August 28 2009

Comanche Peak Nuclear Power Plant, Units 3 & 4
COL Application

Part 2, FSAR

Update Tracking Report

Revision 4

Revision History

Revision	Date	Update Description
0	3/31/2009	Original Issue Updated Chapters: Ch.1, 2, 3, 5, 6, 8, 9, 11, 12, 13, 14, 17 and 19 Incorporated responses to following RAIs: No.1
1	4/24/2009	Updated Chapters: Ch. 2, 6
-	5/1/2009	Updated Chapters: Ch. 1, 5,14 See Luminant Letter no. TXNB-09010 Date 5/1/2009 Incorporated responses to following RAIs: No. 1, 2
2	5/08/2009	Updated Chapters: Ch 1, 2
-	5/26/2009	Updated Chapters: Ch. 7 See Luminant Letter no. TXNB-09020 Date 5/26/2009 Incorporated responses to following RAIs: No. 4, 5
-	6/17/2009	Updated Chapters: Ch. 1,10 See Luminant Letter no. TXNB-09023 Date 6/17/2009 Incorporated responses to following RAIs: No. 6
3	6/30/2009	Updated Chapters: Ch 3 , 9,10,12,14,19
-	8/7/2009	Updated Chapters: Ch. 1, 5, 10

		See Luminant Letter no. TXNB-09028 Date 8/7/2009 Incorporated responses to following RAIs: No. 7, 8
-	8/24/2009	Updated Chapters: Ch. 1, 3, 10 See Luminant Letter no. TXNB-09033 Date 8/24/2009 Incorporated responses to following RAIs: No. 12, 16
-	8/24/2009	Updated Chapters: Ch. 1, 3, 10 See Luminant Letter no. TXNB-09034 Date 8/24/2009 Incorporated responses to following RAIs: No. 17, 20
4	8/28/2009	Updated Chapters: Ch 2, 3, 4, 5, 6, 7, 8, 10, 11, 12, 13, 14

Chapter 1

Chapter 1 Tracking Report Revision List

Change ID No.	Section	FSAR Rev. 0 Page	Reason for change	Change Summary	Rev. of FSAR T/R
CTS-00586	1.2	1.2-3 1.2-4	Consistent with Subsection 9.4.5.2.6	Add “UHS” before “ESW pump”.	0
CTS-00586	1.2	1.2-4	Erratum	Change the number of pumps.	0
CTS-00534	1.8	1.8-13	Consistent with DCD Rev.1	Correct COL 3.2(4) and 3.2(5) to reflect wording changes in DCD Rev1.	0
CTS-00535	1.8	1.8-16	Consistent with DCD Rev.1	Correct COL3.5(2) to reflect wording changes in DCD Rev1.	0
CTS-00536	1.8	1.8-23	Editorial correction	Change “AD/V2” to “AD/V^2”.	0
CTS-00537	1.8	1.8-28	Consistent with DCD Rev.1	Correct COL3.8(19) to reflect wording changes in DCD Rev1.	0
CTS-00527	1.8	1.8-30	Consistent with DCD Rev.1	Correct COL3.9(2) to reflect wording changes in DCD Rev1.	0
CTS-00538	1.8	1.8-33	Consistent with DCD Rev.1	Correct COL3.10(9) to reflect wording changes in DCD Rev1.	0
CTS-00550	1.8	1.8-41	Editorial correction	Delete “these” from COL 6.2(1).	0
CTS-00539	1.8	1.8-43	Editorial correction	Add “and” in COL 6.4(5).	0
CTS-00540	1.8	1.8-55	Editorial correction	Change “an” to “a ” in COL10.3(1).	0
CTS-00541	1.8	1.8-56	Editorial correction	Change “deta” to “data” in COL11.2(3).	0
CTS-00542	1.8	1.8-61	Consistent with DCD Rev.1	Correct COL12.1(1) to reflect wording changes in DCD Rev1.	0
DCD_12.01-2	1.8	1.8-61	Delete Outdated RG	Delete reference to RG8.20, 8.26, and 8.32 from COL12.1(3).	0
CTS-00543	1.8	1.8-64	Consistent with DCD Rev.1	Correct COL13.1(5), 13.2(2) and 13.2(3) to reflect wording changes in DCD Rev1.	0
CTS-00610	13.5.2	1.8-66	Update	Add Subsection “13.5.2.1” in Table 1.8-201.	0
CTS-00544	1.8	1.8-67	Consistent with DCD Rev.1	Correct COL13.6(1)and 13.7(1) to reflect wording changes in DCD Rev1.	0
CTS-00545	1.8	1.8-70	Consistent with DCD Rev.1	Delete COL16.1_3(1).	0
CTS-00546	1.8	1.8-71	Editorial correction	Delete “and” from COL16.1_3.3.2(1).	0

Change ID No.	Section	FSAR Rev. 0 Page	Reason for change	Change Summary	Rev. of FSAR T/R
CTS-00526	1.8	1.8-74	Consistent with DCD Rev.1	Correct COL17.5(1) to reflect wording changes in DCD Rev1.	0
CTS-00530	1.9	1.9-7	Correct Corresponding Section	Delete reference to 5.2.1.2 from RG1.84.	0
CTS-00529	1.9	1.9-16	Correct COLA/FSAR Status	Add “with exceptions” to “Conformance” in RG 4.15.	0
DCD_12.01-2	1.9	1.9-18 1.9-19	Delete Outdated RG	Delete reference to RG8.20, 8.26, and 8.32 from Table1.9-203.	0
RCOL2_14.03-1	Table 1.8-201	1.8-69	Responses to RAI No. 1 Luminant Letter TXNB-09010 Dated 5/1/2009	Add FSAR location “14.2.12.1.90.C8” as resolution of COL 14.2(10).	-
CTS-00703	Table 1.9-201	1.9-4	To Reflect CPNPP Units 3 and 4 compliance with RG 1.23.	Added “Second Prepared Revision, April 1986” in the Revision/Date category and “revision of record CPNPP Units 1 and 2” to the COLA FSAR Status category.	2
RCOL2_10.02.03-01	Table 1.8-201	1.8-54	Response to RAI No. 6 Luminant Letter no.TXNB-09023 Date 06/17/2009	For COL 10.2(1), replace the word “develop” with “establish a” and delete “and then to implement” in the first sentence. Delete the entire second sentence. Insert “A” under the column “COL Applicant Item”; delete “H” and delete “b” from columns labeled “COL Holder Item” and “Rationale”.	-
RCOL2_10.03.06-2	Table 1.8-201	1.8-55	Response to RAI No. 7 Luminant Letter no.TXNB-09028 Date 8/7/2009	Replace the revision number for NSAC-202L from “R3” to “R2”. Insert “and are susceptible to erosion-corrosion damage” at end of 1st sentence for COL 10.3(1).	-
RCOL2_10.03-1	Table 1.8-201	1.8-55	Response to RAI No. 16 Luminant Letter no.TXNB-09033 Date 08/24/2009	Delete COL 10.3(2) description and state “Delete from DCD”.	-

Change ID No.	Section	FSAR Rev. 0 Page	Reason for change	Change Summary	Rev. of FSAR T/R
RCOL2_01-1	Table 1.7-202	1.7-3	Response to RAI No. 20 Luminant Letter no.TXNB-09034 Date 08/24/2009	Delete Figure 9.2.4-201, “Sanitary Wastewater Treatment System Flow Diagram,” from Table 1.7-202.	-

Chapter 2

Chapter 2 Tracking Report Revision List

Change ID No.	Section	FSAR Rev. 0 Page	Reason for change	Change Summary	Rev. of FSAR T/R
CTS-00636	Table 2.0-1R	2.0-3 2.0-13	Editorial correction	Change "X/Q" to "χ/Q". (χ is a Greek letter.)	0
CTS-00637	Table 2.2-203 Table 2.2-206	2.2-28 2.2-33	Editorial correction	Change "CPNPP Units 1 & 2" to "CPNPP Units 1 and 2".	0
CTS-00587	Table 2.3-206	2.3-71	Erratum	Change "5" to "3".	0
CTS-00636	Table 2.3-342	2.3-252 2.3-253	Editorial correction	Change "X/Q" to "χ/Q". (χ is a Greek letter.)	0
CTS-00590	2.4.1.1	2.4-2	Editorial correction	Change "grade" to "floor elevation".	0
CTS-00591	2.4.1.1	2.4-3	Editorial correction	Change "Category I seismic requirement" to "seismic category I requirement".	0
CTS-00661	2.4.1.2.1	2.4-5	Editorial correction	Add "(Figure 2.4.1-207)" after Morris-Sheppard Dam.	0
CTS-00662	2.4.1.2.1	2.4-6	Editorial correction	Add reference numbers according to CTS-00666.	0
CTS-00592	2.4.1.2.3.2	2.4-7	Editorial correction	Change "intake pumping station" to "makeup water intake structure" and "cooling tower makeup pumps" to "makeup water pumps, makeup water jockey pump".	0
CTS-00663	2.4.1.2.3.3	2.4-8	Editorial correction	Add reference numbers as appropriate according to CTS-00666.	0
CTS-00664	2.4.1.2.3.3	2.4-8	Editorial correction	Delete "contributing".	0
CTS-00665	2.4.1.2.3.3	2.4-8	Update	Change "16,113 sq mi" to "25,679 sq mi".	0
CTS-00593	2.4.11.5	2.4-38	Editorial correction	Remove "to the cooling water system flow".	0
CTS-00655	2.4.12.2.4	2.4-46	Editorial correction	Change "X" to "XX".	0
CTS-00513 RCOL2_ 2.4.13-1 through RCOL2_ 2.4.13-7	2.4.12.2.4 2.4.12.2.5 2.4.12.3.1 2.4.12.5 2.4.13	2.4-46 through 2.4-64	To reflect information provided during acceptance review	Re-write section reflecting RAI #1.	0

Change ID No.	Section	FSAR Rev. 0 Page	Reason for change	Change Summary	Rev. of FSAR T/R
CTS-00656	2.4.12.3.1	2.4-51	Editorial correction	Delete “(or are) expected to be”.	0
CTS-00657	2.4.12.3.1	2.4-52	Editorial correction	Change X to lower-case in mathematical expressions.	0
CTS-00658	2.4.12.5	2.4-53	Editorial correction	Add “aquifer”.	0
CTS-00659	2.4.13	2.4-56	Editorial correction	Change “Kd” to K_d”.	0
CTS-00666	2.4.16	2.4-63	Editorial correction	Add new references.	0
CTS-00589	Table 2.4.1-203	2.4-68 through 2.4-70	Erratum	Add reference citations.	0
CTS-00654	Table 2.4.1-203	2.4-68 through 2.4-70	Editorial correction	Change header titles and lower case from MSL to msl.	0
CTS-00655	Table 2.4.1-203	2.4-68 through 2.4-70	Erratum	Change values to match reference.	0
CTS-00588	Table 2.4.1-206	2.4-72	Erratum	Change “8186” to” 6354” and “0.383” to “0.362”. Add reference citations.	0
CTS-00594	2.5.1	2.5-53	Clarification	Add “potable” and “beneath the site”.	0
CTS-00599	2.5.2	2.5-61 2.5-62	Editorial correction	Delete the semi-colon in the bullet item list.	0
CTS-00595	2.5.2	2.5-61	Editorial correction	Remove IBR statement.	0
CTS-00515	2.5.2.5.1	2.5-110 through 2.5-113	To reflect information provided during acceptance review	Add three pages to clarify discussion.	0
CTS-00516	2.5.2.6.1.1 2.5.2.6.1.2	2.5-113 2.5-117	To reflect information provided during acceptance review	Revise Subsection reflecting commitment to NRC.	0
CTS-00667	2.5.4.3.3	2.5-166	Editorial correction	Change "The average elevation of the top of engineering Layer C is about 780 ft to 782 ft below the Unit 3 power block, and about 782 ft to 784 ft below the Unit 4 power block (Figure 2.5.4-214)." to "The average elevation of the top of engineering Layer C is approximately 782 ft below the Unit 3 and Unit 4 power block (Figure 2.5.4-214)".	0
CTS-00597	2.5.4	2.5-121	Editorial correction	Remove IBR statement.	0

Change ID No.	Section	FSAR Rev. 0 Page	Reason for change	Change Summary	Rev. of FSAR T/R
CTS-00514	2.5.4.5.4	2.5-177 2.5-179	To reflect information provided during acceptance review	Revise Subsection reflecting commitment to NRC.	0
CTS-00517	2.5.4.8	2.5-187	To reflect information provided during acceptance review	Revise Subsection reflecting commitment to NRC.	0
CTS-00598	2.5.5	2.5-195	Editorial correction	Remove IBR statement.	0
CTS-00515	2.5.2.5	2.5-224	Editorial correction	Revise Subsection reflecting commitment to NRC.	0
CTS-00515	2.5.7	2.5-227 2.5-228	To reflect information provided during acceptance review	Add references 2.5-432 through 2.5-436	0
CTS-00515	2.5.7	2.5-228	To reflect information provided during acceptance review	Add reference 2.5-432.	0
CTS-00668	Table 2.5.1-201	2.5-229 2.5-230	Editorial correction	Delete "from the Studies of Madole (1988), Crone and Luza (1990), and Swan et al. (1993)" from the title of the table.	0
CTS-00669	Table 2.5.1-201	2.5-230	Editorial correction	Add reference citations.	0
CTS-00672	Table 2.5.1-202	2.5-231	Editorial correction	Delete notes.	0
CTS-00673	Table 2.5.1-203	2.5-232	Editorial correction	Add reference citations.	0
CTS-00673	Table 2.5.1-203	2.5-232	Editorial correction	Delete and rewrite notes.	0
CTS-00670	Table 2.5.1-205	2.5-252	Editorial correction	Add reference citations.	0
CTS-00671	Table 2.5.1-206	2.5-254	Editorial correction	Add reference citations.	0
CTS-00674	Table 2.5.2-227	2.5-312	Editorial correction	Delete references in notes.	0
CTS-00515	List of Tables List of Figures	2-xxxii 2-xlviii	Commitment to NRC	Add Tables 2.5.2-230 through 2.5.2-235. Add Figures 2.5.2-240 through 2.5.2-246.	0

Change ID No.	Section	FSAR Rev. 0 Page	Reason for change	Change Summary	Rev. of FSAR T/R
CTS-00516	List of Tables List of Figures	2-xxxii 2-xlviii	Commitment to NRC	Add Tables 2.5.2-236 and 2.5.2-237. Add Figures 2.5.2-247 through 2.5.2-252.	0
CTS-00515	Tables 2.5.2-230 through 2.5.2-237	-	To reflect information provided during acceptance review	Add new Tables.	0
CTS-00516	Figures 2.5.2-240 through 2.5.2-250	-	To reflect information provided during acceptance review	Add new Figures	0
MET-04	List of Tables	2-xxiv, 2-xxv	Erratum	Add “Dallas” in front of “Fort Worth” and “Airport” after “Fort Worth” for table number 2.3-296	1
CTS-00696	2.2.2.2.8	2.2-5	Increase information as discussed with NRC during the 03-23-25-09 Hazards Analysis Audit	Changed distance for DeCordova to 9.35 miles.	1
CTS-00697	2.2.2.6	2.2-8	Increase information as discussed with NRC during the 03-23-25-09 Hazards Analysis Audit	Added clarification that rail transport of hazardous materials is outside the 5 mile radius of CPNPP 3 & 4	1
CTS-00699	2.2.2.7.1	2.2-9	Increase information as discussed with NRC during the 03-23-25-09 Hazards Analysis Audit	Added clarifying statement that the airports listed were predominant airports in the area outside 10 miles that did not exceed the 1000 D^2 criterion. Added back in the discussion for each predominant airport in the area outside the 10 miles.	1
CTS-00698	2.2.3.1.1.2	2.2-12	Increase information as discussed with NRC during the 03-23-25-09 Hazards Analysis Audit	Added clarifying discussion on how the Wolf Hollow hazardous materials were sceened for the hazards analysis since quantities were not made available.	1
CTS-00698	2.2.3.1.3.1	2.2-17	Increase information as discussed with NRC during the 03-23-25-09 Hazards	Added clarifying discussion on how the Wolf Hollow hazardous materials were sceened for the control room	1

Change ID No.	Section	FSAR Rev. 0 Page	Reason for change	Change Summary	Rev. of FSAR T/R
			Analysis Audit	habitability analysis since quantities were not made available.	
CTS-00696	2.2.3.1.3.2.2	2.2-18	Increase information as discussed with NRC during the 03-23-25-09 Hazards Analysis Audit	Clarified discussion regarding DeCordova was analyzed for Hazards and Control Room Habitablilty analyses even though the distance is outside the 5 mile radius of Units 3 & 4.	1
CTS-00698	Table 2.2-205	2.2-32	Increase information as discussed with NRC during the 03-23-25-09 Hazards Analysis Audit	Added footnote that the quantities of chemicals were not made available for Wolf Hollow and a pointer added to indicate what sections have the sceening criteria utilized for Wolf Hollow.	1
CTS-00696	Table 2.2-214	2.2-43	Increase information as discussed with NRC during the 03-23-25-09 Hazards Analysis Audit	Added IDLH and Max concentration in Control Room and footnote (b) indicating that DeCordova was conservatively analyzed even though it is outside the 5 mile radius of U3/4. Distance to nearest Units 3 and 4 MCR Inlet for DeCordova SES has been revised from 3.6 to 3.7.	1
CTS-00696	Figure 2.2-201		Erratum	Corrected the figure since the location of DeCordova, which is outside the 5 mile radius of CPNPP Units 3 & 4, showed DeCordova inside the 5 mile radius	1
MET-03	2.3.1.2.4	2.3-14	Increase information as discussed with the NRC.	Add “16” to number of days each year; remove “monthly and regional” and add “by county” to wind events to reconcile thunderstorm information.	1
MET-04	2.3.1.2.8	2.3-20	Erratum	Add “the” in front of Dallas Fort Worth Airport	1
MET-13	2.3.2.1.2	2.3-22	Erratum	Replace “2001 through 2006” with “2001 – 2004 and 2006” to describe which data years were used.	1
MET-13	2.3..2.1.3	2.3-27	Erratum	Replace “2001- 2006” with “2001 – 2004 and 2006” to	1

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				describe which data years were used.	
MET-04	2.3.2.1.4	2.3-27	Erratum	Add “Dallas” in front of “Fort Worth”	1
MET-13	2.3.2.2.4	2.3-32	Erratum	Add “Fort” for the years “2001 – 2006”	1
MET-3 MET-13	Table 2.3-211	2.3-83	Erratum	Replace numbers in column “Average per Yr (#/yr) and Replace “2006 and (-24 yr) with “7/31/2006”	1
MET-13	Table 2.3-285	2.3-164	Errata	Replace “2001 – 2006” with “2001 – 2004 and 2006” to describe which data years were used.	1
MET-04	Table 2.3-286	2.3-165	Erratum	Add “Dallas” in front of “Fort Worth” for the title.	1
MET-04	Table 2.3-296	2.3-177	Erratum	Add “Dallas” in front of Fort Worth and “Airport” after Worth in the title	1
MET-04	Table 2.3-299	2.3-180 2.3-181	Erratum	Add “Dallas” in front of ”Fort Worth” in the title	1
CTS-00554	List of Tables	2-xxxiii	Increase information as discussed with the NRC to summarize the reports provided in Luminant’s letter TXNB-08027 to NRC dated November 4, 2008.	Added Tables 2.5.4-228 through 2.5.4-231	2
CTS-00554	List of Figures	2-I	Increase information as discussed with the NRC to summarize the reports provided in Luminant’s letter TXNB-08027 to NRC dated November 4, 2008.	Added Figure 2.5.4-245	2
CTS-00703	Table 2.3-332	2.3-233 2.3-234	To reflect CPNPP Units 3 and 4 compliance with RG 1.23	Added “Second Proposed Revision, April 1986” to the footnotes	2
CTS-00554	2.5.4.10.1	2.5-189	Increase information as discussed with the NRC to summarize the reports provided in	Additional discussion and equations to reflect what calculations and analyses were performed to demonstrate bearing	2

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			Luminant's letter TXNB-08027 to NRC dated November 4, 2008.	capacity.	
CTS-00554	2.5.4.10.2	2.5-190	Increase information as discussed with the NRC to summarize the reports provided in Luminant's letter TXNB-08027 to NRC dated November 4, 2008.	Additional discussion on settlement, including calculations, equations and discussion of laboratory test results, layered versus unlayered method.	2
CTS-00554	2.5.4.10.3	2.5-191	Increase information as discussed with the NRC to summarize the reports provided in Luminant's letter TXNB-08027 to NRC dated November 4, 2008.	Additional information added to excavation rebound potential.	2
CTS-00554	2.5.7	2.5-228	Increase information as discussed with the NRC to summarize the reports provided in Luminant's letter TXNB-08027 to NRC dated November 4, 2008.	Added references 2.5-432 through 2.5-434 to reflect additional discussion on bearing capacity and settlement subsection discussed.	2
CTS-00554	Tables 2.5-4-228 through 2.5.4-231	-	Increase information as discussed with the NRC to summarize the reports provided in Luminant's letter TXNB-08027 to NRC dated November 4, 2008.	Added new tables to reflect bearing capacity discussion and settlement discussion within subsections.	2
CTS-00554	Figure 2.5.4-245		Increase information as discussed with the NRC to summarize the reports provided in Luminant's letter TXNB-08027 to NRC dated November 4, 2008.	Added Figure 2.5.4-245.	2

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HYDSV-23	List of Figures	2xliv	Hydrology Site Safety Visist	Added figures to show flow paths to SCR.	4
HYDSV-06 HYDSV-07	Table 2.0-1R		Hydrology Site Safety Visit	Changed the maximum flood level.	4
HYDSV-04	2.4.1.2	2.4-4	Hydrology Site Safety Visit	Clarified what portions of the Brazos River basin were chosen for the dam failure safety analysis.	4
HYDSV-05	2.4.1.2	2.4-5	Hydrology Site Safety Visit	Updated section to reflect what reservoirs were considered in the dam failure safety analysis.	4
HYDSV-02	2.4.2.1	2.4-12 2.4-13	Hydrology Site Safety Visit	Added maximum flood level and design basis flood elevation.	4
HYDSV-14	2.4.2.2	2.4-13 2.4-14	Hydrology Site Safety Visit	Changed water surface elevation for flood design.	4
HYDSV-06 HYDSV-07	2.4.2.3	2.4-16	Hydrology Site Safety Visit	Changed the tail water elevation.	4
HYDSV-06 HYDSV-07	2.4.3	2.4-18	Hydrology Site Safety Visit	Revised the surface water elevation for the probably maximum flood.	4
HYDSV-06 HYDSV-07	2.4.3.1	2.4-19	Hydrology Site Safety Visit	Revised the critical temporal distribution for the probably maximum precipitation.	4
HYDSV-06 HYDSV-07	2.4.3.3	2.4-20 2.4-21	Hydrology Site Safety Visit	Added discussion justifying the use of the Snyder's hydrograph applicability under PMF conditions and added a storage discharge relationship was linearly extrapolated to account for discharge from elevation 791 ft msl to 795 ft. msl.	4
HYDSV-06 HYDSV-07	2.4.3.4	2.4-22	Hydrology Site Safety Visit	Changed the SCR peak flood volumetric flow rate.	4
HYDSV-06 HYDSV-07	2.4.3.5	2.4-22	Hydrology Site Safety Visit	Changed the surface water elevation for the HEC-HMS and HEC-RAS models.	4
HYDSV-06 HYDSV-07	2.4.3.6	2.4-22 2.4-23	Hydrology Site Safety Visit	Revised the critical fetch length, critical duration wind speed, wave height, runup,	4

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				maximum wind speed, and setup for the dam failure analysis.	
HYDSV-04	2.4.4	2.4-24	Hydrology Site Safety Visit	Clarified assumptions of what dam failures were used in the dam failure analysis and why.	4
HYDSV-09	2.4.4.1	2.4-27	Hydrology Site Safety Visit	Clarified which reservoirs in the Brazos River Basin where used in the flooding analysis. Added discussion of what volumes of reservoir water were used in the dam failure analysis. Changed the maximum surface water elevation.	4
CTS-00817 HYDSV-10 HYDSV-11	2.4.5	2.4-29	Hydrology Site Safety Visit	Edited 5th paragraph 2nd to last sentence of section from "Any effects on the Squaw Creek.. to read "Any effects on SCR…". Added discussion as to why the seismic induced wave and the landslide induced wave is not plausible for SCR. Changed the water surface elevation due to wind activity and changed the PMF coincident wind wave.	4
HYDSV-03	2.4.5	2.4-29	Hydrology Site Safety Visit	Clarified that the plant grade elevation is at 822 ft msl.	4
HYDSV-12 HYDSV-13	2.4.6	2.4-30	Hydrology Site Safety Visit	Added discussion that landslide and seismic induced waves are note plausible for SCR.	4
HYDSV-14	2.4.7	2.4-32	Hydrology Site Safety Visit	Changed the maximum flood elevation. Added a discussion regarding the maximum potential ice thickness and that freezing protection was provided for the ESWS cooling towers and ESW Pump House.	4
HYDSV-16	2.4.11.5	2.4-38	Hydrology Site	Added a discussion regarding the control of the ESWS and CWS cooling	4

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			Safety Visit	towers with makeup flow rates.	
HYDSV-20	2.4.12.2.4	2.4-46 2.4-47	Hydrology Site Safety Visit	Updated the Groundwater Level Fluctuations to include the 2008 precipitation data and the resulting effect on the groundwater level fluctuations results.	4
HYDSV-20	2.4.12.2.4	2.4-46 2.4-47	Hydrology Site Safety Visit	Removed previous RCOL2_2.4.4.13-4 addition of "undifferentiated fill/regolith and" as well as, "indicating perched groundwater at these locations."	4
HYDSV-18 HYDSV-24	2.4.12.2.5.1	2.4-49	Hydrology Site Safety Visit	Revised to clarify the conservatism used in porosity to calculate liquid effluent travel times.	4
HYDSV-23	2.4.12.3.1	2.4-51	Hydrology Site Safety Visit	Revised section to describe the post-construction movement o groundwater to support the liquid effluent release model provided in Section 2.4.13.	4
HYDSV-26	2.4.12.4	2.4-53	Hydrology Site Safety Visit	Revised to reflect that a groundwater monitoring program will be developed before fuel load.	4
CTS-00808 HYDSV-30	2.4.13	2.4-54	Hydrology Site Safety Visit	Corrected Figure typo to 2.4.12-209. Discussed the alternate conceptual model and added a reference to new Figures 2.4.12-212-214.	4
HYDSV-28	2.4.13.1	2.4-55	Hydrology Site Safety Visit	Clarified conclusion that no chemical agents could have an effect on the transport characteristics of the liquid effluent.	4
HYDSV-30	2.4.13.2	2.4-55	Hydrology Site Safety Visit	Added clarification regarding the alternate pathways chosen and introduced new Figures 2.4-12-212 through 2.4.12-214 showing the new pathways and cross sections and discussed the hydraulic gradient figures showing the reason why GW movement SE and SW are not plausible	4

Change ID No.	Section	FSAR Rev. 0 Page	Reason for change	Change Summary	Rev. of FSAR T/R
				release pathways.	
HYDSV-17 HYDSV-19 HYDSV-23 HYDSV-30	2.4.13.2	2.4-55	Hydrology Site Safety Visit	Added paragraph to introduce new cross section figures and pathway figure.	4
HYDSV-17 HYDSV-19 HYDSV-23 HYDSV-30	2.4.13.2	2.4-55	Hydrology Site Safety Visit	Added two more bullets on what alternate conceptual model parameters were used in developing the site conceptual model plausible pathways.	4
HYDSV-17 HYDSV-19 HYDSV-23 HYDSV-30	2.4.13.3	2.4-55	Hydrology Site Safety Visit	Added a discussion that rainfall infiltration is not a contributing factor that would affect the liquid effluent release analysis.	4
HYDSV-29 HYDSV-31	2.4.13.4	2.4-55	Hydrology Site Safety Visit	Corrected the distances to the nearest water supply wells both in the Glen Rose formation and the Twin Mountains formation.	4
HYDSV-17 HYDSV-19 HYDSV-29 HYDSV-31	2.4.13.4	2.4-61	Hydrology Site Safety Visit	Added a clarification as to why the vertical release pathway is not plausible based upon the Unit 1 and 2 study previously performed.	4
HYDSV-23	2.4.13.4	2.4-61	Hydrology Site Safety Visit	Added reference to new Cross Section figures and pathway Figures 2.4-12-212 through 2.4.12-214.	4
HYDSV-17 HYDSV-19 HYDSV-23 HYDSV-30	2.4.13.5	2.4-55	Hydrology Site Safety Visit	Revised to discuss four release pathways. Revised to include discussion of why alternate pathways moving SE or SW from Units 3 or 4 would not be plausible.	4
HYDSV-17 HYDSV-23 HYDSV-30	2.4.13.5	2.4-55	Hydrology Site Safety Visit	Changed to plausible pathways 3a, 3b, 4a, 4b and changed travel times to SCR, and deleted current pathways. Changed travel times and identified the shortest travel time to SCR. Referred to cross section figures and new pathways.	4
HYDSV-17 HYDSV-23 HYDSV-30	2.4.13.7	2.4-55	Hydrology Site Safety Visit	Revised base mat elevation for A/B and specified subsection for site specific hydrogeologic data and core	4

Change ID No.	Section	FSAR Rev. 0 Page	Reason for change	Change Summary	Rev. of FSAR T/R
				boring stratigraphy for A/B.	
HYDSV-17 HYDSV-23 HYDSV-30	2.4.13.7	2.4-55	Hydrology Site Safety Visit	Changed travel times for the new pathways, specified what subsection discusses the comparison of U1/2 vertical pathway study, and made minor editorials.	4
HYDSV-05	References 2.4-269 and 2.4-270	2.4-63	Hydrology Site Safety Visit	Added two new references to describe potential reservoir sites considered in the dam failure analysis.	4
HYDSV-15	References 2.4-271 and 2.4-272	2.4-63	Hydrology Site Safety Visit	Added two new references for the ice effects analysis Section 2.4.7.	4
HYDSV-02	Table 2.4.2-204	2.4-87	Hydrology Site Safety Visit	Added the datum elevation for footnote b.	4
HYDSV-06 HDYSV-07	Table 2.4.2-208	2.4-91	Hydrology Site Safety Visit	Changed the tail water elevation.	4
HYDSV-06 HYDSV-07	Table 2.4.3-202	2.4-93	Hydrology Site Safety Visit	Changed the PMP degree storm orientation.	4
HYDSV-06 HYDSV-07	Table 2.4.3-207	2.4-102	Hydrology Site Safety Visit	Changed the watershed sub-basin characteristics.	4
HYDSV-23	Table 2.4.12-211	2.4-149 through 2.4-152	Hydrology Site Safety Visit	Replaced Groundwater and Velocity Times Based Upon Post-Construction Configuration.	4
HYDSV-02	Figures 2.4.2-201 2.4.2-202 2.4.3-202 2.4.3-209 2.4.4-201 2.4.4-202	--	Hydrology Site Safety Visit	Added horizontal and vertical datums; added additional fetches; clarified watershed boundaries; and added datum sources.	4
HYDSV-20	Figure 2.4.12-209	--	Hydrology Site Safety Visit	Replaced the hydrographs for monitoring wells with expanded scale and precipitation data.	4
HYDSV-23	Figures 2.4.12-212 2.4.12-213 2.4.12-214	--	Hydrology Site Safety Visit	Added new Figures for Groundwater Flow Paths for Liquid Effluent Release and Cross Sections.	4

LIST OF FIGURES (Continued)

HYDSV-23

Table 2.0-1R (Sheet 9 of 12)
Key Site Parameters

CP COL 2.1(1) CP COL 2.2(1) CP COL 2.3(1) CP COL 2.3(2) CP COL 2.3(3) CP COL 2.4(1)	A/B releases (reactor coolant system sample line[f] 0-8 hrs 8-24 hrs 1-4 days 4-30 days	 4.9×10^{-3} s/m^3 2.9×10^{-3} s/m^3 1.8×10^{-3} s/m^3 8.1×10^{-4} s/m^3	Dispersion of releases from the reactor coolant sampling line are bounded by the dispersion values for the plant vent.
CP COL 2.5(1)	Air lock releases in containment[g] 0-8 hrs 8-24 hrs 1-4 days 4-30 days	 6.4×10^{-3} s/m^3 3.8×10^{-3} s/m^3 2.4×10^{-3} s/m^3 1.1×10^{-3} s/m^3	χ/Q values for the air lock releases in containment are bounded by the χ/Q for the Containment Shell release.
	Hydrologic Engineering		
	Parameter Description	**Parameter Value**	
		DCD	**CPNPP 3 and 4**
	Maximum flood (or tsunami) level	1 ft below plant grade	~~788.90~~790.9 ft msl for SCR ~~820.91~~820.83 ft msl for a Local Intense Precipitation at units 3 and 4 site. HYDSV-06 HYDSV-07
	Maximum rainfall rate (hourly)	19.4 in/hr for seismic category I/II structures	19.0 in/hr
	Maximum rainfall rate (short-term)	6.3 in/5 min for seismic category I/II structures	6.2 in/5 min

2.4.1 Hydrologic Description

CP COL 2.4(1) Replace the content of DCD Subsection 2.4.1 with the following.

This subsection describes regional and site hydrological conditions, specifically surface water and groundwater characteristics. Information provided in this subsection includes descriptions of the site and features, hydrosphere, hydrologic characteristics, drainage, dams and reservoirs, proposed water management changes, and surface water users.

2.4.1.1 Site and Facilities

Comanche Peak Nuclear Power Plant (CPNPP) Units 3 and 4 are located on the western end of a peninsula formed by the southern shore of Squaw Creek Reservoir (SCR) and the CPNPP Units 1 and 2 Safe Shutdown Impoundment, approximately 0.49 mi west-northwest of CPNPP Units 1 and 2 in Somervell County, Texas. The CPNPP site is located in Somervell and Hood Counties, Texas approximately 5.2 mi north-northwest of the town of Glen Rose, Texas (Figure 2.1-202).

CPNPP Units 3 and 4 are located approximately 0.49 mi west-northwest of CPNPP Units 1 and 2 as shown in Figure 2.1-201 and utilize mechanical draft cooling towers for circulating and service water system cooling. Cooling water comes from Lake Granbury located approximately 7.13 mi north-northeast of the CPNPP site.

Maximum relief in the CPNPP site area is approximately 220 ft, with elevations ranging from 640 ft to 860 ft above sea level, with slopes that are typically steep, ranging from 15 to 30 degrees or more, and generally exhibiting a stair-stepped appearance. Rock outcrops of limestone and claystone comprise approximately 40 to 60 percent of these slopes. The remaining areas, including the higher flat-topped plateau remnants, are mantled by a thin cover of soil, which at the surface generally consists of silt and sand (Reference 2.4-201). The standard plant ~~grade~~floor elevation of the safety-related facilities is established at 823 ft above msl. The center of the nonsafety-related mechanical draft cooling towers is located about 1,800 ft to the northwest of the CPNPP Unit 3 and 4 center point at a grade elevation of 850 ft msl (Figure 2.1-201). Locations and topographic profiles showing the relationship between the CPNPP site, SCR, and Lake Granbury are illustrated on Figures 2.4.1-201 and 2.4.1-202. Grading and drainage improvements are illustrated on Figure 2.4.2-202. CTS-00590

Lake Granbury, the source of cooling water for the cooling tower system, is discussed in detail in Subsection 2.4.1.2. Cooling water is expected to be withdrawn by an intake structure located approximately 1.31 mi upstream from the DeCordova Bend dam. The cooling water is pumped to the CPNPP Units 3 and 4 cooling system through two pipelines, and the blowdown water from the cooling water system is discharged through two separate pipelines back to Lake Granbury about 1.14 mi downstream from the intake structure. Figure 2.4.1-203 deplicts the

Lowland, and Coastal Plain (Reference 2.4-204). Watershed elevations range from about 4700 ft near the headwaters in eastern New Mexico to sea level near Freeport (Reference 2.4-201). Within the Brazos River Basin, the CPNPP site is located in the Middle-Brazos Lake Whitney watershed, USGS hydrologic unit code 12060202, and Lake Granbury is located in the Middle-Brazos Palo Pinto watershed, USGS hydrologic unit code 12060201 (Reference 2.4-205). These watersheds incorporate portions of Archer, Young, Jack, Stephens, Palo Pinto, Parker, Eastland, Erath, Hood, Somervell, Johnson, Bosque, Hill, McClennan, Limestone, and Falls counties.

Near the site, the Brazos River Channel is located in incised meanders formed by the river. These meanders may be the result of uplift of the area and sea level fluctuations after a mature meandering drainage pattern is attained. The meanders eroded through and are flanked by rock slopes confining the river within a relatively narrow channel. Immediately adjacent to the channel within the meanders is a narrow flood plain. Although accretion and erosion occur within the channel, as is typical of a meandering river, the well-defined meanders indicate that the channel location is closely confined. The geometry of the banks is governed closely by their location with respect to the meander pattern. The bank on the outside of a bend generally is steep; whereas, the bank on the inside of the bend usually has a gentler slope (Reference 2.5-201). HYDSV-04

The Texas Water Development Board (TWDB) lists 44 major reservoirs within the watershed of the Brazos River Basin (Reference 2.4-206). These reservoirs and their associated dams (Figure 2.4.1-206) are utilized for water supply, recreation, flood control, cooling, and power generation. For ~~this study~~the safety analyses, HYDSV-04 the most significant portions of the Brazos River basin are those between Possum Kingdom Lake and Lake Whitney, including Lake Granbury, as this area exhibits HYDSV-04 closely confined basin geometry and includes the highest concentration of major main stem reserviors. As shown on Figure 2.4.1-207 there are seven large manmade impoundments located within 150 stream mi of the DeCordova Bend Dam on Lake Granbury that could affect or be affected by plant operations. These impoundments include:

- Possum Kingdom Lake, on-channel, upstream reservoir located approximately 145 stream mi northwest of DeCordova Bend Dam, in Hydrologic Unit 12060201 (Figure 2.4.1-208).
- Lake Palo Pinto, off-channel, upstream reservoir located approximately 80 stream mi northwest of DeCordova Bend Dam, in Hydrologic Unit 12060201.
- Lake Mineral Wells, off-channel, upstream reservoir located approximately 70 stream mi northwest of DeCordova Bend Dam, in Hydrologic Unit 12060201.

- Lake Granbury, the primary cooling water source for CPNPP Units 3 and 4, on-channel reservoir located approximately 7 mi northeast of the CPNPP site, in Hydrologic Unit 12060201.
- SCR, off-channel reservoir located adjacent north and east of CPNPP Units 3 and 4, in Hydrologic Unit 12060202.
- Wheeler Branch Reservoir, off-channel reservoir located approximately 2 mi south of CPNPP Units 3 and 4, in Hydrologic Unit 12060202.
- Lake Whitney, on-channel, downstream reservoir located approximately 70 stream mi south of DeCordova Bend Dam in Hydrologic Unit 12060202.

Possum Kingdom Lake and Lake Granbury are operated by the Brazos River Authority (BRA), Lake Whitney by the USACE, Lake Palo Pinto by the Palo Pinto Water District No. 1, Lake Mineral Wells by the City of Mineral Wells, SCR by Luminant, and Wheeler Branch Reservoir by the Somervell County Water District. Table 2.4.1-203 provides information on dam and reservoir specifications for these impoundments.

The U.S. Army Corps of Engineers (USACE) maintains water flow rates on its website (Reference 2.4-207) for each day of the year for the major impoundments on the Brazos River, including Possum Kingdom Lake, Lake Granbury, and Lake Whitney.

Reservoir yields for the years 2000 and 2060 were obtained from the 2006 Brazos G Regional Water Plan (Reference 2.4-208). The firm yield is the greatest amount a reservoir could have supplied without shortage during a repeat of historical hydrologic conditions. Safe yield is defined as the amount of water that can be diverted from a reservoir during a repeat of the worst drought of record while still maintaining a reserve capacity equal to a 1-yr supply. Utilization of safe yield versus firm yield is a common practice in west Texas. Safe yield provides additional assurance of supply in an area where water resource alternatives are limited. Reservoir yields were limited to authorized diversions, and the period of record for the firm yield analyses was for the years 1940 through 1997.

For the dam failure analysis discussed in Subsection 2.4.4, 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 and the Paluxy River confluence were analyzed. These reservoirs were chosen for the dam failure analysis based on storage capacity and distance from the Brazos River and the Paluxy River confluence. Hubbard Creek Dam is located approximately 357 miles upstream of the Brazos River and Paluxy River confluence and was included in the dam failure analysis based on its distance from Morris Sheppard Dam and greater storage capacity (324,983 ac-ft), when compared to other upstream reservoirs in the region. HYDSV-05

According to the 2006 Brazos Region G Water Plan, most of the sites in the state that are readily amenable to reservoir development have already been utilized. Many other sites that are amenable to reservoir development have not been thoroughly developed as potential water supplies, even though they have been studied for many years. These projects have been mentioned in previous state water plans, but have not been developed due to permitting problems, environmental impacts, water quality, or cost considerations. Over the last 10 to 20 years, the development of major reservoirs has slowed considerably due to stringent permitting requirements and increased environmental awareness. For these reasons, any major reservoir should be considered only as a long-term solution for the development of the project. If the project is taken to fruition, it would most likely take more than 10 years. HYDSV-05

Seven potential upstream reservoir sites were evaluated in the 2006 Brazos Region G Water Plan (Reference 2.4-208). For the dam failure analysis, the volume, upstream distance from the Brazos River and Paluxy River confluence, and development potential of each proposed reservoir site were considered. All but one of these potential reservoirs, the South Bend Reservoir, were found to contain less storage than Possum Kingdom Lake and were excluded from the dam failure analysis.

The proposed South Bend Site, located approximately 251 miles upstream of the Brazos River and Paluxy River confluence, would store up to 771,604 ac-ft. This reservoir was not recommended as a water management strategy in the 2006 Region G Water Plan, which indicates implementation of the South Bend Reservoir would encounter difficult permitting constraints and would likely require significant treatment due to water quality concerns. Although the proposed South Bend Reservoir would be closer to the Brazos River and Paluxy River confluence and would impound a greater volume of water than the Hubbard Creek Reservoir and Possum Kingdom Lake, the site has not been recommended as a water management strategy for Region G and was not included in the dam failure analysis.

Potential reservoirs sites identified in the 2006 Llano Estacado (Region O) Water Plan (Reference 2.4-269) contain less storage than Possum Kingdom Lake and are at locations greater than 500 miles upstream from the Brazos River and Paluxy River confluence. Based on distance and storage capacity, these potential sites were not included in the dam failure analysis.

Based on information from the 2006 Brazos Region G (Reference 2.4-208) and 2006 Region H (Reference 2.4-270) water plans, there are no proposed main stem reservoirs downstream of Lake Whitney. Failure of downstream structures would reduce the effects of upstream dam failure and were not considered in the dam failure analysis. Similarly, failures of downstream off-channel structures were not considered.

2.4.2 Floods

CP COL 2.4(1) Replace the content of DCD Subsection 2.4.2 with the following.

2.4.2.1 Flood History

Floods in Texas typically are associated with thunderstorms during the summer and hurricanes and tropical storms in the late summer through early fall. (Reference 2.4-228) Historical flooding in the Brazos River watershed above the site has been a result of precipitation runoff. There are no known historical floods due to dam failures, surges, seiches, tsunamis, ice jams, or landslides. Dam failures are discussed in Subsection 2.4.4. Surge and seiches are discussed in Subsection 2.4.5. Tsunamis are discussed in Subsection 2.4.6. Ice effects are discussed in Subsection 2.4.7. Landslides are discussed in Subsection 2.4.9. The maximum recorded water surface elevation associated with floods of record for all rivers and streams in the vicinity are significantly lower than the Comanche Peak Nuclear Power Plant (CPNPP) Units 3 and 4 site grade as discussed below.

The greatest known flood of the Brazos River occurred in 1876 prior to any monitoring. Therefore, quantitative data for this event do not exist (Reference 2.4-214). The USGS gage (08091000) on the Brazos River nearest to the site is located near Glen Rose, Texas just upstream of the confluence with the Paluxy River. Although there are no flood control dams upstream of the gage on the Brazos River, the gage is subject to regulation by Morris Sheppard Dam, completed in 1941 and impounding Possum Kingdom Lake, and DeCordova Bend Dam, completed in 1969 and impounding Lake Granbury. (Reference 2.4-222) The gage drainage area is 25,818 sq mi. The contributing drainage area of the gage is 16,252 sq mi (Reference 2.4-224) and the gage location is shown in Figure 2.4.2-201.

The peak flow measurement period of record for the gage 08091000 is from 1923 to the present. The maximum recorded water surface elevation of 603.58 ft msl occurred on April 28, 1990 and corresponded to a discharge of 79,800 cfs. The discharge was exceeded in 1991, 1981, 1957, and 1935. However, the recorded water surface elevations were less than the flood elevation occurring in 1990. The maximum recorded discharge of 97,600 cfs occurred on May 18, 1935 (Reference 2.4-224). The annual peak stage and discharge measurements for the period of record are provided in Table 2.4.2-201. The datum for USGS gage (08091000) is reported in North American Datum 1927 (NAD27) and National Geodetic Vertical Datum of 1929 (NGVD29). HYDSV-02

The Paluxy River is a tributary of the Brazos River. A USGS gage (08091500) is located upstream of the confluence with the Squaw Creek tributary near Glen Rose, Texas. The gage drainage area is 410 sq mi (Reference 2.4-225) and the gage location is shown in Figure 2.4.2-201. The peak flow measurement period of record for the gage contains periodic measurements in 1908, 1918, and 1922 and is continuous from 1948 to the present. (Reference 2.4-220) The maximum recorded water surface elevation of 636.86 ft msl occurred on April 17, 1908 and corresponded to the maximum recorded discharge of 59,000 cfs (Reference 2.4-225). The annual peak stage and discharge measurements for the period of record are provided in Table 2.4.2-202. The datum for USGS gage (08091500) is reported in NAD27 and NGDV29. HYDSV-02

The USGS gage (08091750) closest to the site is located on Squaw Creek just below the SCR. The gage drainage area is 70.3 sq mi (Reference 2.4-226) and the gage location is shown in Figure 2.4.2-201. The peak flow measurement period of record for the gage is from 1973 to 2006. (Reference 2.4-220) The maximum recorded water surface elevation of 610.90 ft msl occurred on April 8, 1975 and corresponded to the maximum recorded discharge of 9030 cfs. (Reference 2.4-226) Squaw Creek Dam, impounding SCR, was completed in 1977. (Reference 2.4-222) Since completion of the Squaw Creek Dam, the maximum recorded water surface elevation of 610.85 ft msl occurred on June 13, 1989 and corresponded to the maximum recorded discharge of 8940 cfs. (Reference 2.4-220) The annual peak stage and discharge measurements for the period of record~~ed~~ are provided in Table 2.4.2-203. The datum for USGS gage (08091500) is reported in NAD27 and NGDV29. HYDSV-02

Prior to completion of the Squaw Creek Dam, a USGS gage (08091700) was located upstream of the site on the Panter Branch, a tributary of Squaw Creek. The gage drainage area is 7.82 sq mi and the gage location is shown in Figure 2.4.2-201. The peak flow measurement period of record for the gage is from 1966 to 1973. ~~A vertical datum is not provided for the gage.~~ The maximum recorded ~~stage of 21.88 ft~~water surface elevation of 904.88 ft msl occurred on September 16, 1972 and corresponded to the maximum recorded discharge of 3750 cfs. (Reference 2.4-220) The annual peak stage and discharge measurements for the period of record are provided in Table 2.4.2-204. The datum for USGS gage (08091700) is reported in NAD27 and NAVD88. HYDSV-02 HYDSV-02

2.4.2.2 Flood Design Considerations

The type of events evaluated to determine the worst potential flood include: HYDSV-14

- Probable maximum precipitation (PMP) on the total watershed and critical sub-watersheds, including seasonal variations and potential consequent dam failures, with a corresponding water surface elevation of 790.9 ft msl (discussed in Subsection 2.4.3).
- Dam failures, including a postulated domino-type failures of three upstream dams coincident with the Probable Maximum Flood (PMF), with a corresponding water surface level of 774.99 ft msl (discussed in Subsection 2.4.4).
- Two year coincident wind waves with a corresponding water surface level of 807.87 ft msl (discussed in Subsection 2.4.3).

~~The type of events evaluated to determine the worst potential flood include (1) probable maximum precipitation (PMP) on the total watershed and critical sub watersheds including seasonal variations and potential consequent dam failures, as discussed in Subsection 2.4.3, (2) dam failures, as discussed in Subsection 2.4.4, including a postulated domino-type failures of three upstream dams coincident with the Probable Maximum Flood (PMF), (3) local intense precipitation, and (4) two year coincident wind waves as discussed in Subsection 2.4.3.~~ HYDSV-14

Specific analysis of Brazos River flood levels resulting from ocean front surges, seiches, and tsunamis is not required because of the inland location and elevation characteristics of the CPNPP site. Additional details are provided in Subsections 2.4.5 and 2.4.6. Snowmelt and ice effect considerations are unnecessary because of the temperate zone location of CPNPP. Additional details are provided in Subsection 2.4.3 and Subsection 2.4.7. Flood waves from landslides into reservoirs required no specific analysis, in part because of the absence of major elevation relief. In addition, elevation characteristics of the vicinity relative to the associated water features, combined with limited slide volumes prohibit significant landslide induced flood waves. Additional details are provided in Subsection 2.4.9.

The maximum flood level at CPNPP Units 3 and 4 is elevation ~~788.9~~790.9 ft msl. HYDSV-14
This elevation would result from a PMP on the Squaw Creek watershed, as described in Subsection 2.4.3. Coincident wind waves would create maximum waves of ~~4.56~~16.97 ft ~~(trough to crest),~~ resulting in a ~~maximum~~design basis flood HYDSV-14
elevation of ~~793.46~~ 807.87 ft msl. CPNPP Units 3 and 4 safety-related plant elevation is 822 ft msl, providing more than ~~28~~14 ft of freeboard under the worst potential flood considerations.

2.4.2.3 Effects of Local Intense Precipitation

CPNPP Units 3 and 4 drainage system was evaluated for the PMP on the local area. The site is graded such that overall runoff will drain away from safety-related structures directly to the SCR. The PMP flood analysis assumes that storm drainage structures within the local area are non-functioning. Computed water surface elevations in the vicinity of safety-related structures are below site grade elevation of 822 ft msl. The site grading and drainage plan is shown in Figure 2.4.2- 202.

The local intense PMP is defined by Hydrometeorological Report No. 51 (HMR 51) and No. 52 (HMR 52). PMP values for durations from 6-hr. to 72-hr. are determined using the procedures as described in HMR No. 51 for areas of 10-sq mi (Reference 2.4-218). Using the CPNPP location, the rainfall depth is read from the HMR 51 PMP charts for each duration. The 1-sq mi PMP values for durations of 1-hour and less are determined using the procedures as described in HMR 52. (Reference 2.4-219) Using the CPNPP location, the rainfall depth for each duration is read from the HMR 52 1-sq mi PMP charts. A smooth curve is fitted to the points. The derived PMP curve is detailed in Table 2.4.2-205. The corresponding PMP depth duration curve is shown in Figure 2.4.2-203.

$Q = C \cdot i \cdot A$ (Reference 2.4-227) (Equation 3)

where:

Q = Runoff (cfs)

C = Unitless coefficient of runoff

i = Intensity (in/hr)

A = Drainage area (ac)

No runoff losses were assumed. Therefore, the runoff coefficient was assumed equal to one. The weir equation is used to determine the PMF elevation for the peak runoff rate from the sub basins. A tail water elevation at ~~788.9~~790.9 ft msl from a PMF at the SCR was considered for the local site analysis. HYDSV-06 HYDSV-07

The equation for weir is given by the equation:

$Q = C_d \cdot L \cdot HW_r^{1.5}$ (Reference 2.4-223) (Equation 4)

where:

Q = runoff (cfs)

C_d= Overtopping discharge coefficient (Reference 2.4-223)

L = Crest length of overflow section (ft)

HW_r= Head water elevation for the weir (ft)

Site drainage area details are tabulated in Table 2.4.2-207. Resulting PMP water surface elevation at the points of discharge from the local site analysis are shown in Table 2.4.2-208. Drainage areas 1, 2, and 3 result in a maximum water surface elevation of 820.83 ft msl at the point of discharge W1. CPNPP Units 3 and 4 safety-related structures are located above the effects of local intense precipitation at plant elevation 822 ft msl.

Due to the temperate climate and relatively light snowfall, significant icing is not expected. Based on the site layout and grading, any potential ice accumulation on site facilities is not expected to affect flooding conditions or damage safety-related facilities. Ice effects are discussed in Subsection 2.4.7.

2.4.3 Probable Maximum Flood

CP COL 2.4(1) Replace the content of DCD Subsection 2.4.3 with the following.

The guidance in Appendix A of the U.S. Nuclear Regulatory Commission (NRC) Regulatory Guide 1.59 was followed in determining the PMF by applying the guidance of ANSI/ANS-2.8-1992 (Reference 2.4-229). ANSI/ANS-2.8-1992 was issued to supersede ANSI N170-1976, which is referred to by Regulatory Guide 1.59. ANSI/ANS-2.8-1992 is the latest available standard.

The PMF was determined for the Squaw Creek watershed and routed through the SCR to determine a water surface elevation of ~~788.9~~790.9 ft msl. The PMF for the Paluxy River watershed at the confluence with the Brazos River was also examined. The PMF for the Paluxy River and the Squaw Creek watersheds was combined with the Brazos River dam failure flood flow to determine any backwater effects that may affect the site. The Brazos River dam failure flood flow is described in Subsection 2.4.4 and includes the PMF for the Brazos River. The resulting water surface elevation downstream of the Squaw Creek Dam is ~~755.08~~755.21 ft msl. HYDSV-06 HYDSV-07

The CPNPP Units 3 and 4 safety-related facilities are located at elevation 822 ft msl. Therefore, PMF on rivers and streams does not present any potential hazards for CPNPP Units 3 and 4 safety-related facilities.

2.4.3.1 Probable Maximum Precipitation

The PMP is defined by HMR 51 (Reference 2.4-218) and HMR 52 (Reference 2.4-219). HMR 53 (Reference 2.4-230) may be used to derive seasonal estimates of the PMP. The PMP was determined for the Squaw Creek and the Paluxy River watersheds. Using the location of the watersheds, HMR 51 PMP charts are used to determine generalized estimates of the all-season PMP for drainage areas from 10 to 20,000 sq mi for durations from 6 to 72 hr. The resulting depth-area-duration (DAD) values are shown in Table 2.4.3-201.

HMR 52 is used to determine the aerial distribution of PMP estimates derived from HMR 51. The recommended elliptical isohyetal pattern from HMR 52, shown in Figure 2.4.3-201, is used for the watersheds. The watershed model, combining both watersheds, contains 4 subbasins and is shown in Figure 2.4.3-202. The watershed model is discussed in detail in Subsection 2.4.3.3.

HMR 52 computer software (Reference 2.4-231), developed by USACE, is used to determine the optimum storm size and orientation to produce the greatest PMP over the watersheds using the HMR 51 derived DAD table. Several storm centers were examined for each watershed to determine the critical storm center.

In accordance with Appendix A of Regulatory Guide 1.59, the 72-hr PMP storm is combined with an antecedent storm equal to 40 percent of the PMP. Therefore, the complete sequential storm considered includes a 3-day, 40 percent PMP event followed by a 3-day dry period, which is followed by the 3-day full PMP event. Critical temporal distribution was determined by runoff analysis. Multiple temporal distributions were examined, including one-third, center, two-thirds, and end peaking arrangements.

For the Squaw Creek watershed, the critical storm center was found to be near the watershed centroid, identified as point SC X in Figure 2.4.3-202. A storm center at SC2 results in the maximum PMP for the Squaw Creek watershed. The storm center SC X results in a higher runoff and hence SC X is considered to be the critical storm center for the Squaw Creek watershed. The critical storm area was found to be 700 sq mi, corresponding to isohyet H in Figure 2.4.3-201. The critical storm orientation was found to be ~~160~~145 degrees. HYDSV-06 HYDSV-07

The critical 72-hr storm PMP rainfall total is 38.46 in for the Squaw Creek watershed. The standard HMR 52 temporal arrangement of 6-hr precipitation increments is provided in Table 2.4.3-202. The critical temporal distribution was determined by runoff analysis to be an ~~end~~two-thirds peaking arrangement for the Squaw Creek watershed. The hourly end temporal distribution of the 72-hr PMP rainfall for each of the 4 subbasins is provided in Table 2.4.3-203. The corresponding hyetograph is shown in Figure 2.4.3-203. HYDSV-06 HYDSV-07

For the Paluxy River watershed, the critical storm center was found to be near the watershed centroid, identified as point PR Y in Figure 2.4.3-202. The critical storm area was found to be 450 sq mi, corresponding to isohyet G in Figure 2.4.3-201. The critical storm orientation was found to be 172 degrees.

The critical 72-hr storm PMP rainfall total is 35.08 in for the Paluxy River watershed. The standard HMR 52 temporal arrangement of 6-hr precipitation increments is provided in Table 2.4.3-204. The critical temporal distribution was determined by runoff analysis to be a ~~center~~one-third peaking arrangement for the Paluxy River watershed. The hourly temporal distribution of the 72-hr PMP rainfall for each of the 4 subbasins is provided in Table 2.4.3-205. The corresponding hyetograph is shown in Figure 2.4.3-204. HYDSV-06 HYDSV-07

The watersheds do not occur in the orographic regions identified by HMR 51 and HMR 52. Additionally, the area does not contain significant changes in elevation that would require modification to the PMP. Therefore, orographic effects are not considered.

According to HMR 53, the all-season PMP estimates are associated with the warmer summer months. HMR 53 winter precipitation estimates are greatly reduced compared to the all-season PMP estimates. Additionally, snowmelt does not contribute significantly to river floods anywhere in the state (Reference 2.4-214). Therefore, snowmelt is not considered to be a factor in modeling the PMF event.

The potential dam failures consider coincident PMF flows for the Brazos River watershed. The PMP for the Brazos River was not determined. The approach detailed in Appendix B of Regulatory Guide 1.59 was used to derive the peak PMF flow directly. Potential dam failures are discussed in Subsection 2.4.4.

2.4.3.2 Precipitation Losses

Precipitation losses are based on the existing evaluation for CPNPP Units 1 and 2. According to CPNPP Units 1 and 2 FSAR, an initial loss of 0.5 in and a conservative infiltration rate of 0.1 in/hr were determined from USACE records of the Paluxy River watershed (Reference 2.4-214). The recorded Paluxy watershed losses are provided in Table 2.4.3-206.

For evaluation of CPNPP Units 3 and 4, no initial losses were assumed, indicating saturated antecedent moisture conditions at the onset of the antecedent storm. This assumption is more conservative than the guidance provided in ANSI/ ANS-2.8-1992. A constant loss rate of 0.1 in/hr was used in the runoff model. The runoff model is described in Subsection 2.4.3.3.

2.4.3.3 Runoff and Stream Course Models

The runoff and stream course models are based on an existing study for the SCR. The watershed and subbasins are shown in Figure 2.4.3-202. Basin 1 was further subdivided into three subbasins – 1a, 1b, and 1c. Basin 1a represents the drainage area above the SCR, Basin 1b represents the contributing area adjacent to the SCR, and Basin 1c represents the SCR. Drainage areas for each subbasin are provided in Table 2.4.3-207.

Based on USGS quadrangles, the topography of the Squaw Creek watershed generally slopes to the stream course running through the middle of the watershed. The stream course slopes to the southeast from about 1100 ft msl to a low point of 650 ft msl. However, the SCR has inundated elevations below 775 ft msl. The highest point in the basin is the plateau peak of the geographic feature Comanche Peak at elevation 1230 ft msl (Reference 2.4-237).

The Paluxy River basin generally slopes to the river course running through the middle of the watershed. The river course slopes to the southeast from about 1450 ft msl to a low point of 570 ft msl at the confluence with the Brazos River. The highest point in the basin is elevation 1490 ft msl (Reference 2.4-237).

The USACE HEC-HMS, Version 3.1.0 (Reference 2.4-232), modeling software was used for rainfall runoff and routing calculations. The HEC-HMS model watershed routing layout is shown in Figure 2.4.3-205. The unit hydrographs for each basin were based on the existing study using the synthetic Snyder's Unit Hydrograph. Snyder's method was used for the CPNPP Units 1 and 2 unit hydrograph development (Reference 2.4-214), and is applicable under PMF conditions. The Snyder's method provided reasonable estimates for peak direct runoff rate at the CPNPP location and is acceptable in determining the peak direct runoff rate for the CPNPP Units 3 and 4. To represent a conservative approach, the basin characteristics resulting in higher runoff at the CPNPP Units 3 and 4 HYDSV-06 HYDSV-07 were used in the runoff model. Lag times were developed based on the characteristics of each basin. The basin characteristics and lag times are provided in Table 2.4.3-207. HYDSV-06 HYDSV-07

Base flow was determined using the average monthly flow of the ~~8.39~~9.7 cfs from USGS Gage 08091750. The lowest of these monthly flows was used as the base flow. Because the basin areas are different from gage area (~~74~~70.3 sq mi), the base flow was adjusted on the basis of ratio of basin drainage area to the gage area. The adjusted baseflow was applied to the model as a constant rate and is provided in Table 2.4.3-207. HYDSV-06 HYDSV-07 HYDSV-06 HYDSV-07

The Muskingum-Cunge 8-point cross section method was used for the river routing reaches within the HEC-HMS model. Channel slope, length, and cross section data were developed using USGS quadrangles. Manning's roughness coefficients were based on the existing study and compared with accepted published tables by Chow (Reference 2.4-233). Squaw Creek Manning's roughness coefficients range from 0.06 for the channel to 0.09 for the overbanks. The Paluxy River Manning's roughness coefficients range from 0.045 for the channel to 0.07 for the overbanks. To account for variability and uncertainty, the Manning's roughness coefficient of 0.15 has been used within HEC-HMS and HEC-RAS.

SCR is the only reservoir within the Paluxy River and Squaw Creek watersheds. The storage-elevation for the SCR was obtained from following two sources:

- The storage-elevation data for elevation 775 ft msl and below have been obtained from 1997 TWDB Volumetric Survey for SCR. (Reference 2.4-212)

- The storage-elevation data for elevations above 775 ft msl have been obtained from and the Operation and Maintenance Procedures for Squaw Creek Dam prepared by Freese and Nichols in 1997.

The storage-discharge curve for service and emergency spillways has been obtained from the Operation and Maintenance Procedure. The storage discharge relationship was linearly extrapolated to account for discharge from elevation 791 ft msl to 795 ft. msl. The reservoir rating curve is presented in Figure 2.4.3-206. HYDSV-06 HYDSV-07

Methods adopted to account for nonlinear basin response at high rainfall rates include no initial losses as discussed in Subsection 2.4.3.2 and the use of 40 percent PMP antecedent rainfall as discussed in Subsection 2.4.3.1. Snowmelt is not considered to be a factor in modeling the PMF event, as described in Subsection 2.4.3.1.

Because of large magnitude flows and potential backwater effects from flooding of the Paluxy River and the Brazos River, a standard step method, unsteady-flow hydraulic analysis was also performed to assess the resulting water surface elevation downstream of Squaw Creek Dam. The USACE HEC-RAS, Version 3.1.3 (Reference 2.4-234), modeling software was used to route the flood hydrographs obtained from the HEC-HMS model.

The Paluxy River reach through Basin 3 and the Squaw Creek reach through Basin 2 were included in the HEC-RAS model. Cross sections were estimated using the existing study and USGS quadrangles. Cross section interpolations were performed as necessary to provide a stabilized HEC-RAS model.

The Basin 1 hydrograph routed through the SCR and the Paluxy River Basin 3 hydrograph from the HEC-HMS analysis were used as upstream boundary input. The Basin 2 and Basin 4 hydrographs from the HEC-HMS analysis were included as lateral inflows. A constant stage hydrograph, due to the peak dam failure flow described in Subsection 2.4.4, was used as the boundary condition at the downstream end of the Paluxy River. This is a bounding condition including the conservative assumptions that multiple PMF scenarios occur coincidentally and that the peak domino-type dam failure effects are maintained at the confluence throughout the duration of the PMF. A computation interval of 5 min was used in the HEC-RAS model.

2.4.3.4 Probable Maximum Flood Flow

Applying the precipitation, described in Subsection 2.4.3.1, with the precipitation losses, described in Subsection 2.4.3.2, to the runoff model, described in Subsection 2.4.3.3, the SCR peak PMF inflow was determined to be ~~130,000~~221,000 cfs. The routed peak discharge from the SCR is ~~98,000~~148,000 cfs. The resulting inflow and outflow hydrographs are shown in Figure 2.4.3-207. Position of the storm and temporal distribution of the PMP is discussed in Subsection 2.4.3.1. Discussion of dam failure is provided in Subsection 2.4.4. There are no significant current or planned upstream structures. No credit is taken for the lowering of flood levels at the site due to downstream dam failure. HYDSV-06 HYDSV-07

The maximum backwater flow on the downstream end of the Squaw Creek Dam is ~~980,000~~88,130 cfs. The associated backwater analysis does not provide the controlling PMF water surface elevation at the site. HYDSV-06 HYDSV-07

2.4.3.5 Water Level Determinations

The PMF runoff, routed through the SCR, results in a peak water surface elevation of ~~788.9~~790.9 ft msl at ~~the site~~CPNPP Units 3 and 4. The water surface elevation is determined using the HEC-HMS runoff and routing model as described in Subsection 2.4.3.3. The hydrograph for the SCR is provided in Figure 2.4.3-208. HYDSV-06 HYDSV-07

The standard step, unsteady-flow analysis for the Squaw Creek and the Paluxy River watersheds, resulted in a water surface elevation of ~~755.08~~775.21 ft msl on the downstream side of the SCR. The HEC-RAS model described in Subsection 2.4.3.3 was used to translate runoff to the water surface elevation. The resulting HYDSV-06 HYDSV-07 elevation of ~~775.08~~775.21 ft msl is below the elevation of CPNPP Units 3 and 4 safety-related facilities and presents no hazard. In an unlikely event of achieving the water surface elevation described above, possible headcutting on the downstream slope of Squaw Creek could result in failure of the Squaw Creek Dam. However, failure would lower the water surface elevation of the SCR. HYDSV-06 HYDSV-07

2.4.3.6 Coincident Wind Wave Activity

Fetch length was estimated based on USGS Quadrangles. The critical fetch length was found to be ~~2.66~~2.67 mi originating from the east for Fetch 3 as shown in Figure 2.4.3-209. CPNPP is protected from wind wave activity from the west and south by the local topography. Wave height, setup, and runup are estimated using USACE "Coastal Engineering Manual, EM 1110-2-1100" guidance (Reference 2.4-235). HYDSV-06 HYDSV-07

A two-year annual extreme mile wind speed of 50 mph was estimated based on ANSI/ANS-2.8-1992 as shown in Figure 2.4.3-210. The two-year annual extreme mile wind speed was adjusted for duration, based on the fetch length, level, over land or over water, and stability.The critical duration was found to be about 53 min. This corresponds to an adjusted wind speed of ~~50.22~~49.91 mph. HYDSV-06 HYDSV-07

Significant wave height (average height of the maximum 33-1/3 percent of waves) is estimated to be ~~2.72~~2.76 ft, crest to trough. The maximum wave height (average height of the maximum 1 percent of waves) is estimated to be ~~4.56~~4.59 ft., crest to trough. The corresponding wave period is ~~2.2~~2.6 sec. HYDSV-06 HYDSV-07

Slopes of 10:1 and 3:1, horizontal to vertical, in the vicinity of the CPNPP were used to determine the wave setup and runup. Additionally, wind wave activity at the vertical retaining wall was also examined. The runup includes wave setup. Runup for the 10:1 slopes was estimated to be ~~1.05~~2.85 ft. Runup for the 3:1 slopes was estimated to be ~~3.38~~6.98 ft. Runup at the vertical retaining wall on the north side of CPNPP Units 3 and 4 was estimated to be ~~10.50~~16.90 ft. HYDSV-06 HYDSV-07

Wind setup was estimated using additional USACE Hydrologic Engineering Requirements for Reservoirs, EM 1110-2-1420 guidance (Reference 2.4-236). The maximum wind setup was estimated to be 0.07 ft. The maximum total wind wave activity is estimated to be ~~20.31~~16.97 ft and occurs at the vertical retaining wall. The PMF and maximum coincident wind wave activity results in a flood elevation of ~~809.28~~807.87 ft msl.The top elevation of the retaining wall is 805 ft msl. The CPNPP Units 3 and 4 safety-related structures are located at elevation 822 ft msl and are unaffected by flood conditions and coincident wind wave activity. In the event of Squaw Creek Dam failure, the determined fetch length would not be increased. HYDSV-06 HYDSV-07 HYDSV-06 HYDSV-07

2.4.4 Potential Dam Failures

CP COL 2.4(1) Replace the content of DCD Subsection 2.4.4 with the following.

There are no surface water impoundments other than small farm ponds that could impact the SCR. The small farm ponds have negligible storage capacity and a breach would have no measurable effect. Failure of downstream dams, including Squaw Creek Dam, would not affect the CPNPP Units 3 and 4.

~~The critical dam failure event is the assumed domino-type failure of the Hubbard Creek Dam, the Morris Sheppard Dam and the DeCordova Bend Dam coincident with the PMF.~~There are currently three reservoirs located on the main stem of the Brazos River: Possum Kingdom Lake, Lake Granbury, and Lake Whitney. Each of these reservoirs is within 150 river miles of the CPNPP site and most of the main stem Brazos River reservoir storage is concentrated along this reach. Because the site is located off-channel on a tributary of the Brazos River, the most conservative approach for the critical dam failure event would be for this reach of the Brazos River to flood by way of domino-type dam failure of upstream dams, and for flood waters to back up from the Brazos River and Paluxy River confluence onto the site by way of the Squaw Creek catchment. For the dam failure analysis, the peak flow of the probable maximum flood (PMF) coincident with assumed hydrologic domino-type dam failure of three upstream dams were analyzed at the Brazos River and the Paluxy River confluence. Morris Sheppard Dam and De Cordova Bend Dam are located within the portion of the Brazos River Basin identified as most significant for the dam failure analysis; however, for conservatism, the failure of Hubbard Creek Dam, which impounds Hubbard Creek Reservoir, was also used in the dam failure analysis. Hubbard Creek Dam is located approximately 357 miles upstream of Morris Sheppard Dam and was chosen for the dam failure analysis based on its distance from Morris Sheppard Dam and greater storage capacity when compared to other upstream reservoirs in the region. Domino-type failures are included coincident with PMF flows and transposed downstream without any attenuation. Thus, the closely confined basin geometry of this reach and the concentration of major reservoirs were used as the basis for determining this portion of the basin as the most significant for the dam failure analysis. HYDSV-04

The guidance in Appendix B of NRC Regulatory Guide 1.59 is used as an alternative approach to determine the coincident PMF. The Brazos River watershed, locations for the three dams and CPNPP Units 3 and 4 are identified in Figure 2.4.4-201. There are no safety-related structures that could be affected by flooding due to dam failures.

2.4.4.1 Dam Failure Permutations

SCR is located immediately downstream of the site. Squaw Creek is a tributary of the Paluxy River, which is a tributary of the Brazos River. Hubbard Creek Dam is located upstream of the site on a tributary of the Brazos River. Morris Sheppard

As shown in Figure 2.4.4-201, Lake Palo Pinto is located on a tributary of the Brazos River between Morris Sheppard and DeCordova dams. Lake Palo Pinto contains a significantly smaller volume of water than Hubbard Creek Reservoir. Therefore, failure of Palo Pinto Dam in combination with the main stream dams would produce less flooding than inclusion of Hubbard Creek Dam. Because of the relative locations, flooding from simultaneous failure of dams would not combine to create more severe flooding than that discussed.

The volume of water, distance from the Brazos River and Paluxy River confluence, and the development potential of proposed reservoir sites were considered for the dam failure analyses. All but one of these potential reservoirs, the South Bend Reservoir, was found to contain less storage than Possum Kingdom Lake and were excluded from the dam failure analyses. The proposed South Bend Reservoir was not recommended as a water management strategy in the 2006 Brazos River Region G Water Plan (Reference 2.4-208), and therefore, was not included in the dam failure analyses. Also, there are no proposed main stem reservoirs downstream of Lake Whitney. Because of the relative locations and storage volume, flooding from simultaneous failure of dams at potential reservoir sites would not combine to create a more severe flooding than that discussed (see Subsection 2.4.1.2). HYDSV-09

There are no safety-related facilities that could be affected by loss of water supply due to dam failure or water supply blockages due to sediment deposition or erosion during dam failure induced flooding. See Subsection 2.4.11. Landslide potential is addressed in Subsection 2.4.9. There are no safety-related structures that could be affected by waterborne objects. There are no on-site water control or storage structures located above site grade that may induce flooding.

2.4.4.2 Unsteady Flow Analysis of Potential Dam Failures

The methods identified are standard industry methods applied to artificially large floods. The approach described above is conservative and utilizes conservative coefficients resulting in a bounding estimate for dam failure considerations. Therefore, a full unsteady flow analysis to determine dam breach flows and resulting water surface elevations with greater certainty is determined to be unnecessary. Downstream reservoirs have no affect on the results of this analysis. Domino-type failures are included coincident with PMF flows and transposed downstream without any attenuation as discussed above. As discussed below the resulting dam failure flood wave has no effect at the site.

2.4.4.3 Water Level at Plant Site

The potential backwater effect from flooding on the Brazos River is examined based on the assumed hydrologic domino-type dam failures coincident with the PMF. As described above, the assumed hydrologic domino-type dam failures of the Hubbard Creek Dam, the Morris Sheppard Dam, and the DeCordova Bend Dam coincident with the PMF, is transposed to the confluence of the Paluxy River and the Brazos River without any attenuation. Squaw Creek is a tributary of the

Paluxy River. Utilizing FlowMaster computer software (Reference 2.4-241), the Manning's friction method formula is used to determine the water surface elevation at the confluence.

The confluence cross section is determined based on USGS 7.5 minute topographic quadrangles containing 10 ft contour intervals. The bank full elevation of the Brazos River at the confluence is approximately elevation 560 ft msl. (Reference 2.4-214) The confluence cross section stations and elevations in ft msl are shown in Figure 2.4.4-202.

A Manning's roughness coefficient of n = 0.10 is estimated for the Brazos River channel based on published tables by Chow. (Reference 2.4-233) To account for variability and uncertainty of the Brazos River channel on the downstream side of the DeCordova Dam, sensitivity analyses were performed for Manning's roughness coefficient, channel geometry and channel slope.

The resulting maximum water surface elevation at the confluence of Brazos River and Paluxy River cross section is ~~775.04~~774.99 ft msl for the total transposed flow of 6.7 million cfs as shown in Figure 2.4.4-203. CPNPP Units 3 and 4 safety-related facilities are located at elevation 822 ft msl, providing almost 47 ft of freeboard. Additionally, the resulting water surface elevation is below the Squaw Creek Dam crest elevation of 796 ft. Therefore, coincident wind wave activity results would be equivalent to the wind wave activity for SCR (See Subsection 2.4.3.6). In the unlikely event of achieving the water surface elevation described above, possible headcutting on the downstream slope of Squaw Creek Dam could result in failure of the Squaw Creek Dam. However, failure would lower the water surface elevation of SCR. In the event of Squaw Creek Dam failure the fetch length determined by the wind wave activity in Subsection 2.4.3.6 would not be increased. HYDSV-09

2.4.5 Probable Maximum Surge and Seiche Flooding

CP COL 2.4(1) Replace the content of DCD Subsection 2.4.5 with the following.

According to the NRC Regulatory Guide 1.59, “Design Basis Floods for Nuclear Power Plants,” probable maximum surge and seiche flooding is considered based on a probable maximum hurricane (PMH), probable maximum windstorm (PMWS), or moving squall line. (Reference 2.4-229) The region of occurrence for a PMH is along U.S. coastline areas. For a PMWS, the region of occurrence is along coastline areas and large bodies of water such as the Great Lakes. A moving squall is considered for the Great Lakes region.

According to USACE EM 1110-2-1100 (Reference 2.4-235) guidelines, meteorological wind systems generated by thunderstorms and frontal squall lines can generate waves up to 16.4 ft high for inland waters. Additionally, mesoscale convective complex wind systems affecting inland waters are fetch-limited and based on wind speeds of up to about 66 fps or 45 mph. Similar wind speeds are used to determine the coincident wind-generated wave activity discussed in Subsection 2.4.3. The coincident wind wave activity, including wave setup, results in maximum runup of 16.9 ft. The maximum wind setup is estimated to be 0.07 ft. Therefore, the total water surface elevation increase due to wind wave activity is estimated to be 16.97 ft. The resulting PMF coincident with wind wave activity elevation is 807.87 ft msl. HYDSV-11

The USACE guideline procedure for geologic hazard evaluations considers seiche waves greater than 7 ft to be rare. (Reference 2.4-242) The seiche hazard can be screened out for sites located more than 7 ft above the adjacent water body.

CPNPP Units 3 and 4 are located approximately 275 mi inland from the Gulf of Mexico. CPNPP Units 3 and 4 safety-related facilities are located at the plant grade level elevation of 822 ft msl. A surge due to a PMH event would not cause flooding at the site. HYDSV-03

SCR does not connect directly with any of the water bodies considered for such meteorological events associated with surge and seiche flooding. Because of the inland location and elevation characteristics, CPNPP Units 3 and 4 safety-related facilities are not at risk from surge and seiche flooding. Resonance wave phenomena including oscillations of waves at natural periodicity, lake reflection, and harbor resonance are traditionally characteristics of harbors, estuaries, and large lakes and not associated with river settings. Any effects on ~~the Squaw Creek~~SCR produced by similar phenomena would not affect CPNPP Units 3 and 4. CTS-00817

Seismic-induced waves are not plausible for the SCR. Subsection 2.5.3 indicates there are no capable faults, and there is no potential for non-tectonic fault rapture within the 25 mi radius of the CPNPP Units 3 and 4. Additionally, there is no potential for tectonic or non-tectonic deformation within the 5 mi radius of the HYDSV-10

CPNPP Units 3 and 4. The geologic and seismic characteristics for the CPNPP Units 3 and 4 are described in Section 2.5. HYDSV-10

Landslide-induced waves are not plausible for the SCR. Slope stability within the immediate area of the CPNPP Units 3 and 4 is discussed in Subsection 2.5.5.

2.4.6 Probable Maximum Tsunami Hazards

CP COL 2.4(1) Replace the content of DCD Subsection 2.4.6 with the following.

Tsunami risk in the Gulf Coast region, primarily the Caribbean, has been studied to some degree, but no specific hazard maps have been developed for the Gulf Coast at this time. The USACE has developed a general tsunami risk map (Reference 2.4-242), as shown in Figure 2.4.6-201. The Gulf Coast is located in Zone 1, which corresponds to a wave height of 5 ft.

According to the National Oceanic and Atmospheric Administration's tsunami database (Reference 2.4-243), the maximum recorded tsunami wave height along the Gulf Coast or East Coast is about 20 ft. This height was recorded at Daytona Beach, Florida, on July 3, 1992. The database notes that the wave was probably meteorologically induced.

According to a recent USGS study (Reference 2.4-244), very little is known about a landslide-generated tsunami threat from the Mexican coast. Tsunamis generated by earthquakes do not appear to impact the Gulf of Mexico coast. CPNPP Units 3 and 4 are located approximately 275 mi inland from the Gulf Coast. CPNPP Units 3 and 4 safety-related facilities are located at elevation 822 ft msl. Because of their inland location and elevation, CPNPP Units 3 and 4 safety-related facilities would not be at risk from tsunami flooding.

Landslide-induced waves are not plausible for SCR. As discussed in Subsection 2.5.5, the slope stability analysis indicates stable permanent slopes, and therefore hill slope failure-induced waves are not plausible for SCR. HYDSV-12 HYDSV-13

Seismic-induced waves are not plausible for SCR. Subsection 2.5.3 states there are no capable faults and there is no potential for non-tectonic fault rapture within the 25 mi radius of the CPNPP Units 3 and 4. Additionally, there is no potential for tectonic or non-tectonic deformation within the 5 mi radius of the CPNPP Units 3 and 4. The geologic and seismic characteristics for the CPNPP Units 3 and 4 are described in Section 2.5.

2.4.7 Ice Effects

CP COL 2.4(1) Replace the content of DCD Subsection 2.4.7 with the following.

According to the EPA STOrage and RETrieval (STORET) database, two gaging stations located on the SCR and its tributaries recorded water temperatures for different periods between 1973 and 1985. The lowest recorded water temperatures range from 41.9°F to 50°F. The lowest recordings, 41.9°F, occurred on February 10, 1982 at station 11555, Squaw Creek and State Highway 144 (SH 144), Northeast of Glen Rose. (Reference 2.4-245)

Gaging station 11856 is located on Brazos River and gaging station 11976 is located on Paluxy River. The gaging station 11856 on Brazos River at U.S. Highway 67 (US 67) recorded water temperatures from 1968 to 1998. The lowest recorded water temperature at this station was 39.02°F. (Reference 2.4-245) The gaging station 11976 on Paluxy River in City Park recorded water temperatures from 1973 to 1996. The lowest recorded water temperature at this station was 39.2°F. (Reference 2.4-245) This data suggests that Squaw Creek water temperatures generally remain above the freezing point. The recordings are summarized in Table 2.4.7-201.

According to the USACE, ice jams occur in 36 states, primarily in the northern tier of the United States. (Reference 2.4-246) (Figure 2.4.7-201) Texas is not included in this coverage. USACE Cold Regions Research and Engineering Laboratory historical ice jam database (Reference 2.4-247) indicates no ice jams for Squaw Creek. However, the USACE ice jam database reports that Brazos River was obstructed by rough ice at Rainbow near Glen Rose, Texas, on January 22-23 and January 25-28, 1940, with flood stage of 20 ft. (Reference 2.4-247)

CPNPP Units 3 and 4 safety-related facilities are located at elevation 822 ft msl. The SCR spillway elevation is 775 ft msl (Reference 2.4-214). The maximum water surface elevation during a probable maximum flood event is at ~~788.9~~790.9 ft msl, which is more than 30 ft below the CPNPP Units 3 and 4 safety-related facilities. The possibility of inundating CPNPP Units 3 and 4 safety-related facilities due to an ice jam is remote. HYDSV-14

Meteorological records from the Southern Regional Climate Center (SRCC) were examined for areas in the vicinity of CPNPP Units 3 and 4. Records indicate that December and January have the coldest temperatures. For the available period of record from 1971 to 2000, the climate station at Dallas/Fort Worth has a recorded monthly average minimum temperature of 34°F, occurring in January. (Reference 2.4-248)

According to the USACE, frazil ice forms in supercooled turbulent water in rivers and lakes. (Reference 2.4-246) Anchor ice is defined as frazil ice attached to the river bottom, irrespective of the nature of its formation. The potential for freezing (i.e., frazil or anchor ice) and subsequent ice jams on the Squaw Creek and Brazos River is remote. Additionally, sustained periods of subfreezing water temperatures are not characteristic of the region. The climate and operation of SCR prevent any significant icing on the Squaw Creek. There are no safety related facilities that could be affected by ice induced low flow.

According to U.S. Army Corps of Engineers methods (Reference 2.4-269), the maximum potential ice thickness is a function of accumulated freezing-degree days (AFDD). The average maximum AFDD for CPNPP Units 3 and 4 is approximately 100 days (Reference 2.4-270). The resulting maximum potential ice thickness is 7 in. There are no safety-related facilities that could be affected by ice-induced low flow at CPNPP Units 3 and 4. The freezing protection for the essential (sometimes called emergency) service water system (ESWS) four wet mechanical cooling towers is described in Subsection 9.2.1.3. The freezing protection for the ESW Pump House Ventilation System is described in Subsection 9.4.5.2.6. HYDSV-15

The USACE historical database of ice jams was reviewed for the region. See Subsection 2.4.7 for additional discussion. Due to the climate in the region, ice effects are not a concern for low water considerations.

2.4.11.4 Future Controls

According to the FSAR for Comanche Peak Steam Electric Station Units 1 and 2, an initial study by the Brazos River Authority identified three possible sites between Possum Kingdom Reservoir and Lake Granbury for potential control structures. Additionally, there is a possible site between DeCordova Bend Dam and Whitney Dam for a control structure. Issuance by the Texas Water Rights Commission of the permit to build and operate SCR precludes any significant development and control upstream in the Squaw Creek watershed. (Reference 2.4-214) Although the development of future controls on the Brazos River is possible, there are no safety-related facilities that could be affected.

2.4.11.5 Plant Requirements

Makeup water ~~to the cooling water system flow~~ is supplied by the intake as described in Subsection 2.4.1.2.3.2. The intake structure includes necessary intake screens, pumps, etc. to convey the makeup water to the cooling water system flow. Intake screen locations consider the Lake Granbury minimum level. There are no safety-related plant requirements provided by Lake Granbury. CTS-00593

The maximum expected Lake Granbury intake flow rate is approximately 65,400 gpm for the CPNPP Units 3 and 4. The maximum expected Lake Granbury intake flow includes a circulating water system (CWS) Cooling Tower makeup flow rate of 31,200 gpm per unit for Units 3 and 4, an ESWS Cooling Tower makeup flow rate of 274 gpm per unit for Units 3 and 4 and miscellaneous plant use such as make up water flow to raw water storage tanks. The makeup flows to both CWS and ESWS Cooling Towers are essentially continuous. The flows are normally controlled with basin water levels by on/off operation of CWS Cooling Tower makeup water pumps or ESWS Cooling Tower basin makeup control valves. These controls are described in in FSAR Subsections 9.2.5 and Section 10.4.5. Water use and annual mean flow are discussed in Subsection 2.4.1.2. Although the Texas Water Code requires a permit for water use, there are no specific limitations set by state regulations. Water use from the Brazos River and Lake Granbury is administered by the Brazos River Authority. HYDSV-16

Low-flow frequency analysis was performed in accordance with USGS Bulletin 17B using the Log-Pearson Type III distribution method. The USGS gage (08090800) on the Brazos River located near Dennis, Texas between Morris Sheppard Dam and De Cordova Bend Dam was used to analyze the current regulated conditions of the Brazos River at the intake. Table 2.4.11-204 provides a summary of low flow frequencies for selected durations and return periods.

The 30-day 100-yr drought flow rate for Brazos River near Dennis, TX is estimated to be 9.7 cfs. A 100-yr return period is defined as a 1 percent chance groundwater was encountered during excavation or construction of CPNPP Units 1 and 2; therefore, there was no dewatering at the site during or after construction of the units (Reference 2.4-214).

2.4.12.2.4 On-Site/Vicinity Groundwater Level Fluctuations

~~Beginning i~~In October ~~through November~~ 2006, a groundwater investigation was initiated as part of the subsurface study to evaluate hydrogeologic conditions for the CPNPP Units 3 and 4. As part of this groundwater investigation, 47 monitoring wells were installed at 20 locations within the Glen Rose Formation on-site. Figure 2.4.12-208 shows the monitor well locations. Details regarding well construction are presented in Table 2.4.12-208. HYDSV-20 HYDSV-20 RCOL2_2.4.13-4

Due to the variable nature of groundwater reported at the CPNPP site, the well clusters were installed across CPNPP Units 3 and 4 from west to east of the reactor areas to define the groundwater bearing capabilities and properties of the zones likely to be affected, and to identify the hydraulic connectivity between the zones, if any. Monitoring wells were designated as follows, where XX denotes the well or cluster number for the three zones: CTS-00655

A-zone wells: Regolith or undifferentiated fill monitoring wells (MW-12XXa) were installed if greater than 10 ft of soil was encountered above hollow-stem auger refusal.

B-zone wells: Shallow bedrock monitoring wells (MW-12XXb) were generally completed in the upper 40 to 65 ft of bedrock in an apparent zone of alternating stratigraphy; i.e., claystone, mudstone, limestone, and shale sequences.

C-zone wells: Bedrock monitoring wells (MW-12XXc) were generally completed in deeper bedrock zones consisting of alternating stratigraphy and competent bedrock.

Following well development, water levels were measured from November 2006 to ~~November 2007~~May 2008 (Figure 2.4.12-209) to characterize seasonal trends in groundwater levels ~~and to identify preferential flow pathways surrounding CPNPP Units 3 and 4~~. The hydrographs for this groundwater data are presented on Figure 2.4.12-209 ~~for each of the three zones investigated~~and also show precipitation data. The groundwater elevation data is presented by well/cluster location and includes approximate screen elevations for each well in the cluster. In addition, the hydrographs depict rainfall totals for the period of interest. Rainfall data presented was collected from the Opossum Hollow rain gauge located approximately 3.4-mi southwest of the CPNPP Unit 3 and 4 site. Overall, the hydrographs show that water levels in the deeper Glen Rose Formation (C-Zone) do not fluctuate and remain at a constant level near the base of the well or depict a steadily increasing water level, indicating that this water is not actual groundwater. Hydrographs from the shallow bedrock wells (B-Zone) show a slow and steady increase of water levels over time with little to no fluctuations, also suggesting water levels are related to infiltration from the overlying soils and not HYDSV-20 HYDSV-20 actual groundwater. Available historical information on groundwater and groundwater trends in the Glen Rose Formation was presented in Subsection 2.4.12.2.3.

~~Four quarterly groundwater gradient maps were developed for each of the zones investigated. The gradient maps are discussed below for each zone.~~ HYDSV-20

Water Levels and Potentiometric Elevations in the Regolith (A – Zone)

Groundwater steadily increased from December 2006 to July 2007. Water levels remained constant or decreased slightly from August 2007 to ~~November~~February 2007. Hydrographs from the regolith/fill material wells (A-zone) indicate some slight fluctuations that may be tied to seasonal rainfall. In some of the A-zone wells, there appears to be a slight increase in water levels that may correspond to the spring seasons but there is no significant correlation in the A-zone wells across the site in response to rainfall.~~Overall, the water level trend in the regolith monitoring wells appeared to coincide with rainfall totals at the site.~~ HYDSV-20 RCOL2_2.4.13-4

Monitoring well MW-1211a was installed on the northeast portion of CPNPP Units 3 and 4 in undifferentiated fill material. Water levels in this monitoring well were consistent with the normal pool elevation of SCR (775 ft msl) indicating possible hydraulic communication between the former drainage swale and SCR.

Representative potentiometric surface maps for the four quarters (Figure 2.4.12-210 [Sheets 1 through 4]) show that the general shallow (A-Zone) groundwater movement in the vicinity of CPNPP Units 3 and 4 mimics the surface topography, with an apparent groundwater divide along the long axis of the site peninsula. On the northern portion of the peninsula, a northerly flow toward SCR is observed, and a southerly flow toward the Safe Shutdown Impoundment (SSI) is observed on the south side of the site peninsula. HYDSV-20

Water Levels and Potentiometric Elevations in the Shallow Bedrock (B – Zone)

Nine of the 16 wells completed in this zone contained no, or negligible, amounts of water for up to eight months before exhibiting measurable water (greater than 1 ft). The majority of ~~T~~these wells exhibited a slow to steady recharge, with no indication of reliable equilibrium conditions over the monitoring period. HYDSV-20

Six monitoring wells screened in shallow bedrock exhibited no, or slight, changes in water level over the monitoring period. One of these wells (MW-1211b) was installed on the northeast portion of CPNPP Units 3 and 4 in the undifferentiated fill material. During installation, an effort was made to install this well in bedrock; however, due to the thickness and nature of the undifferentiated fill material, the boring was terminated at the bedrock surface (approximately 75 ft below ground surface [bgs]). Water level measurements for this well were consistent with those of regolith monitoring well MW-1211a and the normal pool elevation of SCR over the monitoring period.

One monitoring well screened in the shallow bedrock exhibited variable water levels, with no indication of reliable equilibrium conditions when compared to other wells with similar screened zones. Monitoring well MW-1217b, located near the center point of CPNPP Unit 3 exhibited an approximate 15 ft increase in water level from December 2006 to March 2007 followed by a decline of 5 ft through May 2007. From May 2007 to November 2007, this well exhibited a water level increase of approximately 7 ft, and from January 2008 to May 2009 exhibited a water level decrease of approximately 7 ft. HYDSV-20

Representative potentiometric surface maps (Figure 2.4.12-210 [Sheets 5 through 8]) show (B-Zone) groundwater movement in the vicinity of CPNPP Units 3 and 4 flows to the east in the general direction of the dip of the Glen Rose Formation. HYDSV-20

Water Levels and Potentiometric Elevations in the Bedrock Monitoring Wells (C - Zone)

Of the 13 groundwater monitoring wells screened in bedrock, eight contained no, or negligible, amounts of water over the monitoring period and ~~five~~six exhibited a slow to steady recharge, with no indication of reliable equilibrium conditions. HYDSV-20 RCOL2_2.4.13-4

Representative potentiometric surface maps for the four quarters (Figure 2.4.12-210 [Sheets 9 through 12]) show that the (C-Zone) groundwater movement in the vicinity of CPNPP Units 3 and 4 flows to the east in the general direction of the dip of the Glen Rose Formation. The water levels in the regolith/fill material and the upper zone of the Glen Rose Formation (A-zone and B-zone, respectively) were attributed to surface run-off and were not a true measure of permanent groundwater in the formation. Negligible groundwater has been gauged in the C-zone wells representing essentially dry conditions. Consequently, this zone is not considered a groundwater bearing unit. HYDSV-20 HYDSV-20 RCOL2_2.4.13-4

2.4.12.2.5 Aquifer Characteristics

Groundwater has been identified within the undifferentiated fill, regolith and bedrock beneath the CPNPP Units 3 and 4 sites; therefore, this subsection provides characteristics of these zones. During construction, the undifferentiated fill material and regolith are expected to be removed ~~and replaced with engineered fill material~~in the power block area. The foundation elevation is estimated to be approximately 782 ft msl on the bedrock. Groundwater currently measured in the soil zones (undifferentiated fill material and regolith) and the Glen Rose Formation is considered “perched” and will be ~~dewatered~~removed during construction activities. Characteristics of the Glen Rose Formation indicate that it is not a groundwater bearing unit and a permanent dewatering system will not be required. RCOL2_2.4.13-4

2.4.12.2.5.1 Porosity

Soil Zones

The soils occurring on the CPNPP site are described in the Hood and Somervell counties soil survey information provided by the USDA Natural Resources Conservation Service's on-line Soil Data Mart website (Reference 2.4-259). A total of 18 soil mapping phases representing 17 soil series occur within the CPNPP site boundary. Descriptions of each soil series are provided in Table 2.4.12-210 and the location of the soil mapping phases are shown on Figure 2.4.12-211.

The two soil types mapped in the vicinity of the CPNPP Units 3 and 4 build areas include the Tarrant – Bolar association and Tarrant – Purves association. Physical properties for these soil types indicate clay content ranges of 20 to 60 percent, moist bulk densities of 1.10 gram per cubic centimeter (g/cc) to 1.55 g/cc, saturated hydraulic conductivities between 4.2 x 10-5 centimeters per second (cm/sec) and 1.4 x 10-3 cm/sec, and available water capacities of 0.05 inch per inch (in/in) to 0.18 in/in (Reference 2.4-260).

The site is underlain by a sedimentary rock sequence of the Glen Rose Formation which, at the surface, has been weathered to a clayey, silty, sandy overburden soil with some rock fragments (referred to as regolith). However, most of the CPNPP site is situated in areas disturbed by previous construction activities associated with the construction of CPNPP Units 1 and 2. Porosity in the undifferentiated fill or regolith materials was evaluated based on the grain size distributions from the current investigation:

- Undifferentiated Fill - Based on the grain size distribution of the on-site soils, the total porosity was determined by averaging the porosity range for sand, silt, and clay. The average total porosity of the on-site regolith and undifferentiated fill is assumed to be 0.45. Based on a lack of information regarding effective porosity in the undifferentiated fill, an effective porosity of 0.45 was assumed.

- Regolith – As mentioned above, the average total porosity of the on-site regolith and undifferentiated fill/regolith (soils) is assumed to be 0.45. To estimate the effective porosity of the on-site soils, the arithmetic mean of the effective porosities for fine grained sand, silt, and clay were averaged (Reference 2.4-261). The average effective porosity of the on-site regolith and undifferentiated/regolith is assumed to be 0.20.

Bedrock Zones

The bedrock is comprised of limestone from the Glen Rose Formation. ~~The shallow bedrock porosity values from geotechnical borings B-1007 and B-1029 were used to estimate the porosity in the vicinity of groundwater monitoring well MW-1215b. The porosity values from geotechnical borings B-2000, B-2008, and B-2029 were used to estimate the porosity values in the vicinity of groundwater monitoring well MW-1217b. An average total porosity of the shallow bedrock in the vicinity of CPNPP Units 3 and 4 of 0.24 is assumed. The effective porosity of limestone is assumed to be 0.14 (Reference 2.4-261).~~The results of the HYDSV-18 HYDSV-24 geotechnical analysis performed at the CPNPP Units 3 and 4 site indicated that an average total porosity of the shallow bedrock (limestone and shale) is 25.6 percent and the average total porosity of limestone is 11.9 percent. The Argonne National Laboratory publication, Data Collection Handbook to Support Modeling Impacts of Radioactive Material in Soil, dated April 1993 (Reference 2.4-261) references an arithmetic mean of the effective porosity for limestone of 14 percent. Consequently, the most conservative approach when determining velocity and travel time is to use the measured 11.9 percent porosity value which provides a higher calculated velocity through the shallow bedrock. HYDSV-18 HYDSV-24

2.4.12.2.5.2 Permeability

The permeability of a material is a measure of the ability to transmit water. To assist in determining permeability of the Glen Rose Formation, forty packer-pressure tests were performed in five test borings at 5-foot intervals of varying depth at CPNPP Units 3 and 4 in 2007. The results of these packer tests indicated little to no water take into the Glen Rose Formation; therefore, the formation is essentially impermeable. Detailed examination of cores from test borings revealed minor solutioning features and minimal fractures. Drill water occasionally was lost while drilling through the upper weathered zone and is believed to have occurred at the soil/bedrock interface.

2.4.12.3 Subsurface Pathways

Subsurface pathways include the unsaturated zones and saturated zones beneath the CPNPP Units 3 and 4. Groundwater is the primary transport mechanism for possible liquid effluent release. Groundwater movement and velocity will vary depending on the matrix through which it flows. The rate of flow (i.e. the velocity) of groundwater depends on (1) the hydraulic conductivity and porosity of the medium through which it is moving and (2) the hydraulic gradient. Higher groundwater velocities occur with greater hydraulic conductivity and hydraulic gradient.

Hydraulic conductivity is greatest in the regolith material; therefore, the regolith material typically has a higher rate of flow than the undifferentiated fill and bedrock. Based on information from the field investigation, the bedrock formation in the area of the CPNPP site is poorly developed in that groundwater flow within the bedrock is dominated by isolated layers of claystone, mudstone, limestone, and shale. Movement of water in a granular aquifer can be characterized by use of Darcy's Law; therefore, application of Darcy's Law calculations is appropriate for the regolith, undifferentiated /regolith, and shallow bedrock systems found at CPNPP Units 3 and 4. Average interstitial groundwater flow velocity for the regolith, undifferentiated/regolith, and shallow bedrock systems units was determined using a form of the Darcy equation as follows:

$$V = (K_h \times (EH - EL)/L)/\eta$$ (Reference 2.4-262)

Where: V = groundwater flow velocity, ft/day detail in Subsection 2.4.13 to assist in the development of transport calculations for fate and transport analyses in the event of accidental releases of effluents to groundwater.

2.4.12.3.1 Groundwater Pathways

Although the discussions of groundwater movement is a reasonable scenario for groundwater flow, it is assumed that the actual groundwater is subject to three-dimensional control structures (horizontal, vertical, and any secondary porosity that may be present) and does not have uniform flow across the site.

Groundwater pathways are considered from the Unit 3 and 4 Auxiliary Buildings, where the boric acid tank (BAT) is located, to SCR, which is the nearest potential receptor.~~Two postulated groundwater pathway scenarios, Unit 3 to SCR (through the regolith and the undifferentiated fill) and Unit 4 to SCR (through the undifferentiated fill and regolith), represent the most conservative pathways from a two reactor site where groundwater flow is possible in different directions from each unit. Both flow paths use a conservative straight-line flow path approach, using the shortest distance and the highest measured hydraulic conductivity. A straight line flow path would be considered the most conservative as the actual groundwater pathways are expected to be tortuous, resulting in longer transport times, and hydraulic conductivities (K_h) of the fractures/joints would be (or are) expected to be lower than the highest measured on-site. The straight line distance from Unit 3 to SCR is 530 ft and the straight line distance from Unit 4 to SCR is 607 ft. The steepest measured gradient for the undifferentiated fill material from Unit 3 to SCR is 0.104 ft/ft and from Unit 4 to SCR is 0.109 ft/ft. To calculate the travel time in the undifferentiated fill material from each of the units to SCR, the highest measured hydraulic conductivity of 5.00 X 10^{-4} cm/s was used. Table 2.4.12-211 provides the calculated travel times based on monthly measured gradients.~~ HYDSV-23 CTS-00656 RCOL2_2.4.13-5

Postulated groundwater pathway scenarios include; Unit 3 Auxiliary Building (A/B) to SCR through the regolith and the undifferentiated fill; Unit 3 A/B to SCR through the Glen Rose Limestone; Unit 4 A/B to SCR through the undifferentiated fill and regolith; and Unit 4 A/B to SCR through the Glen Rose Limestone. Due to the planned removal of all overburden material down to the plant grade elevation of 822-ft, and the sub-grade elevation of the boric acid tank of 793 ft, the pathway scenarios through the undifferentiated fill and regolith are considered not plausible and are not discussed further. For the post-construction groundwater pathways the two remaining pathway scenarios, Unit 3 A/B to SCR through the Glen Rose Limestone and Unit 4 A/B to SCR through the Glen Rose Limestone, are considered to represent the most conservative pathways from a two reactor site where groundwater flow is possible in different directions from each unit. Using the most conservative straigh-line approach, two flow paths are considered from Unit 3 A/B to SCR and two flow paths are considered from Unit 4 A/B to SCR (Figure 2.4.12-212). These flow paths consider the most plausible straight-line groundwater flow direction from the release points to SCR and the highest measured hydraulic conductivity. A straight-line flow path would be considered the HYDSV-23 most conservative as the actual groundwater pathways are expected to be tortuous, resulting in longer transport times, and hydraulic conductivities (Kh) of the fractures/joints would be (or are) expected to be lower than the highest measured on-site. HYDSV-23

To estimate groundwater travel time through the Glen Rose Formation, the average porosity of limestone of 0.119, the highest hydraulic conductivity measured at the site (1.37×10^{-5} cm/s), and the steepest hydraulic gradient measured from the monthly gauging events of the nearest groundwater monitoring wells to the Unit 3 and 4 Reactor Buildings (Table 2.4.12-211) were used for the pathway analysis.

For groundwater pathway 3a, it is assumed that an instantaneous release from the BAT would travel out of the Unit 3 A/B northeast towards SCR where it would encounter a minimum of 100 lateral feet of Glen Rose Formation followed by the fill material of the Unit 3 Ultimate Heat Sink (UHS) and then by post-construction engineered fill material before reaching SCR. Since the physical properties of the engineered fill material may change as the design is finalized and the potential exists for groundwater flow through the fill material of the Unit 3 UHS, it is conservatively estimated that an instantaneous release to SCR will occur once the Unit 3 UHS is encountered. The travel time from the Unit 3 A/B through a minimum of 100 feet of Glen Rose Formation to the Unit 3 UHS is 3146 days. Therefore, a conservative estimate of the time it would take a release to travel from the Unit 3 A/B to SCR along pathway 3a is 3146 days.

For groundwater pathway 3b, it is assumed that an instantaneous release from the BAT would travel out of the Unit 3 A/B through the fill material of the Unit 3 Reactor Building (R/B) due east towards SCR where it would encounter a minimum of 80 lateral feet of Glen Rose Formation followed by the fill material of the Unit 3 Essential Service Water (ESW) Pipe Tunnel and an undetermined lateral distance of Glen Rose Formation followed by post-construction engineered fill and undifferentiated fill material before reaching SCR. Since the physical properties of the engineered fill material are unknown at this time and the physical properties of the undifferentiated fill material are estimated, and the potential exists for groundwater flow through the fill material of the Unit 3 ESW Pipe Tunnel, it is conservatively estimated that an instantaneous release to SCR will occur once the ESW Pipe Tunnel is encountered. The travel time from the Unit 3 A/B and R/B through a minimum of 80 feet of Glen Rose Formation to the Unit 3 ESW Pipe Tunnel is 2516 days. Therefore, a conservative estimate of the time it would take a release to travel from the Unit 3 A/B to SCR along pathway 3b is 2516 days.

Cross sections depicting the post-construction groundwater flow pathways from Unit 3 to SCR are presented in Figure 2.4.12-213.

For groundwater pathway 4a, it is assumed that an instantaneous release from the BAT would travel out of the Unit 4 A/B north-northwest towards SCR where it would encounter a minimum of 60 lateral feet of Glen Rose Formation followed by the fill material of the Unit 4 Ultimate Heat Sink (UHS) and then by post-construction engineered fill material before reaching SCR. Since the physical properties of the engineered fill material are unknown at this time and the potential exists for groundwater flow through the fill material of the Unit 4 UHS, it is conservatively estimated that an instantaneous release to SCR will occur once the Unit 4 UHS is encountered. The travel time from the Unit 4 A/B through a minimum of 60 feet of Glen Rose Formation to the Unit 4 UHS is 1916 days. Therefore, a conservative estimate of the time it would take a release to travel from the Unit 4 A/B to SCR along pathway 4a is 1916 days. HYDSV-23

For groundwater pathway 4b, it is assumed that an instantaneous release from the BAT would travel out of the Unit 4 A/B northeast towards SCR where it would encounter a minimum of 120 lateral feet of Glen Rose Formation followed by the fill material of the Unit 4 Ultimate Heat Sink (UHS) and undocumented fill and engineered fill before reaching SCR. Since the physical properties of the undocumented fill are estimated and the physical properties of the engineered fill material are unknown at this time, and the potential exists for groundwater flow through the fill material of the Unit 4 UHS and through the undocumented fill, it is conservatively estimated that an instantaneous release to SCR will occur once the Unit 4 UHS is encountered. The travel time from the Unit 4 A/B through a minimum of 100 feet of Glen Rose Formation to the Unit 4 UHS is 3834 days. Therefore, a conservative estimate of the time it would take a release to travel from the Unit 4 A/B to SCR along pathway 4b is 3834 days.

Cross-sections depicting the post-construction groundwater flow pathways from Unit 4 to SCR are presented in Figure 2.4.12-214.~~Based on the average effective porosity of 0.20 and the parameters stated above, the groundwater travel time from CPNPP Unit 3 to SCR in the undifferentiated fill/regolith is 720.9 days and the travel time in the undifferentiated fill/regolith from Unit 4 to SCR is 782.6 days.~~

~~The undifferentiated fill is expected to be removed and the plant grade elevation of 822 ft would then be situated near the top of the Glen Rose Formation. Using the average porosity of limestone, 0.14, the highest hydraulic conductivity, 1.37 X 10^-5 cm/s, and the steepest gradient measured from the monthly gauging events (Table 2.4.12-211), the travel time from Unit 3 to SCR through the bedrock is 19,615.0 days and the travel time from Unit 4 to SCR through the bedrock is 22,737.6 days.~~ RCOL2_2.4.13-5 CTS-00657 RCOL2_2.4.13-5

The current soil and rock material comprising the hydrologic A-zone (undifferentiated fill and regolith) and B-zones (shallow bedrock) discussed in Subsection 2.4.12.2.4 will be removed for construction of plant foundations, resulting in the removal of the perched groundwater from the power block area. Post-construction surface water infiltration to the Glen Rose Formation limestone will be reduced with the construction of surface water impoundments and an improved drainage system throughout the CPNPP Units 3 and 4 site. The grading and drainage plan and placement of engineered fill material are designed to preclude surface water infiltration into the limestone on which the foundation will be constructed. RCOL2_2.4.13-4

2.4.13 Accidental Releases of Radioactive Liquid Effluent in Ground and Surfacewaters

CP COL 2.4(1) Add the following at the end of the DCD Subsection 2.4.13.

Historical and projected groundwater flow paths were evaluated in Subsection 2.4.12 to characterize groundwater movement from the nuclear island area to a point of exposure. ~~Due to the higher groundwater velocity and faster travel time in the shallow soils (regolith/undifferentiated fill), this flow path is expected to be the bounding pathway of radionuclide migration. This pathway represents the most rapid transport for water released by a liquid tank failure.~~ Figure 2.4.12-203 depicts subsurface conditions that control the movement of groundwater beneath the CPNPP Unit 3 and 4 site. Based on groundwater flow directions (Figure 2.4.12-20~~8~~9, Sheets 1, 4, 7, and 10), different flow paths are applicable from Units 3 and 4 via horizontal groundwater movement to the nearest surfacewater body (SCR). Subsection 2.4.12 provides the locations and users of surface water in the CPNPP site area. CTS-00808 CTS-00808

~~The Twin Mountains Formation is the nearest aquifer used for public supply. Since the Twin Mountains Formation is separated from the shallow soils by approximately 238 ft of the dense, impermeable limestone contained in the Glen Rose Formation, this pathway was not evaluated at the CPNPP Unit 3 and 4 site.~~ RCOL2_2.4.13-4

A conceptual model of radionuclide transport through groundwater to the nearest surfacewater body is described below. The conceptual model and alternate conceptual model developed consider both vertical and horizontal radioactive liquid effluent transport based upon the post-construction configuration of CPNPP Units 3 and 4 (see Figures 2.4.12-212 through 2.4.12-214).~~The US-APWR DCD Subsection 11.2.3.2 evaluates the consequences of postulated failure of the holdup tanks, the waste holdup tanks, and the boric acid tanks. Subsection 11.2.3 indicates cubicles containing tanks of radioactive liquid are steel-lined up to a height of the full tank. In the event that the tank fails, the potential for groundwater contamination is greatly reduced. Consequently, release points are not identified.~~ HYDSV-30 RCOL2_2.4.13-7

2.4.13.1 Identification of Source Term and Soil/Water Distribution of Liquid Effluent RCOL2_2.4.13-6a

In performing the evaluation of Postulated Radioactive Releases Due to Liquid-Containing Tank Failures, the following tanks were considered in determining which tank would have the highest concentration and the largest volume of radionuclides: CTS-00808

Holdup Tank - located in the Auxiliary Building (A/B), a Seismic Category II building.

Waste Holdup Tank - located in the A/B

Boric Acid Evaporator - located in the A/B

Boric Acid Tank - located in the A/B

Volume Control Tank -located in the Reactor Building (R/B). a Seismic Category I Building. RCOL2_2.4.13-6a

Auxiliary Building Sump Tank - located in the A/B

Reactor Building Sump Tank - located in the R/B

Primary Makeup Water Tank - located outside

Refueling Water Storage Auxiliary Tank - located outside

Chemical Drain Tank - located in the A/B

The Volume Control Tank, the Chemical Drain Tank, and Sump Tanks were eliminated from consideration based on smaller volumes and lower radionuclide contents than the Boric Acid Tank (BAT). The Primary Makeup Water Tank was eliminated from consideration based upon the fact that the Primary Makeup Water Tank stores demineralized water from the Treatment System. and low level radioactive condensate water from the Boric Acid Evaporator. Condensate water contains low levels of radionuclide concentrations. including tritium. Additionally, the Refueling Water Storage Auxiliary Tank (RWSAT) was eliminated from consideration because it stores refueling water. Prior to refueling, tank water is supplied to the refueling cavity where the reactor coolant radionuclide concentration dilutes with refueling cavity water. Radionuclide concentration of cavity water is reduced by the purification system of the Chemical and Volume Control System (CVCS) and the Spent Fuel Pit Cooling and Purification System (SFPCS) during refueling operations. Upon refueling completion, part of the cavity water is returned to this tank where the radionuclide concentration is low. Accordingly. the impact of RWST or Primary Makeup Water Storage Tank failure is small.

After eliminating the tanks described above, the remaining tanks left to consider for the failure analysis are those in the A/B, which is a seismic category II Building. As shown in US-APWR DCD Figure 1.2-29, these tanks are located on the lowest elevation of the ~~Auxiliary Building~~A/B at elevation 793 ft ms. In selecting the appropriate tank for the failure analysis, NUREG-0133 and the RATAF Code for Pressurized Water Reactors were utilized. The concentration of the radioactive liquid in the tanks, such as the Boric Acid Evaporator, the Holdup Tank, and the BAT, are larger than the Waste Holdup Tank since they receive reactor coolant water extracted from the Reactor Coolant System. Since the enrichment factor of 50 is considered for the liquid phase of the Boric Acid Evaporator. the radioactive concentrations in the liquid phase of the Boric Acid Evaporator, and in the BAT (which receives the enriched liquid from the Boric Acid Evaporator) becomes large when compared to the other tanks. The BAT has been selected since its volume is RCOL2_2.4.13-6a larger than the liquid phase of the Boric Acid Evaporator. Credit is taken for the removal effect by demineralizers or other treatment equipment for the liquid radioactive waste prior to entering the tank. No chelating agents are used in the plant system design in order to provide chemical control of the reactor coolant. Only a very small amount of chelating agents is used in the sampling system for analysis. The sampling drain, which contains only a small amount of chelating agents is directly sent to the dedicated chemical drain tank and treated separately. Chemical agents used in laboratory analysis are also sent to the chemical drain tank for treatment. Therefore, neither the chelating agents nor the chemical agents used in the sampling analysis will have any effect on the transport characteristics of the source term liquid effluent release analysis. RCOL2_2.4.13-3 HYDSV-28

The source term concentrations considered for these tanks are identified in DCD Table 11.2-17 and show the radioactivity concentrations closest to the nearest potable water supply. The BAT is located in the northeast (NE) corner of the A/B (see DCD Figure 12.3-1). The A/B basemat elevation is at approximately 785 ft msl. The BAT elevation is expected to be at 793 ft msl. Ground level at the site is expected to be at 822 ft msl. The BAT contained the largest concentration and volume of radionuclides that was closest to the effluent concentration limits for Cs-134 and Cs-137, yet well below the 10 CFR 20, Appendix B limits. Isotope concentrations less than 1.0×10^{-3} in fraction of concentration limits are excluded from the evaluation. Since credit cannot be taken for liquid retention by unlined building foundations, it is assumed that 80 percent of the contents of each tank is released to the environment, consistent with the guidance in BTP 11-6, March 2007. In releasing the contents of one tank, it is assumed that 80 percent of the tank volume is discharged and the dilution factor of each tank is 4.4×10^{10} gallons. RCOL2_2.4.13-6b CTS-00810 RCOL2_2.4.13-1

In performing the tank failure analysis, no credit is taken for the distribution of radiological liquid waste to the surrounding subsurface media and groundwater.~~, which is below grade. With the failure of a liquid tank inside the Auxiliary Building and subsequent liquid release to the environment, radionuclides enter the subgrade soils below the surrounding grade. A conservative model assumes the effluent liquid completely fills the soil pore space in an area large enough to contain the tank contents. Radionuclides are then released to the groundwater and transported to SCR where the volume of water contained in the reservoir is expected to dilute their concentration and eliminate impact to potential future water users. The overburden soils continually receive the average annual on-site precipitation. The precipitation that does not runoff or is lost to evapotranspiration infiltrates through the unsaturated zone and contributes to groundwater transport to SCR.~~ RCOL2_2.4.13-7

While groundwater functions as the transport media for fugitive radionuclides, interaction of individual radionuclides with the soil matrix delays their movement. The solid/liquid distribution coefficient, K~~d~~$_d$, is, by definition, an equilibrium constant that describes the process wherein a species (e.g., a radionuclide) is partitioned by adsorption between a solid phase (soil) and a liquid phase (groundwater). Soil properties affecting the distribution coefficient include the CTS-00659 considered. The BAT was selected as the tank that had the greatest volume and largest concentration of radionuclides. Cs-137 and Cs-134 were nuclides of interest in the BAT since credit is taken for removal equipment and demineralizer beds. Cs-137 was one of the nuclides selected for K_d analysis Movement of Cs-134 through the subsurface media would be similar to Cs-137 as they have chemically and radiologically similar characteristics. The purpose of the K_d analysis was to estimate the potential migration of accidental releases from the footprint areas of the proposed new units. The K_d results presented in Table 2.4.13-201 indicate that the radionuclides would be delayed in their movement through the groundwater pathway to SCR. The tank failure analysis assumed no distribution of contaminants (no K_d coefficients used) based upon the site-specific hydrogeological characteristics. It is conservatively assumed that the contaminants would transport along the groundwater pathway horizontally to SCR without retardation or retention in the subsurface media, and that there would be no groundwater dilution prior to reaching SCR. RCOL2_2.4.13-2

2.4.13.2 Development of Alternate Conceptual Model and Site-Specific Geological and Hydrogeological Parameters RCOL2_2.4.13-5

The alternative conceptual models were used to determine a bounding set of plausible groundwater flow paths by considering the nearest surface water body, SCR, current groundwater elevations measured in wells near the proposed power block area, the measured pool elevation of SCR (gradient to the SCR) and a conservative pathway from a postulated release point to SCR.

After exploring alternative transport pathways, six plausible pathways were determined to bound potential release pathways. Refer to Figure 2.4.13-212 and associated cross section Figures 2.4.12-213 and 2.4.12-214 for the horizontal release pathways 3a, 3b, 4a, 4b. Vertical release pathways 3c and 3d were eliminated from consideration as discussed in Subsection 2.4.13.4. Alternate horizontal groundwater pathways from each unit moving from southwest or southeast from the BAT A/B location were eliminated from consideration as this movement would be away from SCR and would not be consistent with the hydraulic gradients for the area surrounding the CPNPP Units 3 and 4 shown on Figure 2.4.12-210, Sheets 1 through 12. HYDSV-30

CPNPP Units 3 and 4 are to be constructed on the Glen Rose Formation. The Glen Rose limestone is essentially impermeable, ranging from 217 to 271 ft thick, and is underlain by the Twin Mountains Formation, which contains the first aquifer beneath the site. Figures 2.5.5-202 and 2.5.5-203 provides a generalized cross section of the pre-construction site conditions. Figures 2.4.12-213 and 2.4.12-214 show the post-construction pathway cross-sections for the shortest distance releases to SCR via groundwater for pathways 3a, 3b and 4a, 4b. The groundwater flow pathways were developed based on groundwater measured in monitoring wells in the CPNPP Unit 3 and 4 plant area and measured elevations in SCR. Wells were installed across the site in zones to define the groundwater bearing capabilities and properties of the zones, and identify the hydraulic RCOL2_2.4.13-4 HYDSV-17 HYDSV-19 HYDSV-23 HYDSV-30 connectivity between the zones, if any. The well zones are defined as A-Zone (regolith or undifferentiated fill material), B-Zone (shallow bedrock) and C-Zone (deeper bedrock) and are described in Subsection 2.4.12.2.4. HYDSV-17 HYDSV-19 HYDSV-23 HYDSV-30

The process used to develop alternative conceptual models of groundwater flow included the following:

- Groundwater flow pathways were developed based on groundwater measured in monitoring wells in the Units 3 and 4 plant area, measured elevations in SCR, surface topography, and observed water levels over time.

- Groundwater measured in all three zones was considered perched based on measurements. Groundwater in the A-zone regolith was attributed to surface water infiltration. Groundwater measured in the undifferentiated fill near SCR was attributed to SCR.

- Groundwater in the B-zone was not continuous across the site Non-equilibrium conditions and the reported dry wells in the B-zone wells indicated that the groundwater was perched. Groundwater located in fill areas near SCR was found to be in communication with SCR. RCOL2_2.4.13-4

- Negligible groundwater was gauged in the C-zone wells, representing essentially dry conditions. Consequently, this zone was not considered a groundwater bearing unit.

- Post-construction section configuration of the A/B building, the Ultimate Heat Sink (UHS) cooling tower structure area and other structures were used in identifying the bounding set of plausible pathways. In addition to Figures 2.4.12-213 and 2.4.12-214 horizontal pathway cross sections, the following site plan views and section plans were utilized in identifying the bounding set of plausible pathways: HYDSV-17 HYDSV-19 HYDSV-23 HYDSV-30

  - Site Plan View Figure 1.2-1R;

  - Power Block at Elevation 793” ft msl Plan View Figure 1.2-2R;

  - ESW Pipe Tunnel Sectional View A-A’ Figure 1.2-202; and

  - Ultimate Heat Sinks A and B Sectional Views Figure 1.2-206.

- Rainfall infiltration effect on the liquid effluent and plausible release pathway is also considered based upon post-construction structures and building configurations. Rainfall infiltration is not considered a contributing factor affecting the source term release pathway. No dilution effects of groundwater or rainfall are considered in the liquid effluent release analysis. Rainfall infiltration effects are discussed in Subsection 2.4.13.3. HYDSV-17 HYDSV-19 HYDSV-23 HYDSV-30

2.4.13.3 Potential Effects of Construction on Groundwater Flow Paths

RCOL2_2.4.13-4

The current soil and rock material comprising the hydrologic A-zone (undifferentiated fill and regolith), and the B-zone (shallow bedrock) will be removed for construction of plant foundations, resulting in the removal of the perched groundwater from the plant area. Post-construction surface water infiltration to the Glen Rose Formation limestone will be reduced with the construction of surface water impoundments and an improved drainage system throughout the Units 3 and 4 site. The grading and drainage plan and placement of engineered fill material are designed to preclude surface water buildup near the plant foundation, reducing the possibility of surface water infiltration into the limestone on which the foundation will be constructed.

During construction, the undifferentiated fill material and regolith will be removed in the power block area, and replaced with engineered fill material. A dewatering system will not be used but rainfall and seepage will be removed during construction.

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In October 2006, a groundwater investigation was initiated as part of the subsurface study to evaluate hydrogeologic conditions for CPNPP Units 3 and 4. As part of this groundwater investigation, 47 monitoring wells were installed at 20 locations within the Glen Rose Formation onsite. Due to the variable nature of groundwater reported at the CPNPP site, the well clusters were installed across the footprint of CPNPP Units 3 and 4 from west to east of the reactor areas to define the groundwater bearing capabilities and properties of the zones likely to be affected, and to identify the hydraulic connectivity between the zones, if any. Following well development, water levels were measured from November 2006 to May 2008 to characterize seasonal trends in groundwater levels.

Rainfall data presented was collected from the Opossum Hollow rain gauge located approximately 3.4-mi southwest of the CPNPP Unit 3 and 4 site. Hydrographs were developed and are presented in Figure 2.4.12-209. These hydrographs show that water levels in the deeper Glen Rose Formation (C-zone) do not fluctuate and remain at a constant level near the base of the well or depict a steadily increasing water level, indicating that this water is not actual groundwater. Hydrographs from the shallow bedrock wells (B-zone) show a slow and steady increase of water levels over time with little to no fluctuations, also suggesting water levels are related to infiltration from the overlying soils and not actual groundwater. Hydrographs from the regolith/fill material wells (A-zone) indicate some slight fluctuations that may be tied to seasonal rainfall. In some of the A-zone wells there appears to be a slight increase in water levels that may correspond to the spring seasons but there is no significant correlation in the A-zone wells across the site in response to rainfall.

The water levels in the regolith/fill material and the upper zone of the Glen Rose Formation (A-zone and B-zone, respectively) were attributed to surface run-off and were not a true measure of permanent groundwater in the formation.

Groundwater steadily increased from December 2006 to July 2007. Water levels remained constant or decreased slightly from August 2007 to February 2007. HYDSV-17 HYDSV-19 HYDSV-23 HYDSV-30

Nine of the 16 wells completed in Shallow Bedrock (B – Zone) contained no, or negligible, amounts of water for up to eight months before exhibiting measurable water (greater than 1 ft). The majority of these wells exhibited a slow to steady recharge with no indication of reliable equilibrium conditions over the monitoring period.

Of the 13 groundwater monitoring wells screened in Bedrock (C-Zone), eight contained negligible to amounts of water over the monitoring period and six exhibited a slow to steady recharge with no indication of reliable equilibrium conditions.

The Grading and Drainage Plan shown on Figure 2.4-202 was developed based upon the effects of local intense precipitation, as discussed in Subsection 2.4.2.3, and aids in moving precipitation away from structures and buildings considered in the plausible pathways for the liquid effluent release analysis.

Rainfall infiltration is not considered a contributing factor affecting the source term release pathway. No dilution effects of groundwater or rainfall are considered in the liquid effluent release analysis.

2.4.13.4 Vertical Liquid Effluent Release Pathway RCOL2_2.4.13-6c

Both SCR and the Units 1 and 2 restricted potable water supplies wells were considered as receptors. The Units 1 and 2 potable water supply wells are restricted access potable water supply wells completed in the Twin Mountains Formation aquifer and approximately 1990 feet south of the Unit 3 A/B. The nearest unrestricted potable water supplies completed in the Glen Rose Formation are approximately 4 miles south of the Unit 3 A/B. and the nearest unrestricted potable water supply wells completed in the Twin Mountains Formation is approximately 1 mi west of the Unit 4 A/B (see FSAR Subsection 2.4.12.3.2 and Figures 2.4.12-204 and 2.4.12-206). The restricted potable water supply wells in Units 1 and 2 (see Figure 2.4.1-213) were not considered as possible receptors based upon the following: HYDSV-29 HYDSV-31

The BAT is at elevation 793 ft msl, while the Auxiliary Building basemat elevation is at 785 ft msl. Since the Auxiliary Building is a Seismic Category II Building, it is assumed that a crack will form in the building during a seismic event, and the radioactive liquid would travel vertically into the surrounding formation. At this basemat elevation of 785 ft msl, the hydrogeologic formation is in the deeper portion of the Glen Rose Formation, which consists primarily of impermeable limestone. For the release to reach the Twin Mountains Formation, which is approximately 150 feet below the Glen Rose Formation, the liquid release would have to travel completely through the Glen Rose Formation. Using the Units 1 and 2 vertical release pathways from the Glen Rose formation to the Twin Mountain RCOL2_2.4.13-6c HYDSV-19 HYDSV-29 HYDSV-31 formation for the Units 3 and 4 vertical pathways is credible based upon the following: HYDSV-19 HYDSV-29 HYDSV-31

- Discrete engineering layers in the Glen Rose formation can be traced in the subsurface throughout the site and correlated approximately 2000 feet away in the CPNPP Units 1 and 2 borings and historical excavation photographs.
- Known post-construction excavation limits can be correlated with the stratigraphy exposed in the Glen Rose formation photographs.

A complete discussion of the core borings stratigraphy and CPNPP Units 1 and 2 historical excavation photographs as compared to CPNPP Units 3 and 4 borings is provided in Subsection 2.5.4.3.1.

Units 1 and 2 performed an analysis and provided a model of this vertical release path (Reference 2.4-214). The results of the model indicate that the only radionuclide that would travel the length of the Glen Rose Formation was Cs-137, and that it would take approximately 400 years to reach the Twin Mountains Formation.

The closest Units 1 and 2 potable water supply well is approximately 1.25 miles away (Figure 2.4.1-213) from either the Unit 3 or Unit 4 Auxiliary Building (Figure 2.4.12-208). Considering that the liquid release would be in the Glen Rose Formation and the travel time vertically to the Twin Mountains formation is approximately 400 years for Cs-137 (one of the radionuclides considered in the Units 3 and 4 tank failure analysis), it is concluded that the vertical pathway to the Twin Mountains Formation is not plausible and accordingly, was eliminated as a pathway.

Because the Units 1 and 2 restricted potable water supplies were eliminated, the time for Cs-137 to travel through the Glen Rose Formation is approximately 400 years, and the nearest unrestricted potable water supply is approximately four miles south of the CPNPP site, the SCR receptor is considered the only plausible horizontal groundwater flow release path. The deeper bedrock is not conductive to groundwater travel due to the impermeable limestone layer. Therefore, the alternate conceptual models chosen were to transport the liquid radioactive release through the undifferentiated fill/regolith and shallow bedrock in a straight-line pathway to SCR (as described in Subsection 2.4.12.3.1 and shown on Figures 2.4.12-212 through 2.4.12-214).

2.4.13.5 Horizontal Liquid Effluent Groundwater Release Pathway RCOL2_2.4.13-5

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Site-specific groundwater flow velocities and travel times are presented in Table 2.4.12-211 and Subsection 2.4.12.1.1. Hydraulic conductivities, porosity, and bulk density of the subsurface soils and bedrock are described in Subsections 2.4.12.2.4, 2.4.12.2.5, 2.4.12.2.5.1. Groundwater pathways are discussed in Subsection 2.4.12.3. Four plausible groundwater pathways were identified.

- Unit 3 A/B to SCR through the regolith and undifferentiated fill
- Unit 3 A/B to SCR through the Glen Rose limestone
- Unit 4 A/B to SCR through the regolith and undifferentiated fill
- Units 4 A/B to SCR through the Glen Rose limestone

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In all four pathways, the location of the most limiting tank. the Boric Acid Tank, was the northeast corner of the Auxiliary Building. The four pathways represent the most conservative straight-line flow paths, or worse-case scenarios. The basis for selecting these pathway scenarios is discussed below. RCOL2_2.4.13-5

Due to the planned removal of all overburden material down to plant grade elevation of 822 ft msl, and the sub-grade floor elevation of the A/B at 785 ft msl, the pathways through the regolith and undifferentiated fill are not considered plausible and are not discussed further. Additionally, as discussed previously in Subsection 2.4.13.2, horizontal pathways through groundwater moving southeast or southwest are not considered plausible as this movement would be away from SCR and would not be consistent with the hydraulic gradients for the area surrounding the CPNPP Units 3 and 4 areas shown on Figure 2.4.12-210, Sheets 1 through 12. HYDSV-17 HYDSV-23 HYDSV-30

Actual groundwater flow from the postulated release point to SCR is expected to be tortuous and result in longer transport times. To define a conservative worse-case scenario, a simplified, straight-line pathway through the two media was utilized. This simplified approach was selected rather than simulating flow through a complex, three-dimensional flow path. The limestone in C-zone beneath the foundation is considered impermeable. Although groundwater was identified within the undifferentiated fill/regolith and bedrock beneath the CPNPP Units 3 and 4 sites, the groundwater was considered "perched" as evidenced by the lack of equilibrium in the groundwater monitoring wells. The four plausible pathways are presented in Table 2.4.12-211. Determination of the actual tortuous pathway utilizing a three-dimensional analysis would be less conservative than the theorized pathways through the undifferentiated fill/regolith or the shallow bedrock limestone. RCOL2_2.4.13-5 HYDSV-17 HYDSV-23 HYDSV-30

To further add conservatism, the highest measured hydraulic conductivity and steepest measured gradient were used in the velocity calculations for transport time to SCR. Actual hydraulic conductivity would be variable along the actual groundwater pathways and would result in a lower effective hydraulic conductivity for the groundwater flow path. The four groundwater pathways and the calculated travel times are presented on Figure 2.4.12-212 and cross section Figures 2.4.12-213 and 2.4.12-214.

To estimate groundwater travel time through the Glen Rose Formation, the site-specific porosity of limestone of 0.119 (see Subsection 2.4.12.2.5.1 for a discussion on selection of this conservative porosity), the highest hydraulic conductivity measured at the site (see Subsection 2.4.12.3 and Table 2.4.12-211), 1.37 X 10^{-5} cm/s, and the steepest hydraulic gradient measure from the monthly gauging events of the nearest groundwater monitoring wells to the Units 3 and 4 Reactor Buildings were used for the pathway analysis. HYDSV-17 HYDSV-23 HYDSV-30

- Pathway 3a (Figure 2.4.12-212 and Cross Section Figure 2.4.12-213) – the instantaneous release of the source term from the BAT in the northeast corner of A/B at elevation 785 ft msl traveling northeast towards SCR through the Glen Rose Formation limestone at this depth for 100 lateral feet would encounter engineered fill material at the Unit 3 Ultimate Heat Sink (UHS) and post-construction fill before reaching SCR. Since the engineering fill material design properties may change as the design is finalized and the potential exists for groundwater flow through the fill material of the Unit 3 UHS, it is conservatively assumed that the liquid effluent is instantaneously released to SCR at the time it encounters the engineered fill material at the SE corner of the Unit 3 UHS. The travel time from the Unit 3 A/B through a minimum of 100 feet Glen Rose Formation at this depth to the SE corner of Unit 3 UHS is 3146 days or 8.62 years.

- Pathway 3b (Figure 2.4.12-212 and Cross Section Figure 2.4.12-213) – the instantaneous release of the source term from the BAT in the northeast corner of the Unit 3 A/B at elevation 785 ft msl traveling due east towards the Unit 3 Reactor Building (RB) towards SCR through a minimum of 80 lateral feet of Glen Rose Formation limestone followed by the Unit 3 Essential Service Water (ESW) Pipe Tunnel and an undetermined lateral distance of Glen Rose Formation limestone followed by post-construction engineering fill and undifferentiated fill material before reaching SCR. Since the engineering fill material design properties may change as the design is finalized and the potential exists for groundwater flow through the fill material of the Unit 3 ESW Pipe Tunnel, it is conservatively assumed that the liquid effluent is instantaneously released to SCR at the time it encounters the engineered fill at the Unit 3 ESW Pipe Tunnel. The travel time from the Unit 3 A/B through a minimum of 80 feet of Glen Rose Formation limestone at this depth to the Unit 3 ESW Pipe Tunnel is 2516 days or 6.89 years.

- Pathway 4a (Figure 2.4.12-212 and Cross Section Figure 2.4.12-214) – the instantaneous release of the source term from the BAT in the northeast corner of Unit 4 A/B at elevation 785 ft msl traveling north-northwest towards SCR at this depth through a minimum of 60 lateral feet of Glen Rose Formation limestone where it would encounter engineered fill at the Unit 4 UHS and engineered fill before reaching SCR. Since the engineering fill material design properties may change as the design is finalized, and the potential exists for groundwater flow through the engineered fill material of the Unit 4 UHS, it is conservatively assumed that the liquid effluent is instantaneously released to SCR at the time it encounters the engineered fill at the Unit 4 UHS. The conservative travel time from the NE corner of the Unit 4 A/B through a minimum of 60 feet of

Glen Rose Formation limestone to the Unit 4 UHS is 1916 days or 5.25 years. HYDSV-17 HYDSV-23 HYDSV-30

- Pathway 4b (Figure 2.4.12-212 and Cross Section Figure 2.4.12-214) – the instantaneous release of the source term from the BAT in the northeast corner of Unit 4 A/B at elevation 785 ft msl traveling northeast towards SCR at this depth through a minimum of 100 lateral feet of Glen Rose Formation limestone where it would encounter engineered fill at the Unit 4 UHS and undocumented fill and engineered fill before reaching SCR. Since the engineering fill material design properties may change as the design is finalized, and the potential exists for groundwater flow through the engineered fill material of the Unit 4 UHS and the undocumented fill, it is conservatively assumed that the liquid effluent is instantaneously released to SCR at the time it encounters the engineered fill at the Unit 4 UHS. The travel time from the Unit 4 A/B through a minimum of 100 feet of Glen Rose Formation limestone to the Unit 4 UHS is 3834 days or 10.50 years.

In all plausible groundwater pathways identified, it was considerably conservative to assume a straight-line flow path to SCR with an instantaneous release of the liquid effluent to the SCR once it encountered the engineered fill at either the UHS or the UHS pipe tunnels. The actual groundwater pathways are expected to be tortuous, resulting in longer transport times, and the hydraulic conductivities of the fractures/joints would be or are expected to be lower than the highest measured on site.

2.4.13.6 Dilution Effects of Horizontal Liquid Effluent Release Pathway RCOL2_2.4.13-1

The computer code model utilized in the tank failure was the RATAF computer code for pressurized water reactors that is provided in NUREG-0133. The RATAF code defines the Hydrological Travel time as the time it takes for the liquid waste of a failed tank to reach the nearest potable water supply or nearest surface water in an unrestricted area.

The tank failure analysis, as described in DCD Subsection 11.2.3.2, was performed in accordance with Standard Review Plan (SRP) 2.4.13 and takes no credit for the dilution effects of groundwater nor retention or retardation in the regolith, undifferentiated fill, or the Glen Rose Formation. Because there is no “unrestricted” potable water supply or surface water body in close proximity to the Comanche Peak site, the analysis was conservatively performed by considering the potential for the liquid radioactive release to reach either the Unit 1 and 2 restricted potable water supply wells or Squaw Creek Reservior (SCR). The vertical pathway to the Twin Mountains formation, where the Unit 1 and 2 potable water supplies exist, was eliminated from consideration. The horizontal pathway through the regolith/undifferentiated fill and shallow bedrock was assumed to be a straight line to SCR. In reality, actual grounwater flow from the postulated release point to SCR would be more tortuous, resulting in longer transport times. RCOL2_2.4.13-1
Therefore, a simplified, straight-line pathway through the two media identified is a

more conservative, worse-case scenario than simulating flow through a complex, three-dimensional flow path. The A-zone undifferentiated fill or regolith, and the B-zone shallow bedrock geologic hydrogeologic characteristics indicate that the liquid release will not concentrate in these zones. It is conservatively assumed that the liquid release would travel with the groundwater through the impermeable limestone to SCR. RCOL2_2.4.13-1

The BTP 11-6 tank failure analysis used an equivalent volume of water reported in SCR of 4.4 x 10^{10} gallons. This same dilution volume was used in the Units 1 and 2 Standard Review Plan (SRP) 2.4.13 and 10 CFR 100.20(c)(3) assessment. Additionally, it was conservatively assumed that the travel time to the SCR was 365 days. It was also assumed that there would be no retardation or retention by the subsurface strata, and that groundwater would not dilute the released liquid radioactive waste. There will be no concentration of the release because there is no credible mechanism in these subsurface strata. Therefore, liquid radioactive waste is expected to move slowly and not concentrate in the subsurface media. It should also be noted that no credit is taken in the tank failure analysis for retardation or retention in the subsurface media, or dilution in the groundwater.

2.4.13.7 Summary of Accidental Releases of Radioactive Liquid Effluent in Ground and Surface Waters RCOL2_2.4.13-7

The tank failure analysis described in the US-APWR DCD Subsection 11.2.3.2 was performed in accordance with Branch Technical Position (BTP) 11-6 for the CPNPP Units 3 and 4. The computer code model used in the BTP 11-6 analysis was performed utilizing the RATAF computer code for pressurized water reactors that is provided in NUREG-0133 entitled "Preparation of Radiological Effluent Technical Specification for Nuclear Power Plants". The RATAF code defines the Hydrological Travel time as the time it takes for the liquid waste of a failed tank to reach the nearest potable water supply or nearest surface water in an unrestricted area. Although the nearest potable water supply and the nearest surface water body are located in the restricted areas of the CPNPP site, the potable water supply wells for the CPNPP Units 1 and 3 and SCR, respectively, were conservatively considered in this evaluation.

The BTP 11-6 tank failure analysis used an equivalent volume of water reported in SCR of 4.4 x 10^{10} gallons. This same dilution volume was used in the Units 1 and 2 Standard Review Plan (SRP) 2.4.13 and 10 CFR 100.20(c)(3) assessments. Additionally, in the BTP 11-6 tank failure analysis, it was conservatively assumed that the travel time to SCR was 365 days, that there is no retardation or retention by the subsurface strata, and that the groundwater did not dilute the released liquid radioactive waste. In the tank failure analysis, the dilution effects of SCR were considered and the concentrations provided in US-APWR DCD Table 11.2-17 show the calculated concentrations based upon the conservative travel time to the SCR of 365 days, with the dilution effects associated with SCR. In this BTP 11-6 evaluation model, it was determined that the BAT contained the largest quantity and concentration of radionuclides that could possibly challenge the

10 CFR 20, Appendix B limits, and that 80 percent of the contents with a 0.12 percent fuel defect level would be delivered to the SCR. RCOL2_2.4.13-7

The BAT is located in the northeast (NE) corner of the A/B where the basemat is at an approximate elevation of 785 ft msl. Site specific hydrogeological data discussed in Subsection 2.4.12.1.1, core boring stratigrahy discussed in Subsection 2.5.4.3.1, and Units 1 and 2 FSAR Subsections 2.4.12 and 2.4.13 were then used to discuss whether the vertical travel path to the Twin Mountains Formation was credible and to evaluate the horizontal travel time of groundwater in the shallow limestone bedrock of the Glen Rose Formation. The Glen Rose Formation limestone is considered impermeable beneath the CPNPP site, and groundwater measured in this limestone is considered "perched". However, in order to evaluate the effects of a postulated vertical release to the Twin Mountains aquifer, a conservative mathematical model with simplifying assumptions was used to model the dispersion of a liquid release through the Glen Rose Formation limestone as described in the CPNPP Units 1 and 2 FSAR Section 2.4.12. The results of this simplified analysis indicate that only one radionuclide, Cs-137, would penetrate the entire 150 feet depth of the Glen Rose Formation limestone to reach the Twin Mountains aquifer and it would take 400 years. HYDSV-17 HYDSV-23 HYDSV-30 RCOL2_2.4.13-7

Based upon this evaluation, and the results of the geologic and hydrogeologic investigations conducted at the CPNPP site discussed in Subsection 2.5.4.3.1, vertical transport of the liquid radioactive release through the Glen Rose Formation limestone to the deeper Twin Mountains aquifer is not considered probable. As a result, the vertical travel path was eliminated. Estimated velocity and travel times were calculated based upon CPNPP site specific data where it was determined that it would take 3146 days or approximately 8.62 years for groundwater carrying the liquid effluent from Unit 3 to reach SCR. Estimated velocity and travel times were calculated for the groundwater carrying the liquid effluent from Unit 4 to SCR was 1916 or 5.25 years. Because vertical migration through the impermeable limestone is not probable, a straight-line flow pathway form the postulated release point to SCR was considered a worse-case scenario and used as the bounding condition for the CPNPP Units 3 and 4 site. Evaluation of the site-specific hydrogeological information (porosity, hydraulic conductivity, groundwater gradient, etc, including equations, assumptions and methods), it was determined that the most conservative time for a liquid release from either the Unit 3 or Unit 4 BAT in the NE corner of either the Unit 3 or Unit 4 A/B to travel horizontally through the Glen Rose Formation limestone to reach SCR was approximately 1916 days or 5.25 years.

Since the DCD Section 11.2.3.2 tank failure analysis conservatively chose a travel time of 365 days to reach SCR. The site-specific hydrogeologic data shows a travel time of approximately 1916 days or 5.25 years, no credit is taken for retardation or suspension in subsurface media, or dilution by the groundwater prior to reaching SCR. Therefore, it is concluded that the limits of 10 CFR 20, Appendix B are met for the BAT Cs-134 and Cs-137 liquid release, and the site-specific hydrogeology bounds the US-APWR DCD Section 11.2.3.2 tank failure release analysis assumptions for travel time and dilution effects of SCR. 10

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2.4-272 Jones, K.F., J.E. Friddell, S.F. Daly, and C.M. Vuyovich, "Severe Winter Weather in the Continental U.S. and Global Climate Cycles," U.S. Army Corps of Engineers TR-04-19, October 2004. HYDSV-15

CP COL 2.4(1)

Table 2.4.2-204
Peak Streamflow of Panter Branch near Tolar, Texas (USGS Station 08091700) 1966–1973

Water Year[a]	Date	Gage Height[b] (ft)	Discharge (cfs)
1966	4/29/1966	14.49	880
1967	5/20/1967	16.90	1650
1968	5/9/1968	21.70	3650
1969	5/7/1969	13.50	610
1970	10/11/1969	13.61	640
1971	7/29/1971	14.53	890
1972	9/16/1972	21.88	3750
1973	4/23/1973	17.72	1990
1974	10/30/1973	10.20	5

a) Water Year = October 1 to September 30

b) ~~No Datum Provided by USGS Data~~Datum = 883 ft msl NAVD88 HDYSV-02
(Reference 2.4-220)

CP COL 2.4(1)

Table 2.4.2-208
Resulting PMP Water Surface Elevation at Points of Discharge

HYDSV-06
HYDSV-07
HYDSV-06
HYDSV-07

Point Of Discharge	Drainage Sub Basins	Peak Runoff at Point of Discharge (cfs)	Crest Length L (ft)	Tailwater Elevation (ft msl)	Discharge Coefficient	Weir Elevation (ft msl)	Over Topping Depth Hw (ft)	Resulting ~~PMP~~ Water Surface Elevation (ft msl)
W1	1+2+3	1,195.44	560	~~788.9~~790.9	2.80	820	0.83	820.83
W2	4+5+6	1,166.70	365	~~788.9~~790.9	2.83	815	1.09	816.09
W3	7+8	2,384.49	490	~~788.9~~790.9	2.90	810	1.44	811.44
W4	9+10+11	4,127.68	315	~~788.9~~790.9	3.02	814	3.14	817.14

CP COL 2.4(1)

Table 2.4.3-202
Squaw Creek Watershed 6-hr Incremental PMP Estimates

Duration (hr)	Incremental PMP (in)
6	0.59
12	0.72
18	0.91
24	1.24
30	1.96
36	5.10
42	21.10
48	2.82
54	1.52
60	1.05
66	0.80
72	0.65
Total	38.46

Note: Values derived from HMR 51, HMR 52, and the use of HMR 52 computer software. The critical storm was determined to be 700 sq mi, with a ~~160~~145 degree storm orientation, centered near the centroid of the Squaw Creek watershed. HYDSV-06 HYDSV-07

CP COL 2.4(1)

Table 2.4.3-207
Watershed Subbasin Characteristics

HYDSV-06
HYDSV-07

Basin	Area (sq mi)	Baseflow (cfs)	L (mi)	Lca (mi)	$C_{\cancel{i}t}$	~~640~~C_p	Lag Time (hr)
Basin 1a & 1c	44.9	~~8.39~~8.86	13.7	6.5	1.1	~~440~~0.76	~~4.07~~1.91
Basin 1b	20.3	~~8.39~~8.86	5.3	2.5	0.6	~~420~~0.64	~~1.26~~1.08
Basin 2	10.65	~~1.51~~1.47	4.6	3.0	0.6	~~420~~0.64	~~1.45~~1.09
Basin 3	24.3	~~2.75~~3.35	4.9	5.3	0.6	~~420~~0.77	~~2.25~~1.35
Basin 4	410.0	~~8.35~~56.57	59.3	29.8	0.6	~~420~~0.80	~~5.69~~4.51

L = length of the main stream from outlet to basin divide

L_{ca} = length along the main stream from the outlet to a point nearest the watershed centroid

C_t & C_p values resulting in higher water surface elevation at the CPNPP Units 3 and 4 were used. HYDSV-06 HYDSV-07

~~CP COL 2.4(1)~~ HYDSV-23

Table 2.4.12-211 (Sheet 1 of 4)
Groundwater Velocity and Travel Times

~~Scenario 1 (Unit 3/MW-1217a to SCR)~~

~~Date~~	~~12/27~~	~~1/23~~	~~2/20~~	~~3/19~~	~~4/10~~	~~5/16~~	~~6/13~~	~~7/16~~	~~8/13~~	~~9/13~~	~~10/16~~	~~11/15~~
~~MW-1217a (ft amsl)~~	~~829.52~~	~~829.45~~	~~829.45~~	~~829.45~~	~~829.45~~	~~829.45~~	~~829.44~~	~~830.31~~	~~829.70~~	~~829.57~~	~~829.54~~	~~829.54~~
~~SCR (ft amsl)~~	~~775.23~~	~~775.42~~	~~775.19~~	~~775.00~~ [(a)]	~~775.36~~	~~775.39~~	~~775.31~~	~~775.33~~	~~775.40~~	~~775.46~~	~~775.48~~	~~775.38~~
~~Hydraulic Gradient~~	~~0.1020~~	~~0.1020~~	~~0.1020~~	~~0.1030~~	~~0.1020~~	~~0.1020~~	~~0.1020~~	~~0.1040~~	~~0.1020~~	~~0.1020~~	~~0.1020~~	~~0.1020~~
~~Velocity (V) (ft/day)~~	~~0.7260~~	~~0.7230~~	~~0.7260~~	~~0.7280~~	~~0.7230~~	~~0.7230~~	~~0.7240~~	~~0.7350~~	~~0.7260~~	~~0.7240~~	~~0.7230~~	~~0.7240~~
~~Travel Time (T) (days)~~	~~730.0~~	~~733.5~~	~~730.4~~	~~727.9~~	~~732.7~~	~~733.1~~	~~732.2~~	~~720.9~~	~~729.9~~	~~732.5~~	~~733.1~~	~~731.8~~

~~Scenario 2 (Unit 3/MW-1217b to SCR)~~

~~Date~~	~~12/27~~	~~1/23~~	~~2/20~~	~~3/19~~	~~4/10~~	~~5/16~~	~~6/13~~	~~7/16~~	~~8/13~~	~~9/13~~	~~10/16~~	~~11/15~~
~~MW-1217b (ft amsl)~~	~~810.94~~	~~820.76~~	~~824.72~~	~~825.06~~	~~823.82~~	~~820.08~~	~~820.38~~	~~821.13~~	~~822.28~~	~~823.83~~	~~825.64~~	~~827.00~~
~~SCR (ft amsl)~~	~~775.23~~	~~775.42~~	~~775.19~~	~~775.00~~ [(a)]	~~775.36~~	~~775.39~~	~~775.31~~	~~775.33~~	~~775.40~~	~~775.46~~	~~775.48~~	~~775.38~~
~~Hydraulic Gradient~~	~~0.0674~~	~~0.0855~~	~~0.0935~~	~~0.0945~~	~~0.0914~~	~~0.0843~~	~~0.0850~~	~~0.0864~~	~~0.0885~~	~~0.0913~~	~~0.0946~~	~~0.0974~~
~~Velocity (V) (ft/day)~~	~~0.0187~~	~~0.0237~~	~~0.0259~~	~~0.0262~~	~~0.0254~~	~~0.0234~~	~~0.0236~~	~~0.0240~~	~~0.0245~~	~~0.0253~~	~~0.0263~~	~~0.0270~~
~~Travel Time (T) (days)~~	~~28,354.1~~	~~22,331.8~~	~~20,442.7~~	~~20,226.2~~	~~20,894.1~~	~~22,656.7~~	~~22,465.6~~	~~22,107.6~~	~~21,598.2~~	~~20,932.9~~	~~20,185.9~~	~~19,615.0~~

~~CP COL 2.4(1)~~

HYDSV-23

Table 2.4.12-211 (Sheet 2 of 4)
Groundwater Velocity and Travel Times

~~Scenario 3 (Unit 4/MW-1215a to SCR)~~

~~Date~~	~~12/27~~	~~1/23~~	~~2/20~~	~~3/19~~	~~4/10~~	~~5/16~~	~~6/13~~	~~7/16~~	~~8/13~~	~~9/13~~	~~10/16~~	~~11/15~~
~~MW-1215a (ft amsl)~~	~~833.79~~	~~835.25~~	~~8325.93~~	~~836.21~~	~~837.27~~	~~837.26~~	~~839.70~~	~~841.18~~	~~841.41~~	~~841.89~~	~~841.81~~	~~841.42~~
~~SCR (ft amsl)~~	~~775.23~~	~~775.42~~	~~775.19~~	~~775.00 [(a)]~~	~~775.36~~	~~775.39~~	~~775.31~~	~~775.33~~	~~775.40~~	~~775.46~~	~~775.48~~	~~775.38~~
~~Hydraulic Gradient~~	~~0.0965~~	~~0.0986~~	~~0.1000~~	~~0.1010~~	~~0.1020~~	~~0.1020~~	~~0.1060~~	~~0.1080~~	~~0.1090~~	~~0.1090~~	~~0.1090~~	~~0.1090~~
~~Velocity (V) (ft/day)~~	~~0.6840~~	~~0.6990~~	~~0.7090~~	~~0.7150~~	~~0.7230~~	~~0.7220~~	~~0.7520~~	~~0.7690~~	~~0.7710~~	~~0.7760~~	~~0.7740~~	~~0.7710~~
~~Travel Time (T) (days)~~	~~887.7~~	~~868.9~~	~~855.9~~	~~849.3~~	~~839.7~~	~~840.2~~	~~807.4~~	~~789.5~~	~~787.5~~	~~782.6~~	~~783.7~~	~~787.2~~

~~Scenario 4 (Unit 4/MW-1215b to SCR)~~

~~Date~~	~~12/27~~	~~1/23~~	~~2/20~~	~~3/19~~	~~4/10~~	~~5/16~~	~~6/13~~	~~7/16~~	~~8/13~~	~~9/13~~	~~10/16~~	~~11/15~~
~~MW-1215b (ft amsl)~~	~~831.35~~	~~831.27~~	~~831.64~~	~~831.60~~	~~832.10~~	~~831.80~~	~~832.91~~	~~833.74~~	~~833.55~~	~~833.54~~	~~833.84~~	~~833.12~~
~~SCR (ft amsl)~~	~~775.23~~	~~775.42~~	~~775.19~~	~~775.00 [(a)]~~	~~775.36~~	~~775.39~~	~~775.31~~	~~775.33~~	~~775.40~~	~~775.46~~	~~775.48~~	~~775.38~~
~~Hydraulic Gradient~~	~~0.0925~~	~~0.0920~~	~~0.0930~~	~~0.0932~~	~~0.0935~~	~~0.0929~~	~~0.0949~~	~~0.0962~~	~~0.0958~~	~~0.0957~~	~~0.0961~~	~~0.0961~~
~~Velocity (V) (ft/day)~~	~~0.0256~~	~~0.0255~~	~~0.0258~~	~~0.0259~~	~~0.0259~~	~~0.0258~~	~~0.0263~~	~~0.0267~~	~~0.0266~~	~~0.0265~~	~~0.0267~~	~~2.64E-02~~
~~Travel Time (T) (days)~~	~~23,665.4~~	~~23,779.8~~	~~23,527.1~~	~~23,464.7~~	~~23,406.8~~	~~23,543.8~~	~~23,057.3~~	~~22,737.6~~	~~22,839.3~~	~~22,866.8~~	~~22,757.1~~	~~23,001.4~~

~~CP COL 2.4(1)~~

Table 2.4.12-211 (Sheet 3 of 4)
Groundwater Velocity and Travel Times

HYDSV-23

~~Assumptions:~~

~~Scenario 1~~

~~Hydraulic gradient is between Unit 3/MW-1217a and SCR~~

~~Pathway Distance (L) = 530 ft.~~

~~Hydraulic Conductivity~~ ~~(K_{η}) = 5.00 x 10^{-4} cm/s~~

~~porosity (η) = 0.20~~

~~Scenario 2~~

~~Hydraulic gradient is between Unit 3/MW-1217b and SCR~~

~~Pathway Distance (L) = 530 ft.~~

~~Hydraulic Conductivity~~ ~~(K_{η}) = 1.37 x 10^{-5} cm/s~~

~~porosity (η) = 0.14~~

~~Scenario 3~~

~~Hydraulic gradient is between Unit 4/MW-1215a and SCR~~

~~Pathway Distance (L) = 607 ft.~~

~~Hydraulic Conductivity~~ ~~(K_{η}) = 5.00 x 10^{-4} cm/s~~

~~porosity (η) = 0.20~~

~~CP COL 2.4(1)~~

Table 2.4.12-211 (Sheet 4 of 4)
Groundwater Velocity and Travel Times

HYDSV-23

~~Scenario 4~~

~~Hydraulic gradient is between Unit 4/MW-1215b and SCR~~

~~Pathway Distance (L) = 607 ft.~~

~~Hydraulic Conductivity (K_{η}) = 1.37 x 10^{-5} cm/s~~

~~porosity (η) = 0.14~~

---

a) ~~775.00 ft was used as surface water elevation for SCR on 3/19 as USGS elevation data was unavailable.~~

~~Conversions: 1 day = 86,400 seconds; 1 foot = 30.48 centimeters.~~

Table 2.4.12-211 (Sheet 1 of 4)
Groundwater Velocity and Travel Times

HYDSV-23

Scenario 1, Pathway 3a (Unit 3/MW-1217a to SCR)

Date	12/27	1/23	2/20	3/19	4/10	5/16	6/13	7/16	8/13	9/13	10/16	11/15
MW-1217a (ft amsl)	810.94	820.76	824.72	825.06	823.82	820.08	820.38	821.13	822.28	823.83	825.64	827.00
SCR (ft amsl)	775.23	775.42	775.19	775.00 [(a)]	775.36	775.39	775.31	775.33	775.40	775.46	775.48	775.38
Hydraulic Gradient	0.0674	0.0855	0.0935	0.0945	0.0914	0.0843	0.0850	0.0864	0.0885	0.0913	0.0946	0.0974
Velocity (V) (ft/day)	0.0220	0.0279	0.0305	0.0308	0.0298	0.0275	0.0277	0.0282	0.0289	0.0298	0.0308	0.0318
Travel Time (T) (days)	4,550	3,587	3,280	3,246	3,356	3,638	3,608	3,550	3,466	3,359	3,242	3,149

Scenario 1, Pathway 3b (Unit 3/MW-1217b to SCR)

Date	12/27	1/23	2/20	3/19	4/10	5/16	6/13	7/16	8/13	9/13	10/16	11/15
MW-1217b (ft amsl)	810.94	820.76	824.72	825.06	823.82	820.08	820.38	821.13	822.28	823.83	825.64	827.00
SCR (ft amsl)	775.23	775.42	775.19	775.00 [(a)]	775.36	775.39	775.31	775.33	775.40	775.46	775.48	775.38
Hydraulic Gradient	0.0674	0.0855	0.0935	0.0945	0.0914	0.0843	0.0850	0.0864	0.0885	0.0913	0.0946	0.0974
Velocity (V) (ft/day)	0.0220	0.0279	0.0305	0.0308	0.0298	0.0275	0.0277	0.0282	0.0289	0.0298	0.0308	0.0318
Travel Time (T) (days)	3,640	2,870	2,624	2,596	2,684	2,911	2,887	2,840	2,772	2,687	2,594	2,519

Table 2.4.12-211 (Sheet 2 of 4)
Groundwater Velocity and Travel Times

HYDSV-23

Scenario 2, Pathway 4a (Unit 4/MW-1215a to SCR)

Date	12/27	1/23	2/20	3/19	4/10	5/16	6/13	7/16	8/13	9/13	10/16	11/15
MW-1215a (ft amsl)	831.35	831.27	831.64	831.60	832.10	831.80	832.91	833.74	833.55	833.54	833.84	833.12
SCR (ft amsl)	775.23	775.42	775.19	775.00 [(a)]	775.36	775.39	775.31	775.33	775.40	775.46	775.48	775.38
Hydraulic Gradient	0.0925	0.0920	0.0930	0.0932	0.0935	0.0929	0.0949	0.0962	0.0958	0.0957	0.0961	0.0961
Velocity (V) (ft/day)	0.0302	0.0300	0.0303	0.0304	0.0305	0.0303	0.0309	0.0314	0.0312	0.0312	0.0313	0.0313
Travel Time (T) (days)	1,989	2,000	1,979	1,974	1,968	1,981	1,939	1,913	1,921	1,923	1,915	1,915

Scenario 2, Pathway 4b (Unit 4/MW-1215b to SCR)

Date	12/27	1/23	2/20	3/19	4/10	5/16	6/13	7/16	8/13	9/13	10/16	11/15
MW-1215b (ft amsl)	831.35	831.27	831.64	831.60	832.10	831.80	832.91	833.74	833.55	833.54	833.84	833.12
SCR (ft amsl)	775.23	775.42	775.19	775.00 [(a)]	775.36	775.39	775.31	775.33	775.40	775.46	775.48	775.38
Hydraulic Gradient	0.0925	0.0920	0.0930	0.0932	0.0935	0.0929	0.0949	0.0962	0.0958	0.0957	0.0961	0.0961
Velocity (V) (ft/day)	0.0302	0.0300	0.0303	0.0304	0.0305	0.0303	0.0309	0.0314	0.0312	0.0312	0.0313	0.0313
Travel Time (T) (days)	3,979	4,000	3,957	3,949	3,936	3,962	3,878	3,826	3,842	3,846	3,830	3,830

Table 2.4.12-211 (Sheet 3 of 4)
Groundwater Velocity and Travel Times

HYDSV-23

Assumptions:

Scenario 1, Pathway 3a

The hydraulic gradient between MW-1217b and SCR is the nearest known hydraulic gradient to the Unit 3 A/B. The highest hydraulic gradient between MW-1217B and SCR was used for this pathway.

Pathway Distance (L) = 100 lateral feet of Glen Rose Formation

Hydraulic Conductivity (K_η) = 1.37 x 10^{-5} cm/s = 0.0388 ft/day

porosity (η) = 0.119

Scenario 1, Pathway 3b

The hydraulic gradient between MW-1217b and SCR is the nearest known hydraulic gradient to the Unit 3 A/B. The highest hydraulic gradient between MW-1217B and SCR was used for this pathway.

Pathway Distance (L) = 80 lateral feet of Glen Rose Formation

Hydraulic Conductivity (K_η) = 1.37 x 10^{-5} cm/s = 0.0388 ft/day

porosity (η) = 0.119

Scenario 2, Pathway 4a

The hydraulic gradient between MW-1215b and SCR is the nearest known hydraulic gradient to the Unit 4 A/B. The highest hydraulic gradient between MW-1215B and SCR was used for this pathway

Pathway Distance (L) = 60 lateral feet of Glen Rose Formation

Hydraulic Conductivity (K_η) = 1.37 x 10^{-5} cm/s = 0.0388 ft/day

Table 2.4.12-211 (Sheet 4 of 4)
Groundwater Velocity and Travel Times

porosity (η) = 0.119 HYDSV-23

Scenario 2, Pathway 4b

The hydraulic gradient between MW-1215b and SCR is the nearest known hydraulic gradient to the Unit 4 A/B. The highest hydraulic gradient between MW-1215B and SCR was used for this pathway

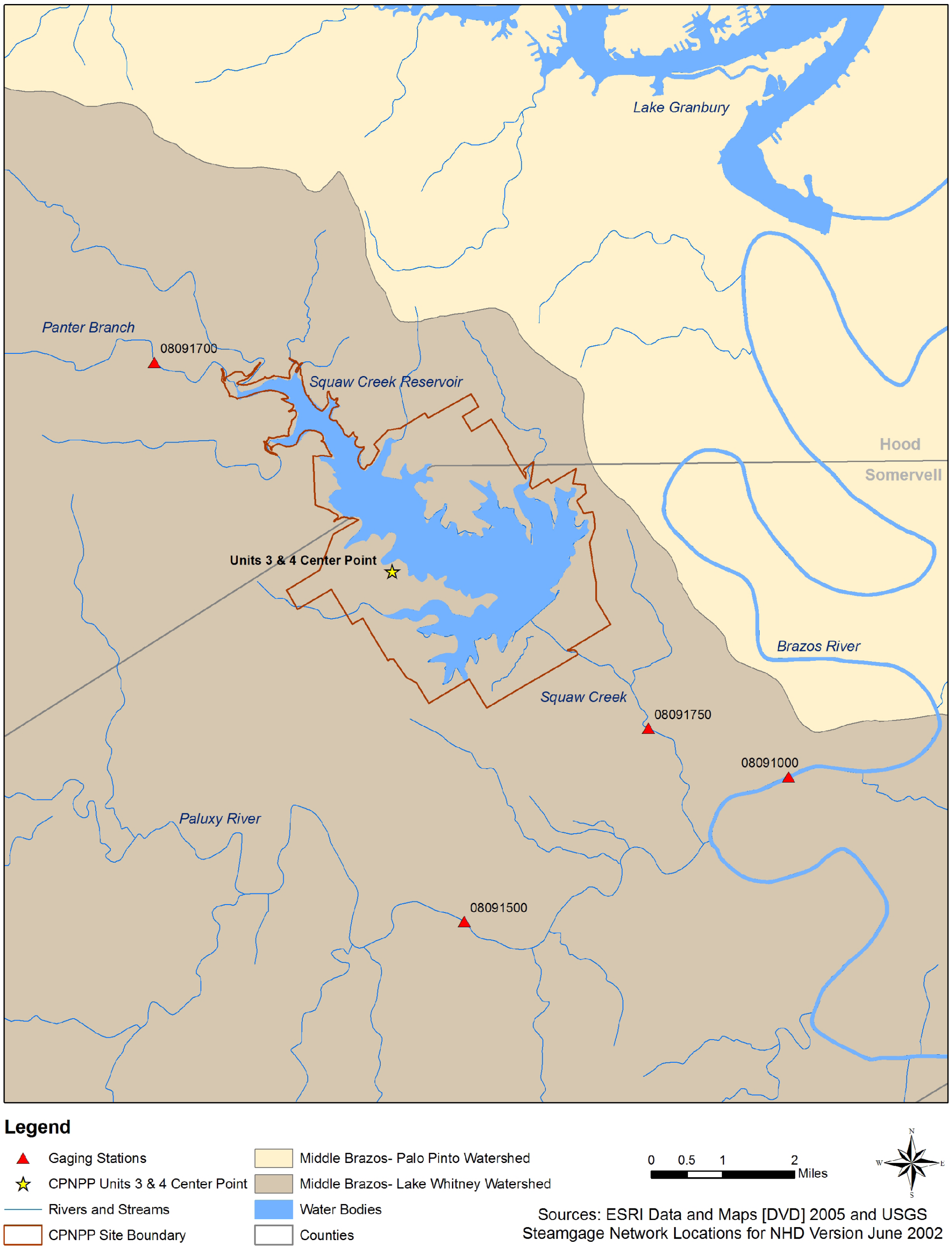
Pathway Distance (L) = 120 lateral feet of Glen Rose Formation

Hydraulic Conductivity (K_η) = 1.37 x 10-5 cm/s = 0.0388 ft/day

porosity (η) = 0.119

(a) - 775.00 ft was used as surface water elevation for SCR on 3/19 as USGS elevation data was unavailable

Conversions: 1day = 86,400 seconds; 1 foot = 30.48 centimeters



HYDSV-02

Figure 2.4.2-201 USGS Gage Locations

HYDSV-02

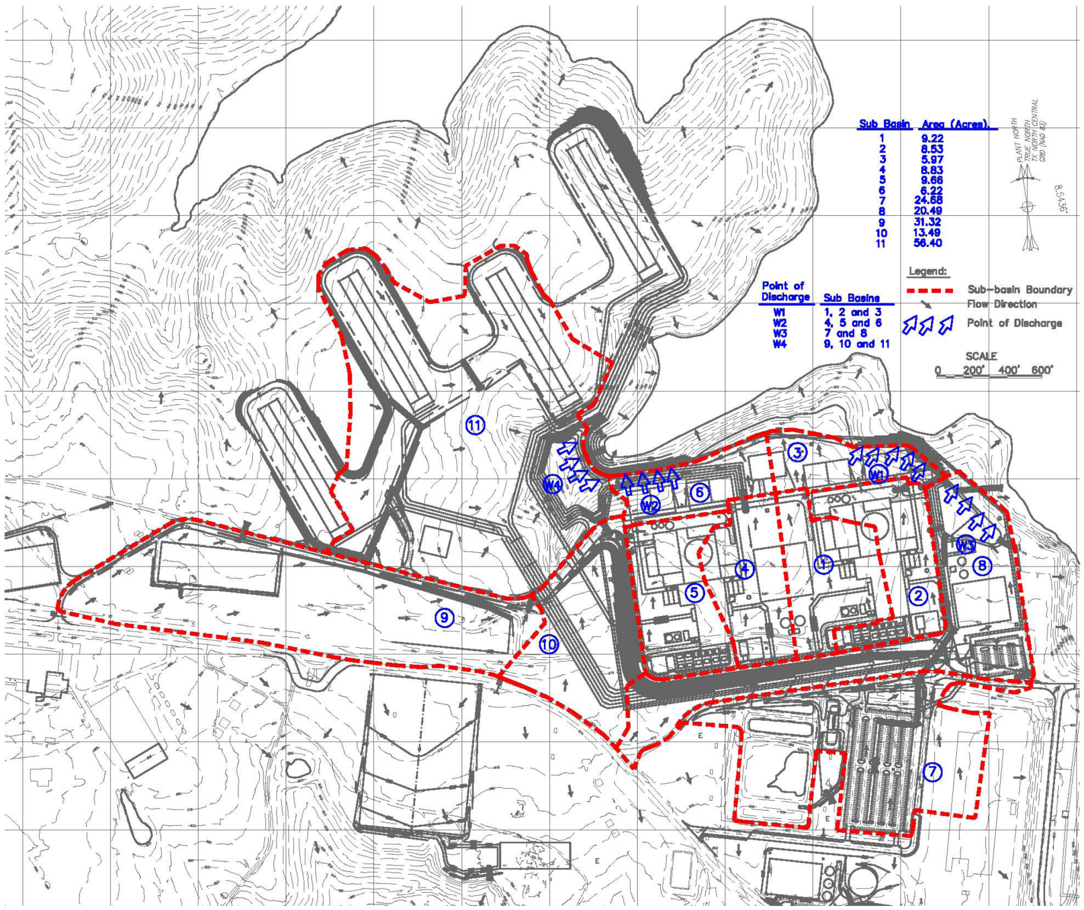


Figure 2.4.2-202 Site Grading and Drainage Plan

HYDSV-02

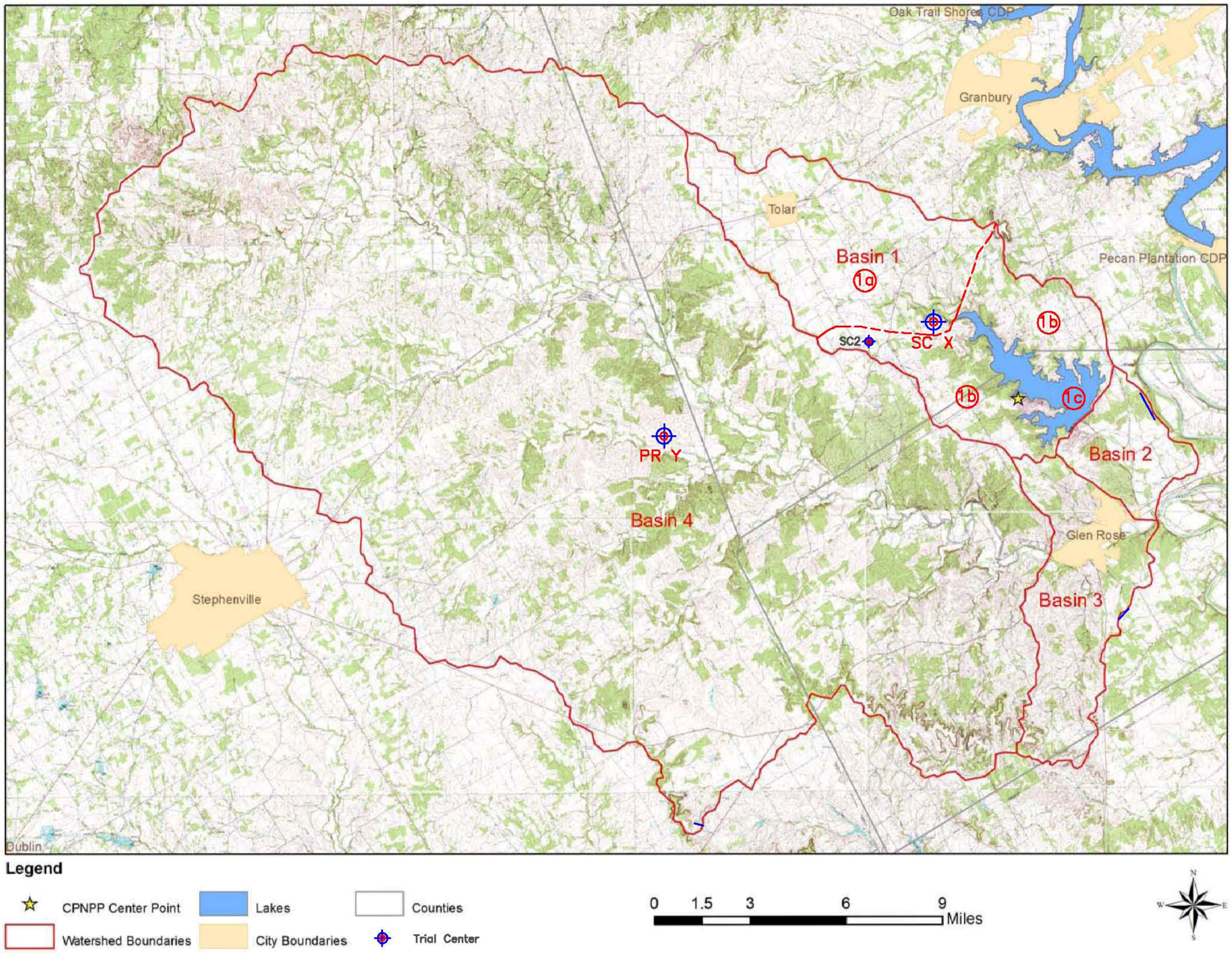
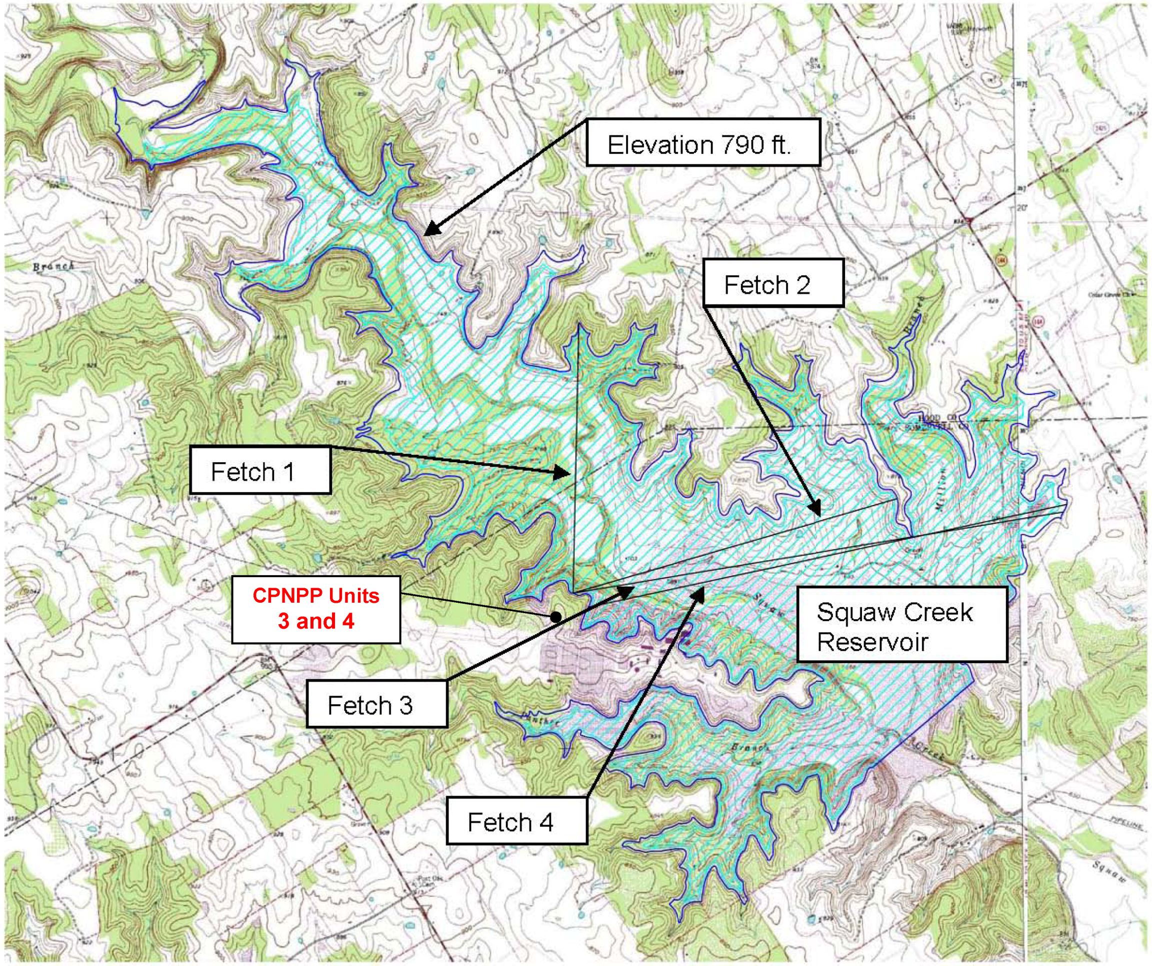
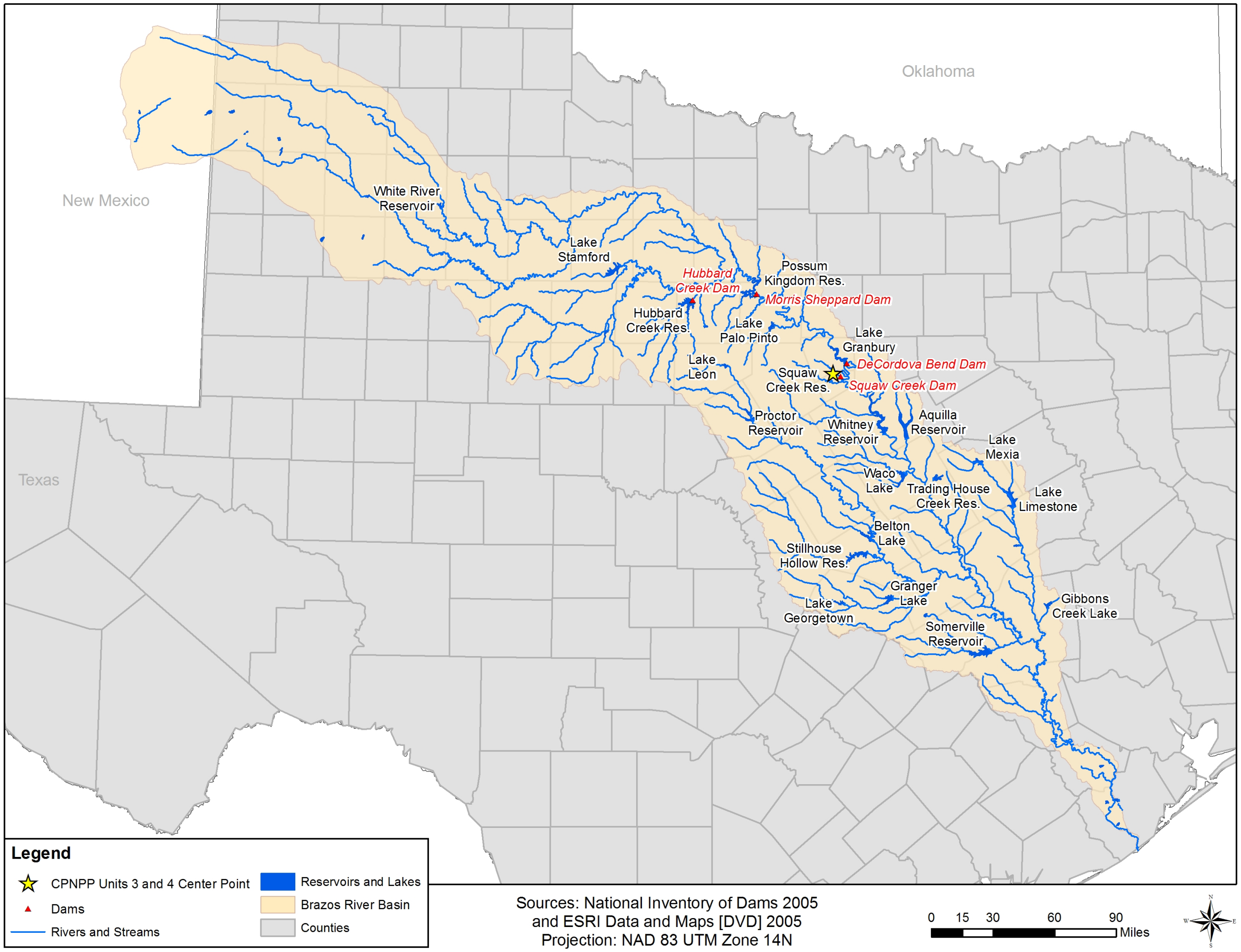


Figure 2.4.3-202 Watershed Sub-basins



HYDSV-02

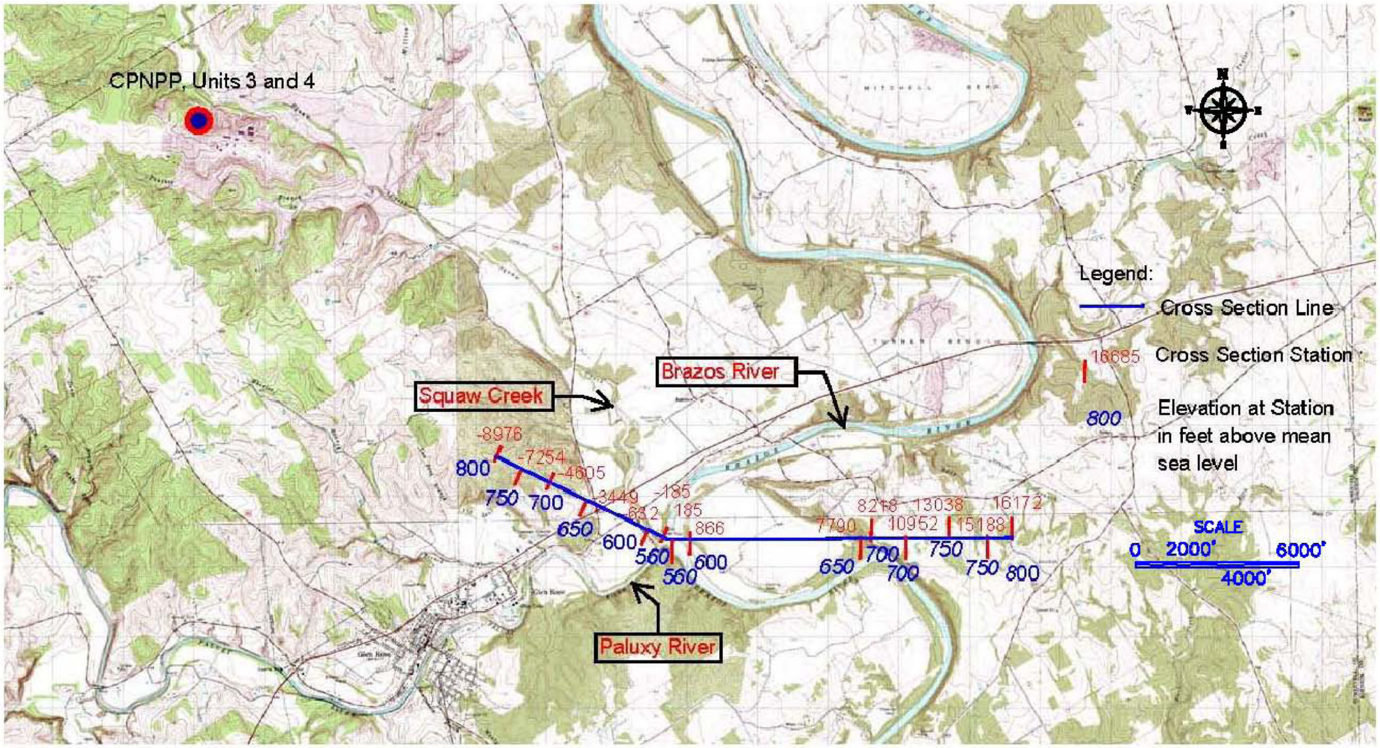
Figure 2.4.3-209 Coincident Wind Wave Activity Fetch Length



HYDSV-02

Figure 2.4.4-201 Brazos River Watershed

HYDSV-02



Source: All Topo Maps for Texas using 1:24,000 Scale 7.5 minute USGS Topographic Maps

Horizontal Datum - NAD27
Vertical Datum - NGVD29

Figure 2.4.4-202 Brazos River - Paluxy River Confluence Cross Section