross Section (Reference 9)



SLOPE STABILITY RESULTS
South Embankment
Normal Pool-El 1000

Figure 7-13 Embankment Section Slope Stability Cross Section (Reference 9)

Breach parameters for the embankment section were determined using USACE RD-13 (Reference 14, 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, and the side slopes of the breach are 1:1 (horizontal:vertical). From above embankment section the dam height is 1,024 ft. – 990 ft. = 34 ft.

Therefore, W_b = 3 * height of dam = 3 * 34 ft. = 102 ft.

HEC-HMS version 2.2.0 release notes (Reference 15, Page 8) were used for the dam break equation including 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

As discussed in Section 4.0, no tailwater was assumed to maximize the water height component of the equation and subsequent breach flow. The water height is 1,071.7 ft. – 990 ft. = 81.7 ft. Therefore the breach flow is $Q = 1.7 * 102 \text{ ft.} * (81.7 \text{ ft.})^{1.5} + 1.35 * 1 * (81.7 \text{ ft.})^{2.5} = 209{,}500$ cfs.

By comparison, the spillway section breach flow was determined by the USACE EM-1110-2-1420 (Reference 12, 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.)

Breach parameters for the spillway section were determined by USACE RD-13 (Reference 14, Table 1, Page 17). For concrete gravity dams, the breach width is a multiple of the monolith widths. In this case the entire spillway section is assumed to fail. The breach width is 729 ft. The side slopes are 0:1 (horizontal:vertical).

The depth of water overtopping the spillway section with gates closed is 1,071.7 ft. – 1,000 ft. = 71.7 ft. The total depth of water is the hydraulic height plus the overtopping depth: 130 ft. + 71.7 ft. = 201.7 ft. Therefore $Q = (8 / 27) * 729 \text{ ft.} * (32.2 \text{ ft/sec}^2)^{0.5} * (201.7 \text{ ft.})^{1.5} = 3{,}511{,}078$ cfs ≈ 3.5 million cfs.

By comparison, the assumed failure of the spillway section produces the greatest breach flow. The total flow is the sum of the spillway breach flow and the remainder of the overtopping flow not affected by the breach width. From above the spillway crest length is 729 ft. and the remaining crest length is 2,011 ft. The remaining overtopping flow was determined by multiplying the initial overtopping flow prior to breaching by a ratio of the unaffected crest length divided by the total crest length: 2,875,263 cfs * 2,011 ft. / 2,740 ft. = 2,110,275 cfs ≈ 2.1 million cfs.

Therefore the total flow from the Morris Sheppard Dam failure is: Q = 3,511,078 cfs + 2,110,275 cfs = 5,621,353 cfs ≈ 5.6 million cfs. This flow was transposed downstream to the De Cordova Bend Dam without any attenuation.

Dam Failure De Cordova Bend Dam

The Morris Sheppard Dam failure flow, including the PMF, was applied to De Cordova Bend Dam without any attenuation. From above, the total flow to determine the water surface elevation is 5.6 million cfs.

According to the NID database (Reference 16), De Cordova Bend Dam, shown in Figure 7- 14, is a concrete gravity structure, 79 ft. high, 2,200 ft. long, and has a 656 ft. long gated spillway with a capacity of 635,000 cfs. It was assumed that if the spillway gates are closed then all flow spills over the dam crest and gates.

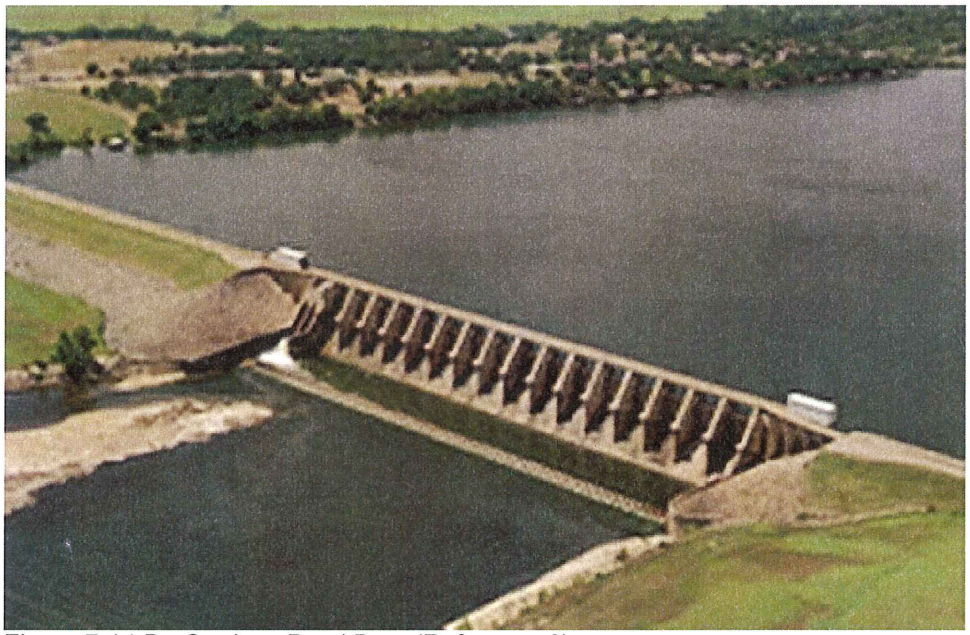


Figure 7-14 De Cordova Bend Dam (Reference 3)

According to the USGS 2006 water year report (Reference 11) for the gauge on Lake Granbury, impounded by De Cordova Bend Dam, the spillway section contains 16-36 ft. wide by 35 ft. high radial gates and 2-7ft. wide by 8 ft. high sluice gates. The top of gates elevation is 693 ft. and the top of dam elevation is 706.5 ft. As shown in Figure 7- 14 the concrete spillway portion of the dam is adjoined by embankment dams. A crest elevation of 706.5 ft. was selected for the embankment dams.

Overtopping was modeled using the standard broad crested weir flow equation defined by the HEC-RAS reference manual (Reference 4, 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.)

As discussed in Section 4.0, no tailwater was assumed to maximize the breach flow equation result. As above, this calculation utilizes the HEC-RAS reference manual recommended weir flow coefficient of 2.6.

The weir flow equation was modified to include both the spillway crest elevation and the dam crest elevation.

$Q_{(spillway+crest)} = C * Ls * (H - Es)^{1.5} + C * Lc * (H - Ec)^{1.5}$

where $Q_{(spillway+crest)}$ = flow (cfs)

C = weir flow coefficient

Ls = length of the spillway crest (ft.)

Es = elevation of the spillway crest, gates closed (ft.)

Lc = total length of the dam crest minus the spillway length (ft.)

Ec = elevation of the dam crest (ft.)

H = overtopping water surface elevation (ft.)

From above the total spillway gate length, Ls, is 16 * 36 ft. + 2 * 7 ft. = 590 ft., at elevation Es = 693 ft. Therefore the remaining crest length, Lc, is 2,200 ft. – 590 ft. = 1,610 ft., at elevation Ec = 706.5 ft. The weir flow equation is:

$5{,}621{,}353 \text{ cfs} = Q = 2.6 * 590 \text{ ft.} * (H - 693 \text{ ft.})^{1.5} + 2.6 * 1{,}610 \text{ ft.} * (H - 706.5 \text{ ft.})^{1.5}$

Solving for H iteratively:

H (ft.)	Q (cfs)
750	1,861,115
800	5,482,428
805	5,910,414
802	5,652,330
801	5,567,162
801.7	5,626,734
801.6	5,618,211
801.64	5,621,620

Based on the above iterations the reservoir's maximum elevation was 801.6 ft. Referencing USGS topography shown in Figure 7- 15, it is not likely for Lake Granbury to overtop at elevation 801.6 ft. When overtopping exceeds the dam crest, flow would also overtop the natural barrier to the south and spread out over the left and right abutments. Flow overtopping to the south would be directed back into the river course downstream of the dam due to the bend in the Brazos River.

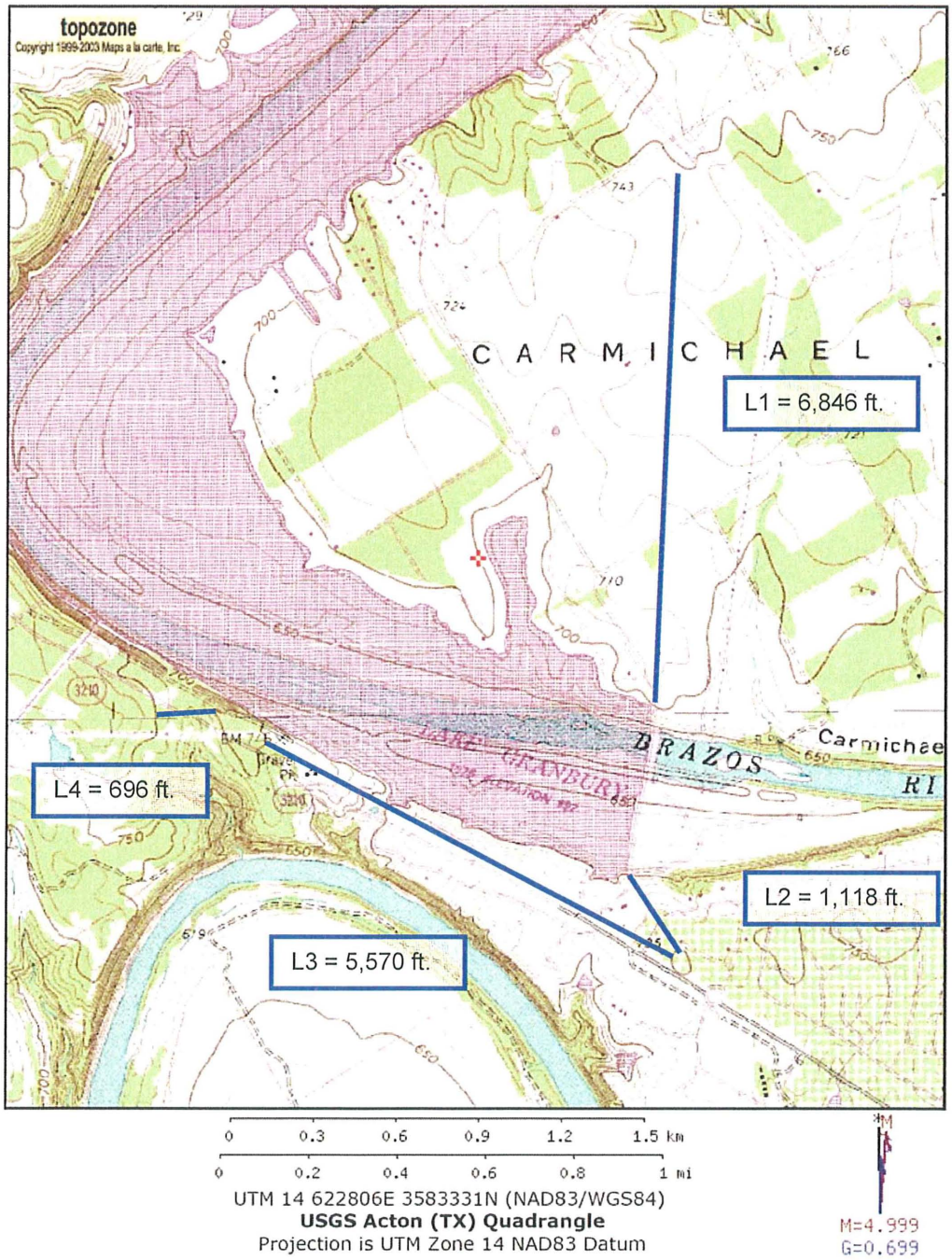


Figure 7-15 De Cordova Bend Dam Area Topography

Each of the four overtopping lengths (L1, L2, L3 and L4) in Figure 7- 15 were estimated through the scale provided. Based on an overtopping water surface elevation of 742.9 ft. it was determined that lengths L1, L2, and L3 have the lowest elevation equal to the dam crest of 706.5 ft. Whereas length L4 has a minimum elevation of 730 ft. The method of segments, as identified in Federal Highway Administration HDS-5 (Reference 25), was used in conjunction with the broad crested weir flow equation to determine the overtopping depth for these sections. The cross sections for the four overtopping lengths are shown in Figure 7-16. Using the method of segments, the weir flow equation was approximated for each length by using the average depth at each cross section. Combined with the flow over the dam the complete weir flow equation becomes:

$$\text{Total } Q = Q_{(\text{spillway+crest})} + Q_{\text{over topping L1}} + Q_{\text{over topping L2}} + Q_{\text{over topping L3}} + Q_{\text{over topping L4}}$$

where:

$$Q_{(\text{spillway+crest})} = C * Ls * (H - Es)^{1.5} + C * Lc * (H - Ec)^{1.5}$$

$$Q_{\text{over topping L1}} = C * ((L_{1a} * (H - 740)/2)^{1.5}) + (L_{1b} * (H - (740+710)/2))^{1.5}) + (L_{1c} * (H - (710+706.5)/2)^{1.5}))$$

$$Q_{\text{over topping L2}} = C * ((L_{2a} * (H - 730)/2)^{1.5}) + (L_{2b} * (H - (730+710)/2))^{1.5}) + (L_{2c} * (H - (710+706.5)/2)^{1.5}))$$

$$Q_{\text{over topping L3}} = C * ((L_{3a} * (H - 730)/2)^{1.5}) + (L_{3b} * (H - (730+710)/2))^{1.5}) + (L_{3c} * (H - (710+706.5)/2)^{1.5}))$$

$$Q_{\text{over topping L4}} = C * ((L_{4a} * (H - 740)/2)^{1.5}) + (L_{4b} * (H - (740+730)/2)^{1.5}) + (L_{4c} * (H - 740)/2)^{1.5}))$$

Note: The equation is not valid for water surface elevations below the dam crest elevation.

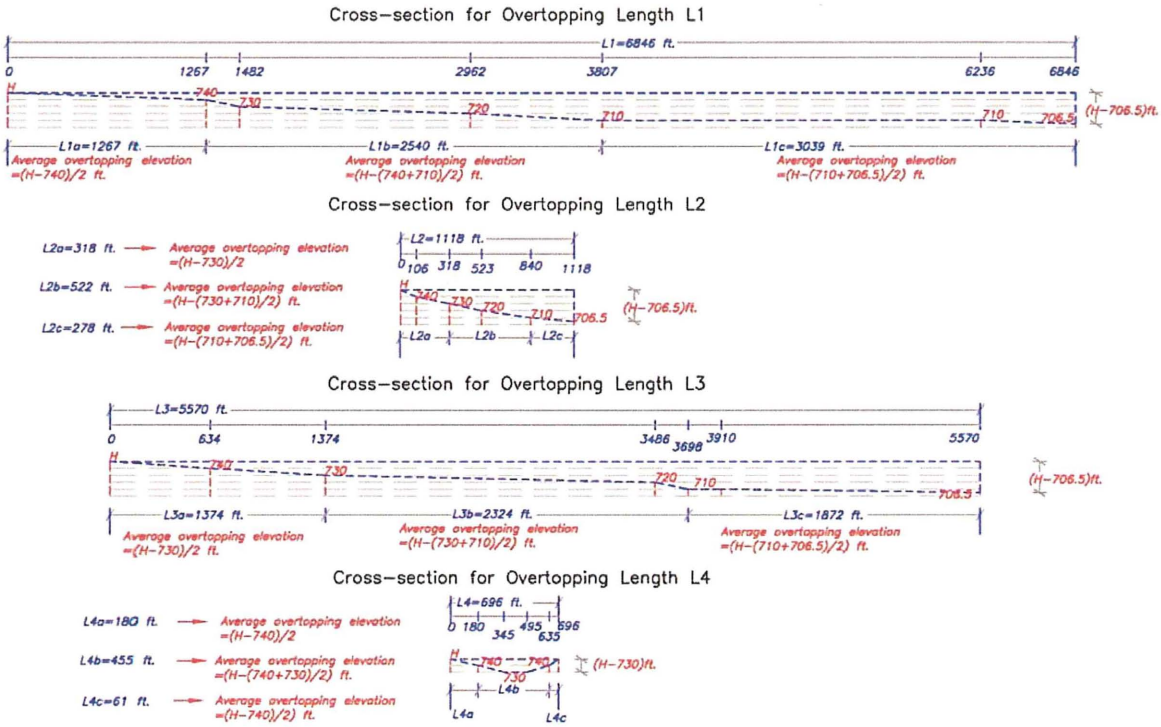


Figure 7-16 Cross-sections for Overtopping Lengths at DeCordova Bend Dam

Substituting the appropriate values, the weir flow equation is:

$$5{,}621{,}353 \text{ cfs} = Q = 2.6 * 590 \text{ ft.} * (H - 693 \text{ ft.})^{1.5} + 2.6 * 1{,}610 \text{ ft.} * (H - 706.5 \text{ ft.})^{1.5}$$
$$C * ((L_{1a} * (H - 740)/2)^{1.5}) + (L_{1b} * (H - (740+710)/2))^{1.5}) + (L_{1c} * (H - (710+706.5)/2)^{1.5}))+$$
$$C * ((L_{2a} * (H - 730)/2)^{1.5}) + (L_{2b} * (H - (730+710)/2))^{1.5}) + (L_{2c} * (H - (710+706.5)/2)^{1.5}))+$$
$$C * ((L_{3a} * (H - 730)/2)^{1.5}) + (L_{3b} * (H - (730+710)/2))^{1.5}) + (L_{3c} * (H - (710+706.5)/2)^{1.5}))$$
$$C * ((L_{4a} * (H - 740)/2)^{1.5}) + (L_{4b} * (H - (740+730)/2)^{1.5}) + (L_{4c} * (H - 740)/2)^{1.5}))$$

$$5{,}621{,}353 \text{ cfs} = Q = 2.6 * 590 \text{ ft.} * (H - 693 \text{ ft.})^{1.5} + 2.6 * 1{,}610 \text{ ft.} * (H - 706.5 \text{ ft.})^{1.5}$$
$$C * ((1267 * (H - 740)/2)^{1.5}) + (2540 * (H - (740+710)/2))^{1.5}) +$$
$$(3039 * (H - (710+706.5)/2)^{1.5})) +$$
$$C * ((318 * (H - 730)/2)^{1.5}) + (522 * (H - (730+710)/2))^{1.5}) +$$
$$(278 * (H - (710+706.5)/2)^{1.5})) +$$
$$C * ((1374 * (H - 730)/2)^{1.5}) + (2324 * (H - (730+710)/2))^{1.5}) +$$
$$(1872 * (H - (710+706.5)/2)^{1.5}))$$
$$C * ((180 * (H - 740)/2)^{1.5}) + (455 * (H - (740+730)/2)^{1.5}) +$$
$$(61 * (H - 740)/2)^{1.5}))$$

Solving for H iteratively:

H (ft.)	Q (cfs)
742.0	5,373,759
743.0	5,656,530
742.8	5,599,491
742.9	5,627,981

There were two postulated failure scenarios, failure of one of the embankment sections, or failure of the spillway section.

Breach parameters for the embankment section were determined using USACE RD-13 (Reference 14, 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, and the side slopes of the breach are 1:1 (horizontal:vertical). From above the dam height is 79 ft.

Therefore, W_b = 3 * height of dam = 3 * 79 ft. = 237 ft.

HEC-HMS version 2.2.0 release notes (Reference 15, Page 8) were used for the dam break equation including 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

As discussed in Section 4.0, no tailwater was assumed to maximize the water height component of the equation and subsequent breach flow. The water height is equal to the height of the dam plus the addition of any overtopping flows: h = 79 ft. + 742.9 ft. – 706.5 ft. = 115.4 ft. Therefore, the breach flow is $Q = 1.7 * 237 \text{ ft.} * (115.4 \text{ ft.})^{1.5} + 1.35 * 1 * (115.4 \text{ ft.})^{2.5} = 692{,}595$ cfs.

By comparison, the spillway section breach flow was determined using the USACE EM-1110-2-1420 (Reference 12, 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.)

Breach parameters for the spillway section were determined using USACE RD-13 (Reference 14, Table 1, Page 17). For concrete gravity dams, the breach width is a multiple of the monolith widths. In this case the entire spillway section is assumed to fail at the breach width of 656 ft. The side slopes are 0:1 (horizontal:vertical).

The depth of water overtopping the spillway section with gates closed is 742.9 ft. – 693 ft. = 49.9 ft. The total depth of water is the dam height plus the overtopping depth: 79 ft. + 49.9 ft. = 128.9 ft. Therefore $Q = (8 / 27) * 656 \text{ ft.} * (32.2 \text{ ft/sec}^2)^{0.5} * (128.9 \text{ ft.})^{1.5} = 1{,}614{,}126 \approx 1.6$ million cfs.

By comparison, the assumed failure of the spillway section produces the greatest breach flow. The total flow is the sum of the spillway breach flow and the remainder of the overtopping flow not affected by the breach width. The spillway crest length is 656 ft and only 590 ft. of the length contains the gates.

The remaining overtopping flow was determined by subtracting the flow over the spillway section from the total overtopping flow of 5,621,353 cfs. The flow over the spillway is a combination of flow over the gates and the piers separating the gates. The total length of the spillway portion containing the piers is 656 ft. – 590 ft. = 66 ft. The total flow over the spillway section at maximum water surface elevation:

$$Q = 2.6 * 590 \text{ ft.} * (742.9 \text{ ft.} - 693 \text{ ft.})^{1.5} + 2.6 * 66 \text{ ft.} * (742.9 \text{ ft.} - 706.5 \text{ ft.})^{1.5} = 578{,}410 \text{ cfs.}$$

The remaining overtopping flow is 5,621,353 cfs – 578,410 cfs = 5,042,943 cfs ≈ 5.04 million cfs.

Therefore, the total flow from the De Cordova Bend Dam failure is: Q = 1,614,126 cfs + 5,042,943 cfs = 6,657,069 cfs ≈ 6.7 million cfs. This flow was transposed downstream without any attenuation to the confluence with the Paluxy River.

Brazos River – Paluxy River Confluence

The flow resulting from upstream dam failures, coincident with the PMF, is applied to the confluence of the Brazos River and the Paluxy River without any attenuation. From above, the total flow to determine the water surface elevation is 6,657,069 cfs. The confluence cross section was derived using USGS quadrangles as shown in Figure 7- 17. Table 3 contains the cross section station data.

Table 7-3 Confluence Cross Section Stations

Station (ft.)	Elevation (ft.)
-8976	800
-7254	750
-4605	700
-3449	650
- 632	600
- 185	560
185	560
866	600
7790	650
8218	700
10952	700
13038	750
15188	750
16172	800

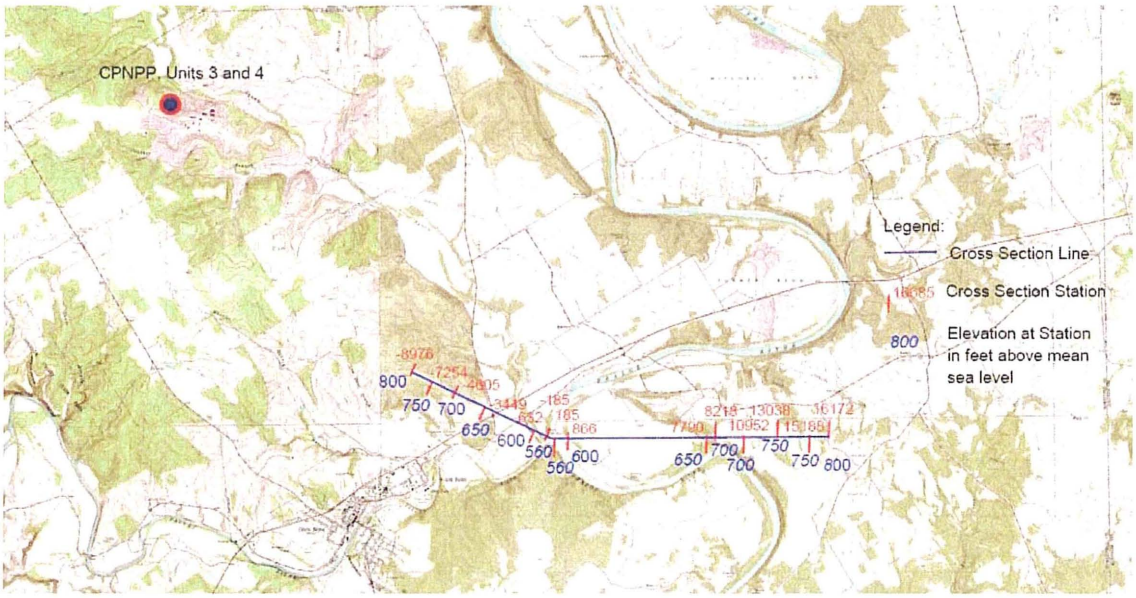


Figure 7-17 Brazos River - Paluxy River Confluence Cross Section

Cross section capacity was evaluated with the combined principles of the continuity equation (Reference 13) and Manning's formula (Reference 10).

$Q = A * 1.486 * R^{2/3} * S^{1/2} / n$

Where Q = flow (cfs)

A = area ($ft.^2$)

R = hydraulic radius = A/P where P = wetted perimeter (ft.)

S = energy slope (ft./ft.)

n = roughness coefficient

From Appendix A, FlowMaster results for the full capacity cross section at elevation 800 ft., A = 3,043,310 ft^2 and P = 25,159 ft., therefore R = 3,043,310 ft^2 / 25,159 ft.= 120.96 ft. The USGS water surface slope was used to approximate the energy slope: (570 ft. – 560 ft.) / 31,680 ft. = 0.000316 ft/ft.

The resulting normal depth calculated using FlowMaster software was 194.01 ft. for the flow of 6.7 million cfs. The resulting water surface elevation at the confluence cross section was 754.01 ft. The resulting water surface elevation is less than the CPNPP Units 3 and 4 plant grade elevation of 822 ft. msl. FlowMaster results are included in Appendix A.

Sensitivity Analysis

As discussed above the flow rate at the confluence of the Paluxy River and the Brazos River is 6.7 million cfs. This large quantity of flow will overtop the channel and floodplains and spread out depending on the topography. Therefore, a sensitivity analysis was conducted for following parameters:

- Manning's Roughness Coefficient
- Cross-section Geometry
- Channel (Brazos River) slope

Manning's Roughness Coefficient Sensitivity Analysis

A sensitivity analysis was performed to account for the land cover variability by increasing the Manning's roughness coefficient by 50 percent. Sensitivity analysis was not performed for a lower Manning's coefficient, because lowering the Manning's roughness coefficient would result in a higher velocity of flow and a lower water surface elevation.

A Manning's roughness coefficient of 0.10 corresponds to major streams with irregular and rough section (Reference 5, Table 5-6, Page 113). The Manning's roughness coefficient sensitivity analysis was performed by increasing the runoff coefficient by 50 percent to 0.15. A Manning's roughness coefficient of 0.15 corresponds to natural streams with very weedy reaches, weedy deep pools, floodways with heavy stand of timber and underbrush. (Reference 5, Table 5-6, Page 112).

The confluence cross section capacity for elevation at 800 ft. msl. and "n" values of 0.10 and 0.15 were computed using FlowMaster. Appendix A, FlowMaster results indicate the confluence cross section capacity at elevation 800 ft for the respective n values as shown below.

n = 0.10
Q = 19,661,842.11 cfs ≈ 20 million cfs.

n = 0.15
Q = 13,107,894.74 cfs ≈ 13 million cfs.

Channel Geometry Sensitivity Analysis

A sensitivity analysis was performed by analyzing the Brazos River channel cross sections upstream and downstream of the Brazos River and the Paluxy River confluence. The cross sections considered for the channel geometry sensitivity analysis extend on either side of the Brazos River up to an elevation of 800 ft. msl. The locations of the upstream and downstream cross sections that were considered for the channel geometry sensitivity analysis are shown in Figure 7-18. The CPNPP site is approximately 4.6 miles from the Brazos River and the Paluxy River confluence. Also, CPNPP site is located on the west side of the Brazos River. It is evident from USGS contour maps that the general topography for the Brazos River reach on the downstream side of the DeCordova Dam slopes towards the southeast side. This general slope towards the southeast side can cause the PMF coincident with dam failure to flow away from the CPNPP site. The resulting water surface elevation at the upstream cross section was 729.89 ft. msl. The resulting water surface elevation at the downstream cross section was 713.44 ft. msl. The resulting water surface elevations are less than the CPNPP Units 3 and 4 plant grade elevation of 822 ft msl. FlowMaster results for the channel geometry sensitivity analysis are included in Appendix A.

Channel Slope Sensitivity Analysis

A channel slope sensitivity analysis was performed by decreasing the slope by 50 percent to account for any variability in the channel slope due to sedimentation. Decreasing the slope causes a lower flow velocity resulting in a higher water surface elevation. Increasing the slope will cause a higher flow velocity resulting in a lower surface elevation. Scouring potential due to the high flow rate and velocity will be mainly along the Brazos River. The resulting water surface elevation for the channel slope sensitivity analysis at the Paluxy River and the Brazos River confluence cross section is 774.99 ft. msl. The resulting water surface elevation for the channel slope sensitivity analysis at the upstream cross section is 747.10 ft. msl. The resulting water surface elevation for the channel slope sensitivity analysis at the downstream cross section is 725.45 ft. msl. The resulting water surface elevations are less than the CPNPP Units 3 and 4 plant grade elevation of 822 ft. msl. FlowMaster results for the channel slope sensitivity analysis are included in Appendix A.

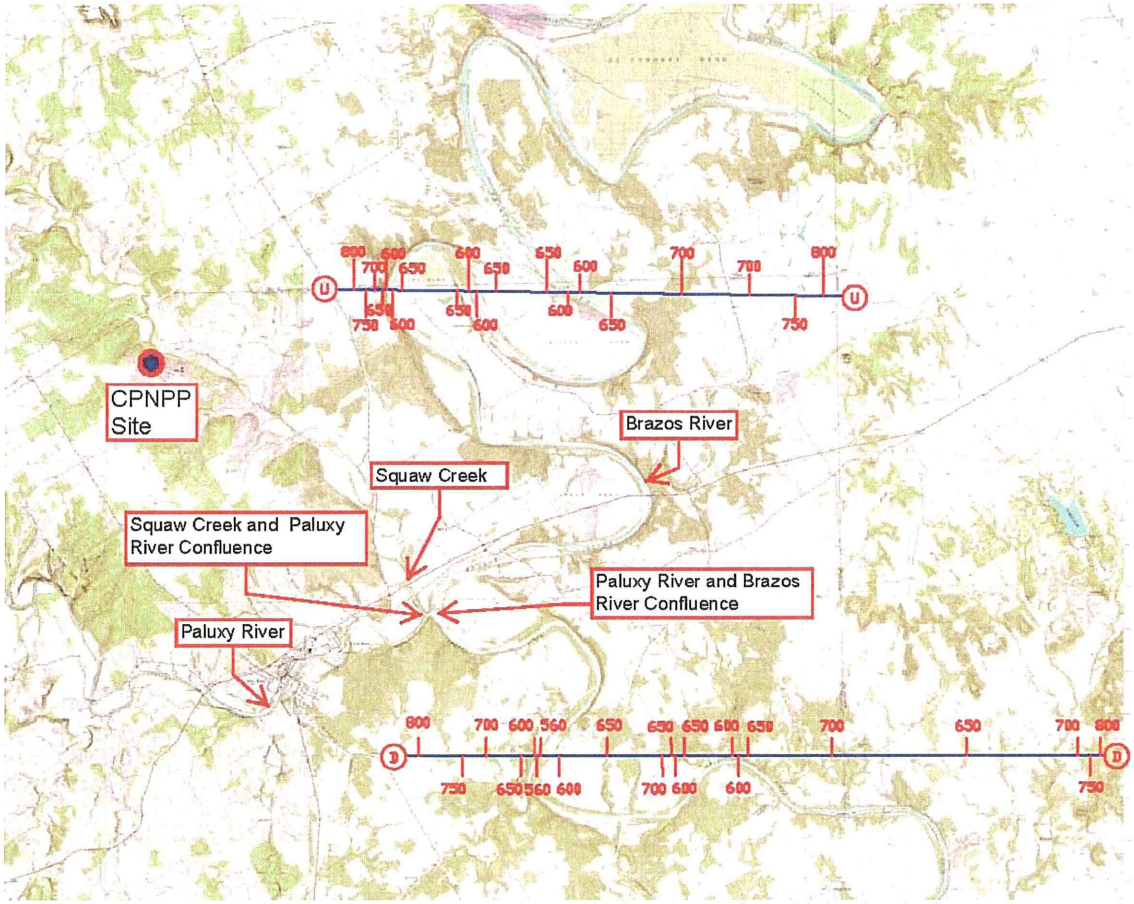


Figure 7-18 Brazos River Cross Sections for Channel Geometry Sensitivity Analysis

8.0 Appendices

FlowMaster results are in Appendix A.

Dam Failures Coincident with PMF - Water Surface Elevation

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00032	ft/ft
Discharge	6657069.00	ft³/s

Section Definitions

Station (ft)	Elevation (ft)
-89+76	800.00
-72+54	750.00
-46+05	700.00
-34+49	650.00
-6+32	600.00
-1+85	560.00
1+85	560.00
8+66	600.00
77+90	650.00
82+18	700.00
109+52	700.00
130+38	750.00
151+88	750.00
161+72	800.00

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(-89+76, 800.00)	(161+72, 800.00)	0.150

Results

Normal Depth	194.01	ft
Elevation Range	560.00 to 800.00 ft	
Flow Area	1943948.62	ft²
Wetted Perimeter	22667.73	ft

Dam Failures Coincident with PMF - Water Surface Elevation

Results

Top Width		22658.93	ft
Normal Depth		194.01	ft
Critical Depth		77.39	ft
Critical Slope		0.11053	ft/ft
Velocity		3.42	ft/s
Velocity Head		0.18	ft
Specific Energy		194.19	ft
Froude Number		0.07	
Flow Type	Subcritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	194.01	ft
Critical Depth	77.39	ft
Channel Slope	0.00032	ft/ft
Critical Slope	0.11053	ft/ft

Dam Failures Coincident with PMF - Water Surface Elevation C/S

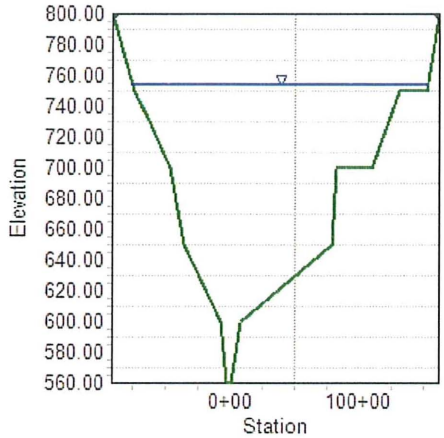
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00032	ft/ft
Normal Depth	194.01	ft
Discharge	6657069.00	ft³/s

Cross Section Image



Paluxy River-Squaw Creek and Brazos River C/S Capacity

Project Description

Friction Method	Manning Formula
Solve For	Discharge

Input Data

Channel Slope	0.00032	ft/ft
Normal Depth	240.00	ft

Section Definitions

Station (ft)	Elevation (ft)
-89+76	800.00
-72+54	750.00
-46+05	700.00
-34+49	650.00
-6+32	600.00
-1+85	560.00
1+85	560.00
8+66	600.00
77+90	650.00
82+18	700.00
109+52	700.00
130+38	750.00
151+88	750.00
161+72	800.00

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(-89+76, 800.00)	(161+72, 800.00)	0.100

Results

Discharge		19661842.11	ft^3/s
Elevation Range	560.00 to 800.00 ft		
Flow Area		3043310.00	ft^2
Wetted Perimeter		25158.64	ft

Paluxy River-Squaw Creek and Brazos River C/S Capacity

Results

Top Width		25148.00	ft
Normal Depth		240.00	ft
Critical Depth		104.31	ft
Critical Slope		0.04115	ft/ft
Velocity		6.46	ft/s
Velocity Head		0.65	ft
Specific Energy		240.65	ft
Froude Number		0.10	
Flow Type	Subcritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	240.00	ft
Critical Depth	104.31	ft
Channel Slope	0.00032	ft/ft
Critical Slope	0.04115	ft/ft

Paluxy River-Squaw Creek and Brazos River C/S Capacity

Project Description

Friction Method	Manning Formula
Solve For	Discharge

Input Data

Channel Slope	0.00032	ft/ft
Normal Depth	240.00	ft

Section Definitions

Station (ft)	Elevation (ft)
-89+76	800.00
-72+54	750.00
-46+05	700.00
-34+49	650.00
-6+32	600.00
-1+85	560.00
1+85	560.00
8+66	600.00
77+90	650.00
82+18	700.00
109+52	700.00
130+38	750.00
151+88	750.00
161+72	800.00

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(-89+76, 800.00)	(161+72, 800.00)	0.150

Results

Discharge		13107894.74	ft^3/s
Elevation Range	560.00 to 800.00 ft		
Flow Area		3043310.00	ft^2
Wetted Perimeter		25158.64	ft

Paluxy River-Squaw Creek and Brazos River C/S Capacity

Results

Top Width		25148.00	ft
Normal Depth		240.00	ft
Critical Depth		93.27	ft
Critical Slope		0.10064	ft/ft
Velocity		4.31	ft/s
Velocity Head		0.29	ft
Specific Energy		240.29	ft
Froude Number		0.07	
Flow Type	Subcritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	240.00	ft
Critical Depth	93.27	ft
Channel Slope	0.00032	ft/ft
Critical Slope	0.10064	ft/ft

Channel Geometry Sensitivity Analysis Upstream

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00032	ft/ft
Discharge	6657069.00	ft³/s

Section Definitions

Station (ft)	Elevation (ft)
0+00	800.00
8+96	750.00
16+01	700.00
21+74	650.00
25+12	600.00
29+53	600.00
37+02	650.00
78+60	650.00
88+00	600.00
94+17	600.00
109+06	650.00
147+95	650.00
164+52	600.00
173+22	600.00
197+56	650.00
251+29	700.00
303+49	700.00
338+67	750.00
360+57	800.00

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00, 800.00)	(360+57, 800.00)	0.150

Channel Geometry Sensitivity Analysis Upstream

Results

Normal Depth		129.89	ft
Elevation Range	600.00 to 800.00 ft		
Flow Area		2211349.11	ft²
Wetted Perimeter		31285.02	ft
Top Width		31272.56	ft
Normal Depth		129.89	ft
Critical Depth		43.45	ft
Critical Slope		0.10992	ft/ft
Velocity		3.01	ft/s
Velocity Head		0.14	ft
Specific Energy		130.03	ft
Froude Number		0.06	
Flow Type	Subcritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	129.89	ft
Critical Depth	43.45	ft
Channel Slope	0.00032	ft/ft
Critical Slope	0.10992	ft/ft

Channel Geometry Sensitivity Analysis Upstream C/S

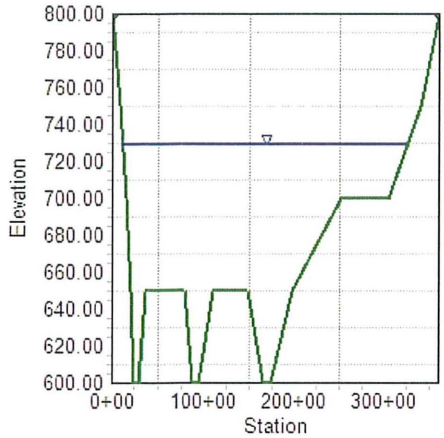
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00032	ft/ft
Normal Depth	128.89	ft
Discharge	6512713.00	ft³/s

Cross Section Image



Channel Geometry Sensitivity Analysis Downstream

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00032	ft/ft
Discharge	6657069.00	ft^3/s

Section Definitions

Station (ft)	Elevation (ft)
0+00	800.00
34+05	750.00
52+32	700.00
79+00	650.00
89+32	600.00
91+24	560.00
93+84	560.00
108+14	600.00
145+41	650.00
187+44	700.00
255+56	650.00
258+70	600.00
265+75	650.00
302+18	600.00
307+17	600.00
314+47	650.00
379+14	700.00
482+71	650.00
568+18	700.00
577+75	750.00
585+40	800.00

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient

Channel Geometry Sensitivity Analysis Downstream

Input Data

Start Station	Ending Station	Roughness Coefficient
(0+00, 800.00)	(585+40, 800.00)	0.150

Results

Normal Depth		153.44	ft
Elevation Range	560.00 to 800.00 ft		
Flow Area		2716993.05	ft²
Wetted Perimeter		52350.33	ft
Top Width		52334.37	ft
Normal Depth		153.44	ft
Critical Depth		73.02	ft
Critical Slope		0.11149	ft/ft
Velocity		2.45	ft/s
Velocity Head		0.09	ft
Specific Energy		153.53	ft
Froude Number		0.06	
Flow Type	Subcritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	153.44	ft
Critical Depth	73.02	ft
Channel Slope	0.00032	ft/ft
Critical Slope	0.11149	ft/ft

Channel Geometry Sensitivity Analysis Downstream C/S

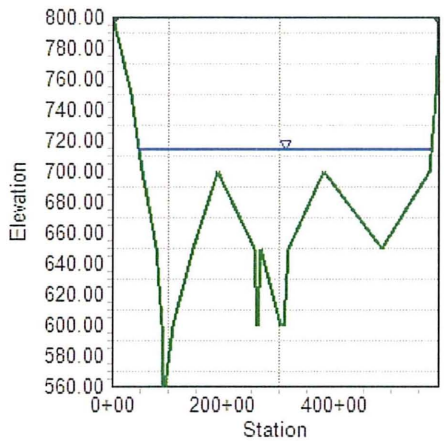
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00032	ft/ft
Normal Depth	153.44	ft
Discharge	6657069.00	ft³/s

Cross Section Image



Brazos River & Paluxy River Confluence CS - Channel Slope Sensitivity

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00016	ft/ft
Discharge	6657069.00	ft³/s

Section Definitions

Station (ft)	Elevation (ft)
-89+76	800.00
-72+54	750.00
-46+05	700.00
-34+49	650.00
-6+32	600.00
-1+85	560.00
1+85	560.00
8+66	600.00
77+90	650.00
82+18	700.00
109+52	700.00
130+38	750.00
151+88	750.00
161+72	800.00

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(-89+76, 800.00)	(161+72, 800.00)	0.150

Results

Normal Depth	214.99	ft
Elevation Range	560.00 to 800.00 ft	
Flow Area	2431386.08	ft²
Wetted Perimeter	23804.33	ft

Brazos River & Paluxy River Confluence CS - Channel Slope Sensitivity

Results

Top Width		23794.69	ft
Normal Depth		214.99	ft
Critical Depth		77.39	ft
Critical Slope		0.11053	ft/ft
Velocity		2.74	ft/s
Velocity Head		0.12	ft
Specific Energy		215.11	ft
Froude Number		0.05	
Flow Type	Subcritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	214.99	ft
Critical Depth	77.39	ft
Channel Slope	0.00016	ft/ft
Critical Slope	0.11053	ft/ft

Brazos River & Paluxy River Confluence CS - Channel Slope Sensitivity

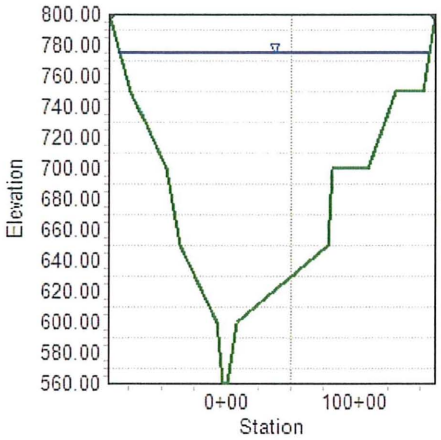
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00016	ft/ft
Normal Depth	214.99	ft
Discharge	6657069.00	ft³/s

Cross Section Image



Channel Slope Sensitivity Analysis Upstream

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00016	ft/ft
Discharge	6657069.00	ft³/s

Section Definitions

Station (ft)	Elevation (ft)
0+00	800.00
8+96	750.00
16+01	700.00
21+74	650.00
25+12	600.00
29+53	600.00
37+02	650.00
78+60	650.00
88+00	600.00
94+17	600.00
109+06	650.00
147+95	650.00
164+52	600.00
173+22	600.00
197+56	650.00
251+29	700.00
303+49	700.00
338+67	750.00
360+57	800.00

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00, 800.00)	(360+57, 800.00)	0.150

Channel Slope Sensitivity Analysis Upstream

Results

Normal Depth		147.10	ft
Elevation Range	600.00 to 800.00 ft		
Flow Area		2761964.09	ft²
Wetted Perimeter		32739.07	ft
Top Width		32725.87	ft
Normal Depth		147.10	ft
Critical Depth		43.45	ft
Critical Slope		0.10992	ft/ft
Velocity		2.41	ft/s
Velocity Head		0.09	ft
Specific Energy		147.19	ft
Froude Number		0.05	
Flow Type	Subcritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	147.10	ft
Critical Depth	43.45	ft
Channel Slope	0.00016	ft/ft
Critical Slope	0.10992	ft/ft

Channel Slope Sensitivity Analysis Upstream C/S

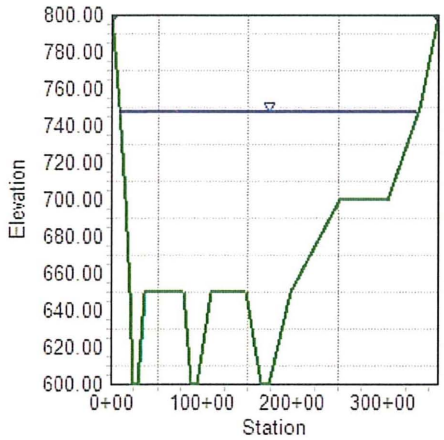
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00016	ft/ft
Normal Depth	147.10	ft
Discharge	6657069.00	ft^3/s

Cross Section Image



Channel Slope Sensitivity Analysis Downstream

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00016	ft/ft
Discharge	6657069.00	ft³/s

Section Definitions

Station (ft)	Elevation (ft)
0+00	800.00
34+05	750.00
52+32	700.00
79+00	650.00
89+32	600.00
91+24	560.00
93+84	560.00
108+14	600.00
145+41	650.00
187+44	700.00
255+56	650.00
258+70	600.00
265+75	650.00
302+18	600.00
307+17	600.00
314+47	650.00
379+14	700.00
482+71	650.00
568+18	700.00
577+75	750.00
585+40	800.00

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient

Channel Slope Sensitivity Analysis Downstream

Input Data

Start Station	Ending Station	Roughness Coefficient
(0+00, 800.00)	(585+40, 800.00)	0.150

Results

Normal Depth		165.45	ft
Elevation Range	560.00 to 800.00 ft		
Flow Area		3349386.73	ft²
Wetted Perimeter		53019.36	ft
Top Width		53002.93	ft
Normal Depth		165.45	ft
Critical Depth		73.02	ft
Critical Slope		0.11149	ft/ft
Velocity		1.99	ft/s
Velocity Head		0.06	ft
Specific Energy		165.51	ft
Froude Number		0.04	
Flow Type	Subcritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

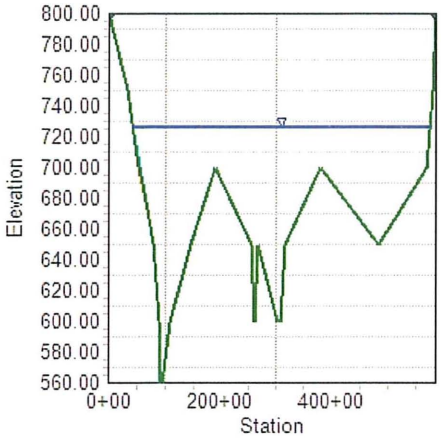
Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	165.45	ft
Critical Depth	73.02	ft
Channel Slope	0.00016	ft/ft
Critical Slope	0.11149	ft/ft

Channel Slope Sensitivity Analysis Downstream C/S

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00016	ft/ft
Normal Depth	165.45	ft
Discharge	6657069.00	ft^3/s

Cross Section Image