Table 2.0-1R (Sheet 8 of 13)
Key Site Parameters

CP COL 2.3(2)	Steam line break releases[8] 0-8 hrs 8-24 hrs 1-4 days 4-30 days	 1.9×10^{-2} s/m^3 1.1×10^{-2} s/m^3 7.1×10^{-3} s/m^3 4.7×10^{-3} s/m^3	See steam line break releases to Main Control Room intake (above)[13]
	Fuel handling area releases[7] 0-8 hrs 8-24 hrs 1-4 days 4-30 days	 1.1×10^{-3} s/m^3 6.7×10^{-4} s/m^3 4.3×10^{-4} s/m^3 2.8×10^{-4} s/m^3	See fuel handling area releases to Main Control Room intake (above)[13]
	Hydrologic Engineering		
CP COL 2.4(1)	**Parameter Description**	**Parameter Value**	
		DCD	**CPNPP 3 and 4**
RCOL2_02.04.02-2 S03	Maximum flood (or tsunami) level	1 ft below plant grade	~~793.66 ft msl for SCR~~ ~~820.98 ft msl for a Local Intense Precipitation at units 3 and 4 site.~~CPNPP Units 3 and 4 plant grade - 822 ft NAVD 88 PMF - 794.09 ft NAVD 88 PMF with coincident wind waves - 811.09 ft NAVD 88 Local Intense Precipitation - 820.93 ft NAVD 88
	Maximum rainfall rate (hourly)	19.4 in/hr for seismic category I and II structures	19.~~0~~1 in/hr
	Maximum rainfall rate (short-term)	6.3 in/5 min for seismic category I and II structures	6.~~2~~3 in/5 min

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 NGVD29.

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 are provided in Table 2.4.2-203. The datum for USGS gage (08091500) is reported in NAD27 and NGVD29.

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. The maximum recorded 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 NAVD 88. RCOL2_02.04.02-2 S03

2.4.2.2 Flood Design Considerations

By examination of the vicinity of CPNPP Units 3 and 4 site and area topography, it was determined that the ~~flooding potential~~maximum water surface elevation at the site would originate from local intense precipitation, the adjacent SCR, or the Brazos River and the Squaw Creek or the Paluxy River tributaries. Squaw Creek joins the Paluxy River just below SCR. The Paluxy River joins the Brazos River just below the junction with Squaw Creek. In addition, coincident wind wave activity is considered. RCOL2_02.04.02-2 S03

The local intense precipitation analysis is approached conservatively. The precipitation selected is the point PMP at the most critical temporal distribution and assumed to apply to the entire site. No losses are assumed. All rainfall is converted to runoff. Conservative estimates for roughness coefficients are utilized in the determination of peak flows. Downstream boundary conditions are based on the maximum water surface elevation for SCR and account for datum conversion.

The SCR flooding analysis is approached conservatively. The PMP is maximized for SCR watershed using the critical storm center, orientation, and temporal distribution. No losses are assumed. All rainfall is converted to runoff. Baseflow is determined based on the maximum average monthly flow for a nearby stream gage. The most recent storage elevation relationship for SCR is utilized. The spillway rating curves are derived to be more conservative than the published elevation discharge curves. The service spillway is evaluated assuming a flooded tailwater condition. The emergency spillway discharge is based on downstream channel flow depth at 90 to 100 percent of the headwater elevation.

Snyder's unit hydrographs are derived based on maximizing the peaking coefficient and minimizing the lag time coefficient. The peak of the unit hydrographs is increased by 20 percent and the time to peak is reduced by 33 percent to account for the effects of nonlinear basin response. A backwater analysis downstream of Squaw Creek Dam to determine tailwater effects is performed by maximizing the flow from adjacent watersheds in conjunction with the maximum downstream elevation on the Brazos River. Conservative estimates for roughness coefficients are utilized. RCOL2_02.04.02-2 S03

The Brazos River flooding analysis is approached conservatively and considers failure of upstream dams under existing and proposed conditions. Upstream tributary dams are assumed to fail under the probable maximum flood (PMF) for the tributary dam's watershed. Dams are assumed to fail in a domino-type manner or simultaneous as applicable to determine maximum downstream effects. No attenuation is assumed and dam failure results are transposed downstream instantaneously. When considering failure of the Brazos River dams, the dam failure results that include the PMF for the tributary dams are combined with the PMF for the Brazos River, which also includes the drainage area for the tributary dams.

Antecedent reservoir elevations are based on maximum recorded elevations or higher crest elevations. Wind setup is included to maximize water surface elevations. Conservative breach parameters are utilized. Breach wave heights and breach flows are evaluated to determine the maximum downstream effects. Although tailwater is considered, conservative roughness coefficients are used to minimize the tailwater effect on the breach wave heights and breach flows, which are dependent on the difference between the headwater elevation and the tailwater elevation. In the vicinity of the site, the Brazos River has been incorporated into the stream course model utilized for the backwater analysis. Conservative roughness coefficients are utilized to maximize the resulting water surface elevation. Datum conversion is accounted for in the comparison to the site grade.

The coincident wind wave activity analysis is approached conservatively. A straight line fetch is assumed instead of using an effective fetch. The maximum PMF elevation of SCR is used to determine the maximum fetch length. The maximum appropriate wind speed for the area is used. Wind setup is included in the analysis. Runup is evaluated for slopes from 10:1 to vertical. Datum conversion is accounted for in the comparison to the site grade.

The summary results of the events evaluated to determine the worst potential flood are provided as follows:

- 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 ~~793.66 ft msl~~794.09 ft NAVD 88 (discussed in Subsection 2.4.3). RCOL2_02.04.02-2 S03
- 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 ~~760.68 ft msl~~768.69 ft NAVD 88 (discussed in Subsection 2.4.4). RCOL2_02.04.02-2 S03
- Two year coincident wind waves with a corresponding water surface level of ~~810.64 ft msl~~811.09 ft NAVD 88 (discussed in Subsection 2.4.3). RCOL2_02.04.02-2 S03

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~~water surface elevation at CPNPP Units 3 and 4 due to PMF is elevation ~~793.66 ft msl~~794.09 ft NAVD 88. This elevation would result from a PMP at CPNPP Units 3 and 4 on the Squaw Creek watershed, as described in Subsection 2.4.3. Coincident wind waves would create maximum waves of ~~16.98~~17 ft resulting in a design basis flood elevation of ~~810.64 ft msl~~811.09 ft NAVD 88 due to PMF. CPNPP Units 3 and 4 safety-related plant elevation is 822 ft ~~msl~~NAVD 88, providing more than 1~~1~~0 ft of freeboard~~ under the worst potential flood considerations~~. RCOL2_02.04.02-2 S03 RCOL2_02.04.02-2 S03

The maximum water surface elevation at CPNPP Units 3 and 4 due to local intense precipitation is 820.93 ft NAVD 88. This elevation would result from a local intense PMP, as described in Subsection 2.4.2.3. The design basis flood elevation due to local intense precipitation is 820.93 ft NAVD 88, providing more than 1 ft of freeboard.

2.4.2.3 Effects of Local Intense Precipitation

The effects of local intense PMP at CPNPP Units 3 and 4 are evaluated by performing a site drainage analysis based on the guidelines provided in Regulatory Guide 1.59 and ANSI/ANS-2.8-1992 (Reference 2.4-229) including determination of the maximum water level associated with potential flooding resulting from a storm producing the PMP on the local area. The site is graded such that overall runoff drains away from safety-related structures to onsite ponds and ultimately the SCR. The PMP flood analysis assumes that all subsurface discharge structures at the site are non-functional or completely blocked. However, in certain cases it is more conservative to allow flow through a drainage structure to affect an adjoining area that flow would not reach if the drainage structure was non-functional. In those cases, both scenarios are examined to determine the worst case flooding. Computed water surface elevations in the vicinity of safety-related structures do not exceed 1 ft below site grade elevation of 822 ft. The site grading and drainage plan is shown in Figure 2.4.2-202.

To analyze the effects of local intense PMP, the site is divided into 32 drainage areas based on the contours shown in the grading and drainage plan as shown in Figure 2.4.2-202. The main channels directing flow away from Units 3 and 4 are identified as the West Channel, the Center North Channel, which empties into the channels north of the ultimate heat sink (UHS) structures, and the Unit 3 Southeast Channel. Other channels carry flow to the main channels and may affect Units 3 and 4, or carry flow from the main channels to pond areas, establishing downstream boundary conditions. The East Channel and Offsite Channel do not directly affect Units 3 and 4, but are analyzed for the potential of adding flow to the adjacent Unit 3 Southeast Channel. USACE HEC-RAS, Version 4.1.0 (Reference 2.4-300), modeling software is used to route the flow through the channels. HEC-RAS channel locations are shown in Figure 2.4.2-206.

Drainage Area 23 is a large area to the west of Units 3 and 4. Runoff from this area is directed to several culverts, some of which would normally carry runoff away from the designated drainage areas. Assuming culverts are non-functional would result in roadway overtopping. It is likely that the roadway overtopping would also carry much of the runoff away from the designated drainage areas. However, it is conservatively assumed that all runoff from Drainage Area 23 enters Drainage Area 24 and Drainage Pond A.

Although Drainage Areas 24 and 32 are fairly large, it is conservatively assumed that the rainfall is transformed to runoff using the most intense rate of local intense PMP. It would be more appropriate for a large drainage area to use a longer response time and the corresponding lesser intensity. All runoff from Drainage Area 24 directly enters Drainage Pond A. All runoff from Drainage Area 32 is directed to the Offsite Channel and directly enters Drainage Pond C. ~~Runoff then enters Drainage Pond B through a culvert structure or overflows an embankment directly to the SCR.~~ RCOL2_02.04.02-2 S03

Runoff flowing north between the two units flows to Drainage Pond A via a culvert located within Drainage Area 6. Because the culvert is assumed to be non-functional, runoff overtops the loop road and may then either flow west to Drainage Pond A or east to Drainage Pond Area B. In order to reach either Drainage Pond, runoff would normally be carried through drainage structures. Assuming the drainage structures are non-functional would cause runoff to spill over retaining walls directly to the SCR.

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.

HMR 52 guidance indicates that PMP rates for 10-sq mi areas are the same as point rainfall. Also indicated in HMR 52, the 1-sq mi PMP rates may also be considered the point rainfall for areas less than 1-sq mi. Therefore, intensities for any drainage areas with durations longer than 1-hr. are derived from the PMP rates for 10-sq mi areas. Intensities for drainage areas with durations equal to or less than 1-hr. are derived from the PMP rates for 1-sq mi areas. The corresponding local intense PMP depth duration curve is shown in Figure 2.4.2-204. The US-APWR plant design is based on a PMP of 19.4 in/hr and 6.~~2~~3 in/5 min. The derived local intense PMP and Intensity duration curve is detailed in Table 2.4.2-206. ~~The derived Intensity Duration Curve corresponding to the local intense PMP is shown in Figure 2.4.2-205. CPNPP Units 3 and 4 site is within the plant design limits for PMP.~~ RCOL2_02.04.02-2 S03 RCOL2_02.04.02-2 S03

The analysis to determine the maximum water surface elevation at safety-related structures included the following steps:

- Estimate peak runoff rate for each drainage area
- Determine water surface elevation at safety-related structures using open channel flow hydraulic modeling software USACE HEC-RAS 4.1.0 (Reference 2.4-300)

Conservative assumptions are made in determining the peak runoff rates at the site. The five minute duration PMP of 6.~~2~~3 inches is used to ~~determine the maximum rainfall intensity of 74.4 inches/hour for the entire watershed~~analyze the effects of local intense precipitation. Additionally, it is assumed there are no losses and all rainfall is converted to runoff. For each drainage area, the peak runoff is determined by multiplying the drainage sub basin area by the maximum rainfall intensity and is listed in Table 2.4.2-207. RCOL2_02.04.02-2 S03

Maximum water surface elevations at safety-related structures are estimated by modeling runoff across the site using HEC-RAS. This model uses standard step backwater equations to estimate the hydraulic flow parameters such as water surface elevations and flow velocities for open channel systems. The steady-state option in the HEC-RAS model is used with input parameters including cross section geometry, Manning's roughness coefficients, and flow boundary conditions.

The site grading and drainage plan is used to determine channel cross section distances and elevations. To ensure a conservative approach, all site structures and buildings are modeled as obstructions and did not allow any storage in the cross section.

Conservative values of the HEC-RAS input parameters intended to maximize the calculated water surface elevations along each of the drainage flow paths or channels are used in the analysis.

- Tailwater conditions for the SCR are assumed to be the peak water surface elevation determined from evaluation of the PMF. The maximum water surface elevation for the SCR is ~~793.66 ft~~794.09 ft NAVD 88, which is used as the downstream boundary condition for runoff entering the SCR. RCOL2_02.04.02-2 S03

- In some cases multiple flow paths are conceivable. Conservative assumptions are made for each channel regarding the accumulation of runoff.

- The flow from a single drainage area may be applied to multiple onsite ponds to maximize the water surface elevation. The flow from each onsite pond is used to determine the downstream boundary condition for the channels flowing into each onsite pond. The standard weir equation is used to estimate overtopping from the pond.

- The grading and drainage plan and Chow (Reference 2.4-233) are used to determine Manning's roughness coefficients. The selected roughness coefficients are increased by a factor of 50 percent to ensure a conservative approach. The coefficients used are n = 0.023~~0~~ for concrete-lined surfaces n = 0.039 for crushed stone/gravel, and n = 0.057 for riprap-lined surfaces. RCOL2_02.04.02-2 S03

- Drainage structures are assumed to be blocked or otherwise non-functional. Inline structures are utilized in HEC-RAS to model overtopping flow at the blocked structures. HEC-RAS utilizes the standard weir flow equation to model overtopping flow. Depending on the shape of the spillway (i.e., broad-crested, ogee-shaped, or sharp-crested) the weir flow coefficient typically ranges from 2.6 to 4.1. A lower weir flow coefficient maximizes the overtopping headwater elevation. Therefore, a weir flow coefficient of 2.6 is selected. HEC-RAS automatically accounts for submergence due to the downstream tailwater.

- Lateral structures are utilized in HEC-RAS to model the overtopping flow leaving the channel. HEC-RAS utilizes the standard weir flow equation to model overtopping flow of lateral structures. A lower weir flow coefficient maximizes the overtopping headwater elevation. Therefore, a weir flow coefficient of 2.6 is selected.

The maximum water surface elevation at the safety-related structures is determined to be ~~820.98 ft at West Channel and~~820.93 ft NAVD 88 at the Center South Channel, which ~~are~~is adjacent to ~~both~~ Unit~~s 3 and~~ 4. The resulting PMP water surface elevations at each channel are listed in Table 2.4.2-208. The safety-related structures are at elevation 822 ft, so they would be safe from ~~potential flooding during~~the potential water surface elevation resulting from the local intense PMP event, even if the entire underground drainage system is completely blocked. RCOL2_02.04.02-2 S03 RCOL2_02.04.02-2 S03

The water surface elevation towards the upstream end of the Unit 3 Southeast Channel exceeds 822 ft. There are no safety-related structures adjacent to the upstream end of the Unit 3 Southeast Channel. The safety-related structures are present towards the downstream end of the Unit 3 Southeast Channel where the water surface elevation is at least one (1) foot or more below the plant grade of 822 ft. There are boundaries that will direct water in a direction away from safety-related structures. The higher water surface elevations in the upstream portions of the Unit 3 Southeast Channel are a function of the channel configuration. The grading and drainage map indicates that the ground elevations at the upstream end of the Unit 3 Southeast Channel are relatively high. The higher elevations correspond to higher water surface elevations in this area. The Unit 3 Southeast Channel has steeper slopes and lower ground elevations as it carries flow that is fully contained in the channel past safety-related structures to the stormwater retention basin ~~N~~northeast of the Unit 3. Therefore, the higher water surface elevations in the upstream cross-sections of the Unit 3 Southeast Channel do not adversely affect the safety-related structures. Consequently, no flooding protection or mitigation measures against flooding are required in FSAR Subsection 2.4.10. RCOL2_02.04.02-2 S03 RCOL2_02.04.02-2 S03

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.

The erosion potential due to the effects of local intense PMP is evaluated for the safety-related structures. The HEC-RAS model used to estimate maximum water surface elevation in the local site analysis identified areas that exhibit supercritical velocities and hydraulic jumps. The selected Manning's roughness coefficients are decreased by a factor of 50 percent to ensure a conservative approach. The coefficients used in the erosion calculation are n = 0.007 for concrete-lined surfaces, n = 0.013 for crushed stone/gravel, and n = 0.017 for riprap-lined surfaces. The grading and drainage plan shows that the designated channels are lined with a concrete bottom and riprap side slopes. All other areas including swales maintain a crushed stone or gravel surface cover. USACE EM 1110-2-1601 (Reference 2.4-301) suggests maximum permissible velocities of 7-13 ft/sec for gravel, which corresponds to roughness coefficients of 0.30-0.45, and velocities of 20-30 ft/sec for concrete, which corresponds to roughness coefficients of 0.016-0.020 ft/sec.

The HEC-RAS model used in the local site analysis is revised to reflect lower Manning's roughness coefficient values. The results are reviewed to identify the areas with a Froude Number greater than 1 (supercritical velocity) and areas containing a hydraulic jump. Supercritical flows are associated with higher velocities that may cause erosion. When a steeper channel slope becomes flat, the flow can quickly transition to subcritical causing the formation of a hydraulic jump, which may further cause erosion to the channel bed.

Each location containing supercritical flow or a hydraulic jump is assessed for impact on safety-related structures based on the location and land cover. The velocity at these locations is compared to the maximum permissible velocity as described in USACE EM 1110-2-1601. There may be a potential for erosion, if the supercritical velocity is greater than the maximum permissible velocity.

The analysis indicates that supercritical flow exists in the Unit 4 UHS, Unit 3 UHS, Center North, Unit 3 East, Unit 3 Southeast, Unit 3 East, and Offsite channels. The supercritical velocities, hydraulic jumps, and their respective locations are listed in Table 2.4.2-209. The results show that supercritical flow at all the locations except for the Unit 3 Southeast Channel is lower than the maximum permissible velocity. Locations of supercritical flow and hydraulic jumps for Unit 4 UHS, Unit 3 UHS, Center North, Unit 3 East, and Unit 3 Southeast, East and Offsite channels are shown in Figure 2.4.2-207.

The Unit 3 Southeast Channel HEC-RAS determined a maximum velocity of ~~13.46~~17.72 ft/sec. ~~The Unit 3 Southeast Channel at this location is mostly gravel except portions of the left overbank, which are concrete.~~ The velocity in the gravel-lined areas is slightly more than the maximum permissible velocity for gravel. However, the Manning's roughness coefficient used to achieve this velocity is 50 percent lower than the normal values. Therefore, the velocity in the Unit 3 Southeast Channel is not expected to be more than the maximum permissible velocity. RCOL2_02.04.02-2 S03

The Unit 3 UHS Channel HEC-RAS determined a maximum velocity of 14.26 ft/sec. The Unit 3 UHS Channel is composed of concrete and gravel. The velocity in the concrete and gravel areas is slightly more than the maximum permissible velocity. However, the Manning's roughness coefficient used to achieve this velocity is 50 percent lower than the normal values. Therefore, the velocity in the Unit 3 UHS Channel is not expected to be more than the maximum permissible velocity. RCOL2_02.04.02-2 S03

2.4.3 Probable Maximum Flood (PMF) on Streams and Rivers

CP COL 2.4(1) Add the following at the end of DCD Subsection 2.4.3.

The guidance in Appendix A of the U.S. Nuclear Regulatory Commission (NRC) Regulatory Guide 1.59 and in NUREG/CR-7046 was followed in determining the probable maximum flood (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 and NUREG/CR-7046. ~~ANSI/ANS-2.8-1992 is the latest available standard.~~ RCOL2_02.04.02-2 S03 RCOL2_02.04.02-2 S03

The PMF was determined for the Squaw Creek watershed and routed through the SCR to determine a water surface elevation of ~~793.66 ft msl~~749.09 ft NAVD 88. 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 ~~761.11 ft msl~~769.11 ft NAVD 88. RCOL2_02.04.02-2 S03 RCOL2_02.04.02-2 S03

The CPNPP Units 3 and 4 safety-related facilities are located at elevation 822 ft ~~msl~~NAVD 88. Therefore, PMF on rivers and streams does not present any potential hazards for CPNPP Units 3 and 4 safety-related facilities. RCOL2_02.04.02-2 S03

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 watershed and the combined Squaw Creek and Paluxy River watersheds to maximize the effects of flooding downstream of the SCR. 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.

~~Considering only the SCR watershed, Basin 1, the critical storm center for the SCR watershed was found to be near the Squaw Creek 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 SCR watershed. However, the storm center SC X results in a higher runoff and hence SC X is considered to be the critical storm center for the SCR watershed. The critical storm area was found to be 100 sq mi, corresponding to isohyet D in Figure 2.4.3-201. The critical storm orientation was found to be 181 degrees.~~ RCOL2_02.04.02-2 S03

~~The critical 72-hr storm PMP rainfall total is 42.53 in for the SCR watershed. The standard HMR 52 temporal arrangement of 6-hr precipitation increments is provided in Table 2.4.3-208. The critical temporal distribution was determined by the runoff analyses to be a two-thirds peaking arrangement for the SCR watershed. The hourly temporal distribution of the 72-hr PMP rainfall for the SCR watershed, Basin 1, is provided in Table 2.4.3-209. The corresponding hyetograph is shown in Figure 2.4.3-211.~~

~~For the remaining portion of the Squaw Creek watershed and the Paluxy River watershed, the critical PMP for each basin was determined considering the combined areas for both watersheds.~~

~~For the remaining portion of the Squaw Creek watershed, Basin 2, 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 145 degrees.~~

~~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 two-thirds peaking arrangement for the Squaw Creek watershed. The hourly two-thirds temporal distribution of the 72-hr PMP rainfall for Basin 2 is provided in Table 2.4.3-203. The corresponding hyetograph is shown in Figure 2.4.3-203.~~

~~For the Paluxy River watershed, Basins 3 and 4 are 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.~~ RCOL2_02.04.02-2 S03

~~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 two-thirds peaking arrangement for the Paluxy River watershed. The hourly temporal distributions of the 72-hr PMP rainfall for Basins 3 and 4 are provided in Table 2.4.3-205. The corresponding hyetographs are shown in Figure 2.4.3-204 and 2.4.3-212.~~

The critical storm center within the Paluxy River watershed (Basin 4) results in the maximum PMP for the overall watershed (Basins 1, 2, 3 and 4 combined) at the confluence of Paluxy River and Squaw Creek (Figure 2.4.3-202). A higher PMP for the SCR watershed can result in a higher water surface elevation at CPNPP Units 3 and 4. Also, a smaller watershed area results in more intense precipitation. Therefore, an additional PMP was determined for the SCR watershed (Basin 1) only as a separate watershed. The PMPs for the above mentioned scenarios were analyzed individually to determine the critical storms, the storms resulting in the highest peak runoff.

The critical 72-hr total PMP for the overall watershed is 35.22 in. The critical storm for the overall watershed has an area of 450 sq mi and it is centered near the overall watershed centroid with an orientation of 295 degrees.The critical 72-hr total PMP for the SCR watershed is 43.19 in. The critical storm for the SCR watershed has an area of 50 sq mi and it is centered near the SCR watershed centroid with an orientation of 301 degrees.

The critical storm centers for the overall watershed and the SCR watershed are indicated as PR 10 and SC 4, respectively, in Figure 2.4.3-202. The PMP estimates with the standard HMR 52 temporal arrangement of 6-hr precipitation increments are provided in Table 2.4.3-202.

RCOL2_02.04.02-2 S03

Overall Watershed

Critical Temporal Duration	Two-third
Basin 1 Hourly Estimate for Critical Temporal Duration	Table 2.4.3-211 Hyetograph- Figure 2.4.3-219
Basin 2 Hourly Estimate for Critical Temporal Duration	Table 2.4.3-212 Hyetograph- Figure 2.4.3-220
Basin 3 Hourly Estimate for Critical Temporal Duration	Table 2.4.3-213 Hyetograph- Figure 2.4.3-221
Basin 4 Hourly Estimate for Critical Temporal Duration	Table 2.4.3-214 Hyetograph- Figure 2.4.3-222

SCR Watershed

Critical Temporal Duration	Two-third
Hourly Estimate for Critical Temporal Duration	Table 2.4.3-215 Hyetograph- Figure 2.4.3-223

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

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. Additionally, no loss rate was assumed for the duration of the modeled events. All rainfall is transformed to runoff. 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.~~4~~5 (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~~developed 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 were used in the runoff model. The basin characteristics are provided in Table 2.4.3-207.

RCOL2_02.04.02-2 S03

RCOL2_02.04.02-2 S03

Basin area, length of stream, and length of stream to the basin centroid are measureable parameters. ~~The basin areas from the existing study were confirmed based on USGS topography. The length of stream and the length of stream to the basin centroid were calculated and compared with the existing study results. The more conservative smaller values were used to determine unit hydrograph characteristics.~~The basin area, length of the stream, and length of the stream to basin centroid were determined for each basin using ArcGIS computer software (Reference 2.4-302)

RCOL2_02.04.02-2 S03

Base flow was determined using the average monthly flow of the 46 cfs from USGS Gage 08091750. The highest of these monthly flows was used as the base flow. Because the basin areas are different from gage area (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.

The Snyder's lag time coefficient and peaking coefficient were selected to maximize runoff. Lag time coefficients range from 1.8 to 2.2. However, lag time coefficients have been found to vary from 0.4 in mountainous areas to 8.0 along the Gulf of Mexico. Lower lag time coefficients are more conservative. Therefore, a 0.4 lag time coefficient has been selected. Peaking coefficients range from 0.4 to 0.8. Higher peaking coefficients are more conservative. Therefore, a 0.8 peaking coefficient has been selected.

Using the watershed subbasin characteristics provided in Table 2.4.3-207, the Snyder's unit hydrograph method was applied to derive unit hydrographs for each subbasin. The resulting Snyder's unit hydrograph characteristics and equations utilized are provided in Table 2.4.3-210. To account for nonlinear basin response at high rainfall rates, the peak of the unit hydrograph for each subbasin has been increased by 20 percent and time to peak reduced by 33 percent. The unit hydrograph was then adjusted to maintain the unit hydrograph characteristic of 1 in of runoff. The derived and modified to account for nonlinear basin response unit hydrographs are provided for each subbasin. The Basin 1a ~~and 1c~~ unit hydrographs are shown in Figure 2.4.3-213. The Basin 1b unit hydrographs are shown in Figure 2.4.3-214. The Basin 2 unit hydrographs are shown in Figure 2.4.3-215. The Basin 3 unit hydrographs are shown in Figure 2.4.3-216. The Basin 4 unit hydrographs are shown in Figure 2.4.3-217.

RCOL2_02.04.02-2 S03

RCOL2_02.04.02-2 S03

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 significant reservoir within the Paluxy River and Squaw Creek watersheds. The storage-elevation rating curve for the SCR is provided in Figure 2.4.3-206 and was obtained from the following two sources:

- The storage-elevation data for elevation 775 ft msl and below have been obtained from the TWDB Volumetric Survey for SCR conducted in 2007. (Reference 2.4-212)

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

In order to project flows beyond those provided in the Operation and Maintenance Procedures for Squaw Creek Dam, the spillway rating curves have been reconstituted using the methods of the U.S. Bureau of Reclamation Design of Small Dams for the service spillway with an ogee crest and the methods of the Federal Highway Administration Hydraulic Design Series Number 5 for the emergency spillway. It is assumed that the ogee crest is submerged 1 ft by tailwater flooding up to elevation 776 ft. The ogee crest discharge coefficient was determined to range from 0 to 3.71 for an overtopping depth of 1 ft to 20 ft. Submergence effects cease as the depth of overtopping flow approaches 4 ft.

Although the emergency spillway crest is not affected by tailwater, submergence is accounted for based on the effects of flow in the channel immediately downstream from the spillway. The rating curve in the Operation and Maintenance Procedures accounts for downstream channel depth of flow from 100 percent to 90 percent of the overtopping headwater depth. Based on the effects of downstream flow, discharge coefficients were derived to range from 1.46 to 2.55 for an overtopping depth of 1 ft to 12 ft.

The combined service spillway and emergency spillway rating curve is provided in Figure 2.4.3-218.

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~~4.1.0 (~~Reference 2.4-234~~Reference 2.4-300), modeling software was used to route the flood hydrographs obtained from the HEC-HMS model. RCOL2_02.04.02-2 S03

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 ~~319,000~~342,954 cfs. The routed peak discharge from the SCR is ~~206,000~~218,206 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. RCOL2_02.04.02-2 S03

Based on the individual basin controlling PMP, the peak flow for Squaw Creek Basin 2 was determined to be ~~31,300~~38,000 cfs, using the two-thirds temporal distribution at the storm center ~~SC X~~PR 10. The peak flow for Paluxy River Basin 3 was determined to be ~~85~~100,000 cfs, using the two-thirds temporal distribution at the storm center PR ~~Y~~10. The peak flow for Paluxy River Basin 4 was determined to be 9~~45~~54,000 cfs, using the two-thirds temporal distribution at the storm center PR ~~Y~~10. RCOL2_02.04.02-2 S03

The individual basin PMP distributions provide maximum peak flows and the temporal distributions are aligned for all basins. Therefore, the maximum backwater flow is determined using the two-thirds temporal distribution at the storm center SC ~~X~~4 for Basin 1~~ and 2~~, and PR ~~Y~~10 for Basins 2, 3, and 4. The maximum backwater flow on the downstream end of the Squaw Creek Dam is ~~181,880~~212,107 cfs. The associated backwater analysis does not provide the controlling PMF water surface elevation at the site. RCOL2_02.04.02-2 S03 RCOL2_02.04.02-2 S03

2.4.3.5 Water Level Determinations

The PMF runoff, routed through the SCR, results in a peak water surface elevation of ~~793.0 ft msl~~793.43 ft NAVD 88 at 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. RCOL2_02.04.02-2 S03

Elevations are provided with reference to the National Geodetic Vertical Datum of 1929 (NGVD 29). The plant site elevation is referenced to the North American Vertical Datum of 1988 (NAVD 88). According to the National Geodetic Survey (Reference 2.4-290), the datum shift of NAVD 88 minus NGVD 29 is equal to between 0 and +0.66 ft for the site. Therefore, it is conservative to account for a maximum conversion of +0.66 ft when comparing water surface elevations determined using NGVD 29 to elevations at the site in NAVD 88. Considering conversion, the SCR maximum water surface elevation of ~~793.66~~794.09 ft NAVD 88 is well below the CPNPP Units 3 and 4 safety-related structures elevation of 822 ft NAVD 88. RCOL2_02.04.02-2 S03

The standard step, unsteady-flow analysis for the Squaw Creek and the Paluxy River watersheds, resulted in a water surface elevation of ~~760.45 ft msl~~768.45 ft NAVD 88 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. Considering datum conversion, the resulting elevation of ~~761.11 ft msl~~769.11 ft NAVD 88 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. RCOL2_02.04.02-2 S03 RCOL2_02.04.02-2 S03

2.4.3.6 Coincident Wind Wave Activity

Fetch length was estimated based on USGS Quadrangles and the PMF maximum water surface elevation of SCR. The critical fetch length was found to be 2.7 mi originating from the east 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).

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 49.91 mph.

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

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 2.85 ft. Runup for the 3:1 slopes was estimated to be 6.99 ft. Runup at the vertical retaining wall on the north side of CPNPP Units 3 and 4 was estimated to be 16.9~~0~~3 ft. RCOL2_02.04.02-2 S03

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 0.0~~8~~7 ft. The maximum total wind wave activity is estimated to be ~~16.98~~17 ft and occurs at the vertical retaining wall. The PMF and maximum coincident wind wave activity results in a flood elevation of ~~810.64 ft msl~~811.09 ft NAVD 88. ~~Elevations are provided with reference to the National Geodetic Vertical Datum of 1929 (NGVD 29). The plant site elevation is referenced to the North American Vertical Datum of 1988 (NAVD 88). According to the National Geodetic Survey, the datum shift of NAVD 88 minus NGVD 29 is equal to between 0 and +0.66 in for the site. Therefore, it is conservative to~~ RCOL2_02.04.02-2 S03 RCOL2_02.04.02-2 S03

~~account for a maximum conversion of +0.66 ft when comparing water surface elevations determined using NGVD 29 to elevations at the site in NAVD 88. Considering conversion, the coincident wind wave activity water surface elevation is 810.64 ft NAVD 88.~~ The top elevation of the retaining wall is ~~795~~817 ft ~~msl~~NAVD 88. ~~Although the coincident wind wave activity water surface elevation exceeds the top elevation of the retaining wall, the water surface elevation is maximized by assuming a vertical surface continues above elevation 805 ft msl.~~ The CPNPP Units 3 and 4 safety-related structures are located at elevation 822 ft ~~msl~~NAVD 88 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. RCOL2_02.04.02-2 S03

elevation (ft)
S = side slope of breach

The first term of the dam breach equation ($1.7 * W_b * h^{1.5}$) is discharge over a broad-crested weir. The coefficient 1.7 is in System International (S.I.) units and must be converted to U.S. Customary units. As presented in Bos, 1989 (Reference 2.4-302), discharge over a broad-crested weir with rectangular control section is defined as: RCOL2_02.04.02-2 S03

$$Q = 2/3 * (2/3*g)^{0.5} * b_c * H_c^{1.5}$$

Where the variable g (gravitational acceleration constant) defines the coefficient of the discharge. For S.I. units $g=9.81\ m/s^2$, then C=1.7; for U.S. Customary units $g=32.2\ ft/s^2$, then C=3.09.

The second term of the dam breach equation ($1.35 * S * h^{2.5}$) is the equation over a v-notched weir. The coefficient 1.35 is in S.I. units and must be converted to U.S. Customary units. As presented in Bos, 1989 (Reference 2.4-302), discharge over a V-notch weir is defined as:

$$Q = C_e * 8/15 * (2*g)^{0.5} * \tan(\Theta/2) * h^{2.5}$$

Where

Θ = angle of v-notch opening (degrees). Based on the trigonometry equations tan (Θ/2) is the same as the side slope of the breach.
C_e = effective discharge coefficient (approximately 0.58)

The variable g (gravitational acceleration constant) defines the coefficient of the discharge. For S.I. units $g=9.81\ m/s^2$, then C=1.37; for U.S. Customary units $g=32.2\ ft/s^2$, then C=2.48.

Dam breach equation in form of the U.S. Customary units:

$$Q = 3.09 * W_b * h^{1.5} + 2.48 * S * h^{2.5}$$

Where:

Q = flow (cfs)
W_b = breach width (ft.)
h = water height (ft.)
S = side slope of the breach (ft/ft)

Alternatively, a breach wave height is computed using the method described in ANSI/ANS-2.8-1992 (Reference 2.4-229).

h = 4 * (headwater - tailwater) / 9

where

h = breach wave height (ft)

Breach characteristics are estimated based on the guidance included in the U.S. Army Corps of Engineers RD-13 (Reference 2.4-240). Estimated breach flows or breach wave heights combined with additional spillway flows and overtopping flows are transposed to the next downstream structure without any attenuation. The transposed flow is combined with coincident PMF flow and a resulting overtopping depth and breach flow or breach wave height is then determined.

Hubbard Creek Dam

A coincident PMF of 600,000 cfs is estimated for the 1107 sq. mi drainage area of Hubbard Creek Dam. The antecedent reservoir elevation is assumed to be at the emergency spillway elevation of 1194.0 ft. This exceeds the maximum recorded water surface elevation. The emergency spillway and fuse plug overtopping elevation is determined to be 1207.4 ft, which does not exceed the dam crest elevation.

Because the service spillway consists of a drop inlet structure interior to the reservoir, it is assumed the full capacity of the service spillway, 30,000 cfs, contributes to downstream flooding in addition to the PMF flow. The tailwater elevation is determined to be 1128.7 ft using the combined flow of 630,000 cfs. The tailwater is well below the spillway elevation.

The wind setup fetch distance is determined to be 11.4 mi using the USGS 1210 ft contour as the basis for the overtopping elevation. The average depth is determined to be 30.0 ft. The wind setup is determined to be 1.0 ft using a wind speed of 60 mph. Therefore, dam failure is evaluated using a headwater elevation of 1208.4 ft.

The following overtopping failures of Hubbard Creek Dam are considered:

- Overtopping failure of the main embankment dam
- Overtopping failure of the embankment fuse plug

A breach width of three times the dam height and 1:1 side slopes are assumed for the main dam. The breach flow is ~~49~~880,000 cfs, accounting for tailwater. Breach flow is added to the combined PMF and service spillway flow for a total of 1,~~12~~510,000 cfs. Alternatively, the breach wave height is 35.5 ft, accounting for tailwater. RCOL2_02.04.02-2 S03 RCOL2_02.04.02-2 S03

The bottom of the fuse plug is determined to be at an elevation of 1170 ft, which is above the tailwater elevation. Therefore, no tailwater effects are considered for the fuse plug failure. The entire 4000 foot long fuse plug is assumed for the breach width along with 1:1 side slopes. The resulting breach flow is ~~1,64~~2,970,000 cfs, which is added to the combined PMF and service spillway flow for a total of ~~2,27~~4,480,000 cfs. Alternatively, the breach wave height is 17.1 ft. RCOL2_02.04.02-2 S03

The potential Hubbard Creek Dam failure effects to be considered (transposed downstream without attenuation to Morris Sheppard Dam) are a breach flow of ~~2,27~~3,600,000 cfs from the fuse plug or a breach wave height of 35.5 ft from the main dam. RCOL2_02.04.02-2 S03

Lake Stamford Dam

A coincident PMF of 350,000 cfs is estimated for the 360 sq. mi drainage area of Lake Stamford Dam. The antecedent reservoir elevation is assumed to be at the dam crest elevation of 1436.8 ft, which exceeds the maximum recorded water surface elevation. It is assumed the service and emergency spillway capacities are not available to accommodate any portion of the PMF. The overtopping elevation is determined to be 1448.0 ft. The tailwater elevation is determined to be 1409.1 ft for the PMF flow. The tailwater is well below the dam crest elevation. RCOL2_02.04.02-2 S03

The wind setup fetch distance is determined to be 10.7 mi using the USGS 1450 ft contour as the basis for the overtopping elevation. The average depth is determined to be 27.7 ft.The wind setup is determined to be 1.0 ft using a wind speed of 60 mph. Therefore, dam failure is evaluated using a headwater elevation of 1449.0 ft.

Overtopping failure of Lake Stamford Dam is considered. A breach width of three times the dam height and 1:1 side slopes are assumed. Accounting for tailwater, the breach flow is ~~12~~210,000 cfs. Breach flow is added to the PMF for a total of ~~47~~560,000 cfs. Alternatively, the breach wave height is 17.8 ft, accounting for tailwater. The potential Lake Stamford Dam failure effects are to be considered for combination with the proposed Cedar Ridge Reservoir Dam failure effects and transposed downstream without attenuation to Morris Sheppard Dam. RCOL2_02.04.02-2 S03

Fort Phantom Hill Dam

A coincident PMF of 410,000 cfs is estimated for the 478 sq mi drainage area of Fort Phantom Hill Dam. The antecedent reservoir elevation is assumed to be at the levee crest elevation of 1643.0 ft. This exceeds the maximum recorded water surface elevation. It is assumed spillway capacity is not available to accommodate any portion of the PMF. The overtopping elevation is determined to be 1651.1 ft. The tailwater elevation is determined to be 1576.9 ft for the PMF flow. The tailwater is well below the levee and dam crest elevations.

The wind setup fetch distance is determined to be 7.9 mi using midway between the USGS 1650 ft and 1660 ft contours as the basis for the overtopping elevation. The average depth is determined to be 24.0 ft. The wind setup is determined to be

0.9 ft using a wind speed of 60 mph. Therefore, dam failure is evaluated using a headwater elevation of 1652.0 ft.

Because the levee is not as high, only overtopping failure of Fort Phantom Hill Dam is considered. A breach width of three times the dam height and 1:1 side slopes are assumed. The breach flow is ~~35~~630,000 cfs, accounting for tailwater. Breach flow is added to the PMF for a total of ~~76~~1,040,000 cfs. Alternatively, the breach wave height is 33.4 ft, accounting for tailwater. The potential Fort Phantom Hill Dam failure effects are transposed downstream without attenuation to the proposed Cedar Ridge Reservoir Dam. RCOL2_02.04.02-2 S03

Cedar Ridge Reservoir Dam

A coincident PMF of 810,000 cfs is estimated for the 2748 sq. mi drainage area of the proposed Cedar Ridge Reservoir Dam. Because the upstream dam failure effects include the Fort Phantom Hill Dam PMF of 410,000 cfs, only 400,000 cfs is added to the upstream dam failure effects to represent the contribution from the proposed Cedar Ridge Reservoir PMF. The antecedent reservoir elevation is assumed to be at the dam crest elevation of 1510.0 ft.

The overtopping elevation is determined to be 153~~0~~3.~~1~~2 ft for the combined PMF and upstream dam failure effects flow of 1,~~16~~440,000 cfs. The corresponding tailwater elevation is determined to be 1441.7 ft, which is well below the dam crest elevation. RCOL2_02.04.02-2 S03

Alternatively, the upstream dam failure breach wave height is added to the antecedent reservoir elevation to determine the corresponding flow. The flow is 2,500,000 cfs at an overtopping elevation of 1543.4 ft. The contributing portion of the proposed Cedar Ridge Reservoir coincident PMF is added for the combined PMF and upstream dam failure breach wave height of 2,900,000 cfs. The resulting overtopping elevation is determined to be 1547.0 ft. The corresponding tailwater elevation is determined to be 1471.3 ft, which is well below the dam crest elevation.

The wind setup fetch distance is determined to be 6.9 mi using the USGS 1550 ft contour as the basis for the overtopping elevation. The average depth is determined to be 68.2 ft. The wind setup is determined to be 0.3 ft using a wind speed of 60 mph. Therefore, dam failure is evaluated using a headwater elevation of 153~~0~~3.~~4~~5 ft for an overtopping flow of 1,~~16~~440,000 cfs or 1547.3 ft for an overtopping flow of 2,900,000 cfs. RCOL2_02.04.02-2 S03

The following overtopping failure conditions of the proposed Cedar Ridge Reservoir Dam are considered:

- Overtopping flow of 1,~~16~~440,000 cfs with a headwater elevation 153~~0~~3.~~4~~5 ft and a tailwater elevation 1441.7 ft RCOL2_02.04.02-2 S03

- Overtopping flow of 2,900,000 cfs with a headwater elevation 1547.3 ft and a tailwater elevation 1471.3 ft

A breach width of three times the dam height and 1:1 side slopes are assumed. Based on an overtopping flow of 1,~~16~~440,000 cfs and accounting for tailwater, the breach flow is ~~710~~1,240,000 cfs. Breach flow is added to the PMF and overtopping flow for a total of ~~1,87~~2,680,000 cfs. Alternatively, the breach wave height is 3~~9~~8.~~5~~4 ft, accounting for tailwater. Based on an overtopping flow of 2,900,000 cfs and accounting for tailwater, the breach flow is ~~560~~1,010,000 cfs. Breach flow is added to the PMF and overtopping flow for a total of 3,~~46~~910,000 cfs. Alternatively, the breach wave height is 3~~3~~8.~~8~~4 ft, accounting for tailwater. RCOL2_02.04.02-2 S03

The potential Cedar Ridge Reservoir Dam failure effects to be considered (transposed downstream without attenuation to Morris Sheppard Dam) are a breach flow of 3,~~46~~910,000 cfs or a breach wave height of 3~~9~~8.~~5~~4 ft. When combined with the Lake Stamford Dam failure effects, the total upstream dam failure effects are ~~3,93~~4,470,000 cfs or a wave height of 5~~7~~6.~~3~~2 ft. The combined upstream dam failure effects exceed the potential failure effects from Hubbard Creek Dam. Therefore, the controlling dam failure scenario includes the domino-type failures Fort Phantom Hill Dam, proposed Cedar Ridge Reservoir Dam, Morris Sheppard Dam, and De Cordova Bend Dam. In addition Lake Stamford Dam is assumed to fail simultaneous with the Cedar Ridge Reservoir Dam. RCOL2_02.04.02-2 S03 RCOL2_02.04.02-2 S03

Morris Sheppard Dam

For the 13,310 sq. mi contributing drainage area of Morris Sheppard Dam, the greater 16,113 sq. mi contributing drainage area of De Cordova Bend Dam is used to determine the coincident PMF of 1,450,000 cfs is estimated. Although, the maximum historical elevation was recorded prior to construction of the emergency spillway, it is assumed the antecedent reservoir elevation is the maximum historical elevation of 1003.6 ft. Assuming the spillway gates are closed and overtopped by the antecedent reservoir elevation, the combined emergency spillway and gate overtopping flow is 40,000 cfs.

The upstream dam failure effects are added to the coincident PMF and antecedent reservoir elevation flow for a total overtopping flow of 5,~~42~~960,000 cfs. The overtopping elevation is determined to be 107~~5~~9.~~7~~9 ft. The corresponding tailwater elevation is determined to be 97~~3~~7.~~0~~6 ft, which is well below the spillway crest and top of gates elevations. RCOL2_02.04.02-2 S03

Alternatively, the upstream dam failure breach wave height is added to the antecedent reservoir elevation and combined with the coincident PMF to determine the corresponding flow. At an overtopping elevation of 10~~60~~59.~~9~~8 ft the flow is 3,~~670~~541,000 cfs. The combined PMF and upstream dam failure breach wave height flow is ~~5~~4,~~120~~991,000 cfs. The resulting overtopping elevation is determined to be 107~~3~~2.3 ft. The corresponding tailwater elevation is determined to be 9~~70~~69.~~3~~2 ft, which is well below the spillway crest and top of gates elevations. RCOL2_02.04.02-2 S03

The wind setup fetch distance is determined to be 2.3 mi using the USGS 1080 ft contour as the basis for the overtopping elevation. The average depth is determined to be 12~~0~~4.~~5~~6 ft. The wind setup is determined to be 0.1 ft using a wind speed of 60 mph. Therefore, dam failure is evaluated using a headwater elevation of 10~~75~~80.~~8~~0 ft for an overtopping flow of 5,~~42~~960,000 cfs or 107~~3~~2.4 ft for an overtopping flow of ~~5~~4,~~120~~991,000 cfs. RCOL2_02.04.02-2 S03

The following overtopping failures of Morris Sheppard Dam are considered:

- Overtopping failure of the spillway section.
- Overtopping failure of the embankment section.
- Overtopping failure of the buttress section at the left abutment.
- Overtopping failure of the buttress section between the spillway and embankment sections.

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

The following overtopping failure conditions of Morris Sheppard Dam are considered:

- Overtopping flow of 5,~~42~~960,000 cfs with a headwater elevation 10~~75~~80.~~8~~0 ft and a tailwater elevation 97~~3~~7.~~0~~6 ft. RCOL2_02.04.02-2 S03
- Overtopping flow of ~~5~~4,~~120~~991,000 cfs with a headwater elevation 107~~3~~2.4 ft and a tailwater elevation 9~~70~~69.~~3~~2 ft.

A breach width of the entire spillway section and vertical side slopes are assumed. Based on an overtopping flow of 5,~~42~~960,000 cfs and accounting for tailwater, the breach flow is ~~1~~2,2~~4~~70,000 cfs. Breach flow is added to the overtopping flow for a total of ~~6~~8,~~66~~230,000 cfs. Alternatively, the breach wave height is 45.~~7~~6 ft, accounting for tailwater. Based on an overtopping flow of ~~5~~4,~~120~~991,000 cfs and accounting for tailwater, the breach flow is ~~1~~2,~~25~~300,000 cfs. Breach flow is added to the PMF and overtopping flow for a total of ~~6~~7,~~370~~291,000 cfs. Alternatively, the breach wave height is 45.9 ft, accounting for tailwater. RCOL2_02.04.02-2 S03

The bottom of the embankment section is determined to be at an elevation of 990 ft. This is above the tailwater elevation. Therefore, no tailwater effects are considered for the embankment section failure. A breach width of three times the dam height and 1:1 side slopes are assumed. Based on an overtopping flow of

5,~~42~~960,000 cfs the resulting breach flow is ~~23~~460,000 cfs. Breach flow is added to the overtopping flow for a total of ~~5~~6,~~65~~420,000 cfs. Alternatively, the breach wave height is ~~38~~40.~~2~~0 ft. Based on an overtopping flow of ~~5~~4,~~120~~991,000 cfs the resulting breach flow is ~~220~~389,000 cfs. Breach flow is added to the overtopping flow for a total of 5,3~~4~~80,000 cfs. Alternatively, the breach wave height is 3~~7~~6.~~1~~7 ft. RCOL2_02.04.02-2 S03

The potential Morris Sheppard Dam failure effects, transposed downstream without attenuation to De Cordova Bend Dam, to be considered are a spillway section breach flow of ~~6~~8,~~66~~230,000 cfs or a breach wave height of 45.9 ft. RCOL2_02.04.02-2 S03

De Cordova Bend Dam

The Morris Sheppard Dam failure effects include the PMF for the Brazos River at De Cordova Bend Dam. Therefore, no additional flow is combined with the upstream failure effects. For the overtopping flow, the antecedent reservoir elevation is assumed to be at the dam crest elevation of 706.5 ft. Because of topography conditions around the reservoir, above elevation 700 ft. the reservoir is capable of spilling over low lying elevations along the south rim of the reservoir into the Brazos River well downstream from the dam. Based on the overtopping flow of ~~6~~8,~~66~~230,000 cfs and a reduced weir flow coefficient of 1.54, the headwater is determined to be 7~~66~~72.~~4~~7 ft. The corresponding tailwater is determined to be 75~~1~~5.~~1~~7 ft. Tailwater is determined for only the ~~4~~5,6~~7~~40,000 cfs portion of total flow that overtops the dam and adjacent abutment areas. The remaining flow overtops the south rim of the reservoir. RCOL2_02.04.02-2 S03

Alternatively, for the breach wave height, it is assumed the antecedent reservoir elevation is the maximum historical elevation of 693.6 ft. The upstream dam failure breach wave height is added to the antecedent reservoir elevation to determine the corresponding flow. At an overtopping elevation of 739.5 ft the flow is 3,~~27~~300,000 cfs. The corresponding tailwater elevation is determined to be 734.2 ft. Tailwater is determined for only the 2,75~~0~~1,000 cfs portion of total flow that overtops the dam and adjacent abutment areas. The remaining flow overtops the south rim of the reservoir. Although, the tailwater exceeds the dam crest elevation, it is determined that at the overtopping elevation the weir flow coefficient does not require reduction. RCOL2_02.04.02-2 S03

The wind setup fetch distance is determined to be 5.~~3~~9 mi using the USGS 770 ft contour as the basis for the overtopping elevation. The average depth is determined to be ~~67.9~~58.4 ft. Using a wind speed of 60 mph, the wind setup is determined to be 0.3 ft. Therefore, dam failure is evaluated using a headwater elevation of 7~~66~~73.~~7~~0 ft for a total overtopping flow of ~~6~~8,~~66~~230,000 cfs or 739.8 ft for a total overtopping flow of 3,~~27~~300,000 cfs. RCOL2_02.04.02-2 S03 RCOL2_02.04.02-2 S03 RCOL2_02.04.02-2 S03

The following overtopping failures of De Cordova Bend Dam are considered:

- Overtopping failure of the spillway section.
- Overtopping failure of the embankment section.

The following overtopping failure conditions of De Cordova Bend Dam are considered:

- Overtopping flow of ~~6~~8,~~66~~230,000 cfs with a headwater elevation 7~~66~~73.~~7~~0 ft and a tailwater elevation 77~~5~~5~~1~~.~~1~~7 ft. RCOL2_02.04.02-2 S03
- Overtopping flow of 3,~~27~~300,000 cfs with a headwater elevation 739.8 ft and a tailwater elevation 734.2 ft.

A breach width of the entire spillway section and vertical side slopes are assumed. Based on an overtopping flow of ~~6~~8,~~66~~230,000 cfs and accounting for tailwater, the breach flow is ~~7~~150,000 cfs. Breach flow is added to the overtopping flow for a total of ~~6~~8,~~73~~380,000 cfs. Alternatively, the breach wave height is 7.~~0~~7 ft, accounting for tailwater. Based on an overtopping flow of 3,~~27~~300,000 cfs and accounting for tailwater, the breach flow is ~~2~~30,000 cfs. Breach flow is added to the overtopping flow for a total of 3,~~29~~330,000 cfs. Alternatively, the breach wave height is 2.5 ft, accounting for tailwater. RCOL2_02.04.02-2 S03

A breach width of three times the dam height and 1:1 side slopes are assumed for the embankment section. Based on an overtopping flow of ~~6~~8,~~66~~230,000 cfs and accounting for tailwater, the breach flow is ~~3~~60,000 cfs. Breach flow is added to the overtopping flow for a total of ~~6~~8,~~6~~290,000 cfs. Based on an overtopping flow of 3,~~27~~300,000 cfs and accounting for tailwater, the breach flow is 1~~0~~1,000 cfs. Breach flow is added to the overtopping flow for a total of 3,~~280~~311,000 cfs. Alternatively, because of the tailwater effects, the embankment section breach wave heights are identical to those determined for the spillway section. RCOL2_02.04.02-2 S03

The overtopping failure of the entire spillway section results in the greatest breach flow.Because of the tailwater effects, the breach wave height was added to the downstream tailwater elevation to determine a corresponding flow. However, the result did not exceed the breach flow. Considering the breach flow and overtopping flow, including overtopping flow spreading out beyond the abutments and spilling over the south rim of the reservoir, the total outflow is determined to be ~~6~~8,~~73~~380,000. This flow is transposed downstream without any attenuation to the confluence of the Paluxy River near its confluence with Squaw Creek to determine the relevant water surface elevation. RCOL2_02.04.02-2 S03

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 Fort Phantom Hill Dam, the proposed Cedar Ridge Dam, the Lake Stamford Dam, the Morris Sheppard Dam, and the De Cordova 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 HEC-RAS computer software (Reference 2.4-234), the stream course model described in Subsection 2.4.3.3 is used as a basis to determine the water surface elevation at the confluence.

The HEC-RAS stream course model is appended to include cross sections for the Brazos River. The selected cross sections are identified in Figure 2.4.4-202. As discussed in Subsection 2.4.4.3, a Manning's Roughness coefficient of 0.15 is also used for the Brazos River. The peak flows from the HEC-HMS model described in Subsection 2.4.3 for the Paluxy River and Squaw Creek were included as inputs for the Brazos River tributaries. The transposed ~~6~~8,~~73~~380,000 cfs from the dam failure scenario is included as the Brazos River input. The HEC-RAS model was run using steady state conditions to determine the water surface elevation at the confluence. RCOL2_02.04.02-2 S03

The resulting maximum water surface elevation at the confluence of Brazos River and Paluxy River cross section is 76~~0~~8.0~~2~~3 ft ~~msl~~NAVD 88 for the total transposed flow combined with the peak tributary flows as shown in Figure 2.4.4-203. 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. RCOL2_02.04.02-2 S03

Elevations are provided with reference to the National Geodetic Vertical Datum of 1929 (NGVD 29). The plant site elevation is referenced to the North American Vertical Datum of 1988 (NAVD 88). According to the National Geodetic Survey, the datum shift of NAVD 88 minus NGVD 29 is equal to between 0 and +0.66 in for the site. Therefore, it is conservative to account for a maximum conversion of

+0.66 ft when comparing water surface elevations determined using NGVD 29 to elevations at the site in NAVD 88. Considering conversion, the confluence water surface elevation of 76~~0~~8.6~~8~~9 ft NAVD 88 is well below the CPNPP Units 3 and 4 safety-related structures elevation of 822 ft NAVD 88. RCOL2_02.04.02-2 S03

2.4.5 Probable Maximum Surge and Seiche Flooding

CP COL 2.4(1) Add the following at the end of DCD Subsection 2.4.5.

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 16.93 ft. The maximum wind setup is estimated to be 0.0~~8~~7 ft. Therefore, the total water surface elevation increase due to wind wave activity is estimated to be ~~16.98~~17 ft. The resulting PMF coincident with wind wave activity elevation is ~~810.64 ft msl~~811.09 ft NAVD 88. RCOL2_02.04.02-2 S03 RCOL2_02.04.02-2 S03

Seismic seiches mainly depend on factors such as frequency and magnitude of the excitation, depth and geometry of the water body, and the sediment properties surrounding the water body (References 2.4-292 and 2.4-293). The risk of the occurrence of seismic seiches greater than about 5 ft in height at the SCR site is considered very low because a comprehensive study by Barberopoulou (References 2.4-293 and 2.4-294) found that Lake Union, Washington, a site with geometry, geology, and seismicity conditions that are much more favorable for seismic seiche development, indicated a maximum seismic seiche height of about 5 ft. Lake Union is therefore considered to be a conservative bounding case for SCR, and maximum seismic seiche heights at the SCR location are not expected to exceed those for Lake Union. The CPNPP Units 3 and 4 site finish grade elevation of 822 ft provides an approximately 28-ft margin over the maximum SCR water level during the PMF event, which is significantly larger than the expected maximum seismic seiche height of 5 ft.

According to the guidance of ANSI/ANS-2.8-1992 (Reference 2.4-229), the region of occurrence for a PMH shall be considered for U.S. coastline areas and areas within 100 to 200 miles bordering the Gulf of Mexico. CPNPP Units 3 and 4 are located approximately 275 mi inland from the Gulf of Mexico and outside the region of occurrence for a PMH. Therefore, a PMH was not considered. CPNPP Units 3 and 4 safety-related facilities are located at the plant grade level elevation of 822 ft ~~msl~~NAVD 88. A surge due to a PMH event would not cause flooding at the site. RCOL2_02.04.02-2 S03

2.4.7 Ice Effects

CP COL 2.4(1) Add the following at the end of DCD Subsection 2.4.7.

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~~NAVD 88. The SCR spillway elevation is 775 ft ~~msl~~NAVD 88 (Reference 2.4-214). The maximum water surface elevation during a probable maximum flood event and coincident wind waves is at ~~810.64 ft msl~~811.09 ft NAVD 88, which is more than 1~~1~~0 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. RCOL2_02.04.02-2 S03 RCOL2_02.04.02-2 S03

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

2.4.10 Flooding Protection Requirements

CP COL 2.4(1) Add the following at the end of DCD Subsection 2.4.10.

CPNPP Units 3 and 4 safety-related facilities are not exposed to flooding from all events identified in Subsection 2.4.2. The critical flooding event is identified in Subsection 2.4.2 and discussed in detail in Subsection 2.4.3. The maximum flood level is a result of the ~~probable maximum precipitation on the Squaw Creek watershed and includes the effects of coincident wind wave activity~~maximum local intense precipitation at Center South Channel adjacent to both Units 3 and 4. Based on the design information provided in the referenced subsections, flood protection measures and emergency procedures to address flood protection are not required. RCOL2_02.04.02-2 S03

2.4.14 Technical Specifications and Emergency Operation Requirements

CP COL 2.4(1) Add the following after the paragraph in DCD Subsection 2.4.14.

The grade elevation of CPNPP Units 3 and 4 is above the ~~probable~~ maximum flood ~~(PMF)~~ elevation; therefore, due to plant grade elevation and the unique "always in place" four tank design of the UHS there are no requirements for emergency protective measures designed to minimize the impact of adverse hydrology-related events on safety-related facilities, and none are incorporated into the Technical Specifications or emergency procedures. RCOL2_02.04.02-2 S03

2.4-232 U.S. Army Corps of Engineers, Hydrologic Engineering Center, Hydrologic Modeling System, HEC-HMS computer software, version 3.5~~1.0.~~,Build ~~1206~~1417, August 10, 2010. RCOL2_02.04.02.2 S03

2.4-233 Chow, V.T., "Open Channel Hydraulics", McGraw-Hill Inc., New York, 1959.

2.4-234 ~~U.S. Army Corps of Engineers, Hydrologic Engineering Center, River Analysis System, HEC-RAS computer software, version 3.1.3., May 2005~~Not Used RCOL2_02.04.02.2 S03

2.4-235 U.S. Army Corps of Engineers, "Coastal Engineering Manual," EM 1110-2-1100, Part 2, April 30, 2002 (Change 2: June 1, 2006).

2.4-236 U.S. Army Corps of Engineers, "Hydrologic Engineering Requirements for Reservoirs," EM 1110-2-1420, October 31, 1997.

2.4-237 U.S. Geological Survey, Geospatial Data Gateway Website, http://datagateway.nrcs.usda.gov/gatewayhome.html, accessed December 27, 2007.

2.4-238 U.S. Army Corps of Engineers, Hydrologic Engineering Center, Hydrologic Modeling System, HEC-HMS Release Notes, Version 2.2.0, August 2002.

2.4-239 U.S. Army Corps of Engineers mathematical expressions provided by EM 1110-2-1420, "Hydrologic Engineering Requirements for Reservoirs."

2.4-240 Owen, J.H., "Flood Emergency Plans, Guidelines for Corps Dams," U.S. Army Corps of Engineers, RD-13, June 1980.

2.4-241 Bentley Systems, Inc., FlowMaster computer software, Service Pack 3, Copyright 2005.

2.4-242 U.S. Army Corps of Engineers, Seismic Design for Buildings, TI 809-04, December 31, 1998.

2.4-243 National Oceanic and Atmospheric Administration, National Geophysical Data Center Tsunami Database, Website http://www.ngdc.noaa.gov/seg/hazard/tsu_db.shtml, data extracted January 2008.

2.4-244 Atlantic and Gulf of Mexico Tsunami Hazard Assessment Group, "The Current State of Knowledge Regarding Potential Tsunami Sources Affecting U.S. Atlantic and Gulf Coasts – A Report to the Nuclear Regulatory Commission," U.S. Geological Survey Administrative report, September 2007.

2.4-298 C.-Y.Wang, M. Manga, Earthquakes and Water, Lecture Notes in Earth Sciences 114, 218p, Springer Verlag Berlin Heidelberg, 2010.

2.4-299 U.S. Geological Survey, Ground Water Atlas of the United States, Oklahoma, Texas, HA-730-E, Website, http://pubs.usgs.gov/ha/ha730/ch_e/E-text8.html, Accessed, July 19, 2011, Table 7.

2.4-300 U.S. Army Corps of Engineers, Hydrologic Engineering Center - River Analysis System, HEC-RAS Computer Software, Version 4.1.0, January 2010.

2.4-301 U.S. Army Corps of Engineers, Engineering & Design, Hydraulic Design of Flood Control Channels, EM 1110-2-1601, Washington D.C., July 1 1991.

2.4-302 Bos M.G., "Discharge Measure Structures", Third Revised Edition, International Institute for Land Reclamation and Improvement (ILRI), Netherlands, 1989. RCOL2_02.04.02-2 S03

CP COL 2.4(1)

Table 2.4.2-205
Hourly Rainfall Depth for PMP

RCOL2_02.04.02-2 S03

Hour	Cumulative PMP (in)	Incremental PMP (in)	Hour	Cumulative PMP (in)	Incremental PMP (in)
1	19.~~0~~10	19.~~0~~10	37	~~43.88~~44.04	0.21~~9~~
2	22.39	3.~~3~~29	38	44.~~07~~25	0.21~~8~~
3	24.61	2.23	39	44.~~2~~46	0.21~~8~~
4	26.44	1.82	40	44.~~41~~66	0.~~17~~20
5	28.04	1.60	41	44.~~5~~87	0.21~~6~~
6	29.~~50~~83	1.~~46~~79	42	~~44.74~~45.08	0.21~~6~~
7	30.~~86~~94	1.~~36~~11	43	~~44.89~~45.29	0.21~~6~~
8	32.12	1.~~26~~18	44	45.~~04~~50	0.21~~5~~
9	33.26	1.14	45	45.71~~9~~	0.21~~5~~
10	34.29	1.03	46	45.~~33~~91	0.~~14~~20
11	35.20	0.91	47	~~45.47~~46.12	0.21~~4~~
12	36.0~~0~~9	0.89~~0~~	48	~~45.60~~46.33	0.21~~3~~
13	36.70	0.~~70~~61	49	~~45.73~~46.44	0.11~~3~~
14	37.30	0.61	50	~~45.85~~46.55	0.11~~3~~
15	37.84	0.54	51	~~45.98~~46.67	0.12
16	38.33	0.48	52	46.~~10~~78	0.11~~2~~
17	38.76	0.43	53	46.~~21~~89	0.11~~2~~
18	39.16	0.39	54	~~46.32~~47.00	0.11
19	39.52	0.36	55	~~46.43~~47.11	0.11
20	39.8~~5~~7	0.3~~3~~5	56	~~46.54~~47.22	0.11
21	40.~~16~~25	0.3~~1~~8	57	~~46.64~~47.34	0.12~~0~~
22	40.~~45~~60	0.~~29~~35	58	~~46.74~~47.45	0.11~~0~~
23	~~40.73~~41.00	0.~~28~~40	59	~~46.84~~47.56	0.11~~0~~
24	41.~~00~~33	0.~~27~~33	60	~~46.94~~47.67	0.11~~0~~
25	41.~~26~~54	0.21~~6~~	61	47.~~03~~78	0.11~~0~~
26	41.~~51~~75	0.21~~5~~	62	47.~~13~~89	0.~~09~~11
27	41.~~7~~96	0.21~~5~~	63	~~47.22~~48.01	0.~~09~~12
28	42.~~00~~16	0.20~~4~~	64	~~47.31~~48.12	0.~~09~~11
29	42.~~2~~37	0.21~~3~~	65	~~47.40~~48.23	0.~~09~~11
30	42.~~46~~58	0.21~~3~~	66	~~47.49~~48.34	0.~~09~~11
31	42.~~68~~79	0.21~~2~~	67	~~47.57~~48.45	0.~~09~~11
32	~~42.90~~43.00	0.21	68	~~47.66~~48.56	0.~~09~~11
33	43.~~1~~21	0.21	69	~~47.75~~48.68	0.~~09~~12
34	43.~~3~~41	0.20	70	~~47.83~~48.79	0.~~09~~11
35	43.~~51~~62	0.21~~0~~	71	~~47.92~~48.90	0.~~08~~11
36	43.~~70~~83	0.21~~9~~	72	~~48.00~~49.01	0.~~08~~11

CP COL 2.4(1)

RCOL2_02.04.02-2 S03

Table 2.4.2-206
5 Minute Rainfall Depth for Local Intense PMP

Minutes	Cumulative PMP (in)	~~Rainfall Intensity~~ Incremental PMP (in.~~/hr~~)	Minutes	Cumulative PMP (in)	Rainfall Intensity (in)
5	6.~~2~~30	~~74.4~~6.30	185	24.78	~~8.0~~0.16
10	8.~~12~~05	~~48.7~~1.75	190	24.94	~~7.9~~0.16
15	9.~~7~~80	~~38.8~~1.75	195	25.10	~~7.7~~0.16
20	11.23	~~33.7~~1.43	200	25.25	~~7.6~~0.16
25	12.73	~~30.6~~1.50	205	25.41	~~7.4~~0.15
30	14.20	~~28.4~~1.47	210	25.56	~~7.3~~0.15
35	15.55	~~26.7~~1.35	215	25.71	~~7.2~~0.15
40	16.59	~~24.9~~1.04	220	25.86	~~7.1~~0.15
45	17.38	~~23.2~~0.79	225	26.01	~~6.9~~0.15
50	18.02	~~21.6~~0.63	230	26.15	~~6.8~~0.15
55	18.55	~~20.2~~0.53	235	26.29	~~6.7~~0.14
60	19.~~0~~10	~~19.0~~0.55	240	26.44	~~6.6~~0.14
65	19.44~~0~~	~~17.9~~0.34	245	26.58	~~6.5~~0.14
70	19.76	~~16.9~~0.32	250	26.72	~~6.4~~0.14
75	20.09	~~16.1~~0.33	255	26.87~~5~~	~~6.3~~0.15
80	20.40	~~15.3~~0.31	260	26.99	~~6.2~~0.12
85	20.69	~~14.6~~0.29	265	27.12	~~6.1~~0.13
90	20.96	~~14.0~~0.27	270	27.~~26~~31	~~6.1~~0.19
95	21.23	~~13.4~~0.26	275	27.~~39~~45	~~6.0~~0.14
100	21.48	~~12.9~~0.25	280	27.~~52~~60	~~5.9~~0.15
105	21.72	~~12.4~~0.24	285	27.~~6~~75	~~5.8~~0.15
110	21.95	~~12.0~~0.23	290	27.~~7~~89	~~5.7~~0.14
115	22.17	~~11.6~~0.22	295	~~27.91~~28.04	~~5.7~~0.15
120	22.39	~~11.2~~0.22	300	28.~~04~~17	~~5.6~~0.13
125	22.60	~~10.8~~0.21	305	28.31~~6~~	~~5.5~~0.14
130	22.80	~~10.5~~0.20	310	28.~~29~~45	~~5.5~~0.14
135	23.00	~~10.2~~0.20	315	28.~~41~~59	~~5.4~~0.14
140	23.20	~~9.9~~0.19	320	28.~~54~~73	~~5.4~~0.14
145	23.39	~~9.7~~0.19	325	28.~~66~~87	~~5.3~~0.14
150	23.57	~~9.4~~0.19	330	~~28.78~~29.00	~~5.2~~0.13
155	23.75	~~9.2~~0.18	335	~~28.90~~29.14	~~5.2~~0.14
160	23.93	~~9.0~~0.18	340	29.~~02~~8	~~5.1~~0.14
165	24.12~~1~~	~~8.8~~0.19	345	29.~~14~~2	~~5.1~~0.14
170	24.28	~~8.6~~0.16	350	29.~~26~~55	~~5.0~~0.13
175	24.45	~~8.4~~0.17	355	29.~~38~~69	~~5.0~~0.14
180	24.61	~~8.2~~0.17	360	29.~~50~~83	~~4.9~~0.14

CP COL 2.4(1)

Table 2.4.2-207 (Sheet 1 of 2)
Site Drainage Area Peak Runoff

Drainage Sub Basin	Area A (ac)	Peak Runoff Q (cfs)
1	0.~~70~~69	53~~2~~
2	2.~~09~~31	176~~57~~
3	~~3.11~~2.95	225~~33~~
4	1.~~28~~35	~~96~~103
5	3.~~43~~59	273~~57~~
6	2.44	186~~3~~
7	0.73~~4~~	56~~5~~
8	1.78~~5~~	136~~1~~
9	4.~~88~~79	36~~6~~5
10	4.35~~2~~	331~~24~~
11	1.04~~6~~	79
12	3.23~~6~~	246~~4~~
13	1.65~~3~~	126~~3~~
14	1.73~~9~~	132~~5~~
15	1.~~63~~72	131~~23~~
16	4.47~~6~~	341~~35~~
17	0.88	67~~6~~
18	1.29	98~~7~~
19	0.49	38~~7~~
20	1.03	78~~7~~
21	1.80	138~~5~~
22	5.85	446~~39~~

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CP COL 2.4(1)

Table 2.4.2-207 (Sheet 2 of 2)
Site Drainage Area Peak Runoff

Drainage Sub Basin	Area A (ac)	Peak Runoff Q (cfs)
23	38.08	2,903~~857~~
24	62.72	4,781~~05~~
25	1.65~~7~~	126~~5~~
26	0.~~51~~49	37~~9~~
27	6.~~59~~64	~~494~~506
28	2.46~~1~~	187~~96~~
29	~~4.94~~5.88	~~371~~448
30	2.26~~84~~	~~213~~172
31	3.60	275~~0~~
32	~~21.30~~34.99	2,668~~423~~

RCOL2_02.04.02-2 S03

CP COL 2.4(1)

Table 2.4.2-208
PMP Maximum Water Surface Elevations

Feature	Max Water Surface Elevation (ft)	Adjacent Unit
Drainage Pond A	816.1	N/A
Drainage Pond B	815.~~1~~4	N/A
Drainage Pond C	813.9	N/A
Unit 4 UHS Channel	819.~~80~~75	Unit 4
West Channel	820.~~98~~75	Unit 3 and Unit 4
Center South Channel	820.93~~8~~	Unit 3 and Unit 4
Unit 3 UHS Channel	819.~~66~~80	Unit 3
Unit 3 North Channel	820.13~~4~~	Unit 3
Center North Channel	820.~~44~~80	Unit 3 and Unit 4
Unit 4 North Channel	820.15~~1~~	Unit 4
Unit 3 East Channel	820.84~~8~~	Unit 3
Unit 3 Southeast Channel	822.~~70~~23*	None
	81~~9~~8.~~77~~53	Unit 3
East Channel	821.84~~8~~*	None
Off-site Channel	82~~0~~1.~~78~~92	None

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Note:
* Based on discussions provided in Subsection 2.4.2.3, the higher water surface elevation (>821 ft) in the Off-site Channel, East Channel and the Unit 3 Southeast Channel do not adversely affect the safety-related structures.

Table 2.4.2-209
Summary of Results Identifying Super-critical Velocities and Hydraulic Jumps

Channel	Maximum Supercritical Velocity (ft/sec)	Hydraulic Jump	Land Cover	Maximum Permissible Velocity (ft/sec)	
Unit 4 UHS Channel	5.~~5~~45	Yes	Gravel	7-13	RCOL2_02.04.02-2 S03
West Channel	-	-	-	-	
Center South Channel	-	-	-	-	
Unit 3 UHS Channel	5.10~~3~~	Yes	Gravel	7-13	RCOL2_02.04.02-2 S03
	14~~2~~.~~9~~26	Yes	Gravel & Concrete	7-13	
Unit 3 North Channel	-	-	-	-	
Center North Channel	~~4.64~~-	~~No~~-	~~Gravel & Concrete~~-	~~7-13~~-	RCOL2_02.04.02-2 S03
Unit 4 North Channel	-	-	-	-	
Unit 3 East Channel	~~6.90~~7.29	Yes	Gravel & Concrete	7-13	RCOL2_02.04.02-2 S03
Unit 3 Southeast Channel	~~13.46~~17.72*	Yes	Gravel & Concrete	7-13	
East Channel	~~17.50~~11.37*	Yes	Gravel & Concrete	7-13	
Off-site Channel	~~13.91~~23.44*	Yes	Gravel & Concrete	7-13	

Note:
*Based on discussions provided in Subsection 2.4.2.3, the maximum supercritical velocity in the Unit 3 Southeast Channel, East Channel, and the Off-site Channel does not adversely affect the safety-related structures.

CP COL 2.4(1)

Table 2.4.3-201
Watershed PMP (in) Depth-Area-Duration Relationship

Area (sq mi)	Duration (hr) 6	12	24	48	72
10	29.~~7~~83	~~35.3~~36.09	~~40.0~~41.33	~~45.0~~46.33	~~48.0~~49.01
200	22.~~2~~33	~~26.8~~27.19	32.~~0~~65	36.~~0~~76	~~39.6~~40.45
1000	~~15.9~~16.19	~~20.7~~21.18	~~25.8~~26.21	30.~~0~~62	33.~~4~~88
5000	9.35	13.~~1~~05	17.~~8~~90	22.~~0~~39	25.~~0~~42
10,000	7.11	10.35	14.~~4~~58	18.~~5~~67	21.~~0~~76
20,000	5.~~1~~20	8.23	11.~~5~~64	15.~~0~~27	~~17.8~~18.11

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Note: Values derived from the all-season PMP charts published in HMR 51.

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

RCOL2_02.04.02-2 S03

~~Duration (hr)~~6-hour increment (hour from 0)	Overall PMP (in.)	~~Incremental PMP (in)~~Squaw Creek Reservoir PMP (Basin 1 Only) (in.)
6	0.60	0.59
12	0.72	0.72
18	0.92	0.92~~1~~
24	1.26	1.27~~4~~
30	1.99	~~1.96~~2.05
36	4.80	5.~~10~~58
42	18.05	~~21.10~~25.01
48	2.82	2.98~~2~~
54	1.54	1.56~~2~~
60	1.06	1.0~~5~~6
66	0.81	0.80
72	0.66	0.65
Total	35.22	~~38.46~~43.19

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 145 degree storm orientation, centered near the centroid of the Squaw Creek watershed.~~Values derived based on HMR 51 and HMR 52 using HMR 52 computer software.

~~CP COL 2.4(1)~~

RCOL2_02.04.02-2 S03

Table 2.4.3-203 ~~(Sheet 1 of 3)~~
~~Squaw Creek Subbasin, Basin 2, Hourly Incremental PMP Estimates~~Not Used

~~Time (hr)~~	~~Hourly Incremental PMP (in)~~ ~~Basin 2~~
~~0100~~	~~0.10~~
~~0200~~	~~0.10~~
~~0300~~	~~0.10~~
~~0400~~	~~0.10~~
~~0500~~	~~0.10~~
~~0600~~	~~0.10~~
~~0700~~	~~0.11~~
~~0800~~	~~0.11~~
~~0900~~	~~0.11~~
~~1000~~	~~0.11~~
~~1100~~	~~0.11~~
~~1200~~	~~0.11~~
~~1300~~	~~0.12~~
~~1400~~	~~0.12~~
~~1500~~	~~0.12~~
~~1600~~	~~0.12~~
~~1700~~	~~0.12~~
~~1800~~	~~0.12~~
~~1900~~	~~0.13~~
~~2000~~	~~0.13~~
~~2100~~	~~0.15~~
~~2200~~	~~0.15~~
~~2300~~	~~0.15~~
~~2400~~	~~0.15~~
~~2500~~	~~0.15~~
~~2600~~	~~0.15~~
~~2700~~	~~0.21~~
~~2800~~	~~0.21~~
~~2900~~	~~0.21~~
~~3000~~	~~0.21~~
~~3100~~	~~0.21~~

~~CP COL 2.4(1)~~

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Table 2.4.3-203 ~~(Sheet 2 of 3)~~
~~Squaw Creek Subbasin, Basin 2, Hourly Incremental PMP Estimates~~Not Used

~~Time (hr)~~	~~Hourly Incremental PMP (in)~~ ~~Basin 2~~
~~3200~~	~~0.21~~
~~3300~~	~~0.29~~
~~3400~~	~~0.30~~
~~3500~~	~~0.32~~
~~3600~~	~~0.33~~
~~3700~~	~~0.35~~
~~3800~~	~~0.37~~
~~3900~~	~~0.60~~
~~4000~~	~~0.66~~
~~4100~~	~~0.73~~
~~4200~~	~~0.81~~
~~4300~~	~~0.92~~
~~4400~~	~~1.04~~
~~4500~~	~~1.42~~
~~4600~~	~~2.12~~
~~4700~~	~~3.10~~
~~4800~~	~~6.42~~
~~4900~~	~~2.67~~
~~5000~~	~~1.89~~
~~5100~~	~~0.56~~
~~5200~~	~~0.51~~
~~5300~~	~~0.47~~
~~5400~~	~~0.44~~
~~5500~~	~~0.41~~
~~5600~~	~~0.39~~
~~5700~~	~~0.25~~
~~5800~~	~~0.25~~
~~5900~~	~~0.25~~
~~6000~~	~~0.25~~
~~6100~~	~~0.25~~
~~6200~~	~~0.25~~

~~CP COL 2.4(1)~~

Table 2.4.3-203 ~~(Sheet 3 of 3)~~
~~Squaw Creek Subbasin, Basin 2, Hourly Incremental PMP Estimates~~Not Used

RCOL2_02.04.02-2 S03

~~Time (hr)~~	~~Hourly Incremental PMP (in)~~ ~~Basin 2~~
~~6300~~	~~0.18~~
~~6400~~	~~0.18~~
~~6500~~	~~0.18~~
~~6600~~	~~0.18~~
~~6700~~	~~0.18~~
~~6800~~	~~0.18~~
~~6900~~	~~0.13~~
~~7000~~	~~0.13~~
~~7100~~	~~0.13~~
~~7200~~	~~0.13~~

~~CP COL 2.4(1)~~

Table 2.4.3-204
~~Paluxy River Watershed 6-hr Incremental PMP Estimates~~Not Used

RCOL2_02.04.02-2 S03

~~Duration (hr)~~	~~Incremental PMP (in)~~
~~6~~	~~0.60~~
~~12~~	~~0.72~~
~~18~~	~~0.92~~
~~24~~	~~1.25~~
~~30~~	~~1.97~~
~~36~~	~~4.64~~
~~42~~	~~18.18~~
~~48~~	~~2.77~~
~~54~~	~~1.52~~
~~60~~	~~1.06~~
~~66~~	~~0.81~~
~~72~~	~~0.65~~
~~Total~~	~~35.08~~

~~Note: Values derived from HMR 51, HMR 52, and the use of HMR 52 computer software. Critical storm was determined to be 450 sq mi, with a 172 degree storm orientation, centered near the centroid of the upper Paluxy River watershed.~~

~~CP COL 2.4(1)~~

Table 2.4.3-205 ~~(Sheet 1 of 3)~~ ~~Paluxy River Watershed Subbasin Hourly Incremental PMP Estimates~~Not Used

RCOL2_02.04.02-2 S03

	~~Hourly Incremental PMP (in)~~	
~~Time (hr)~~	~~Basin 3~~	~~Basin 4~~
~~0100~~	~~0.10~~	~~0.10~~
~~0200~~	~~0.10~~	~~0.10~~
~~0300~~	~~0.10~~	~~0.10~~
~~0400~~	~~0.10~~	~~0.10~~
~~0500~~	~~0.10~~	~~0.10~~
~~0600~~	~~0.10~~	~~0.10~~
~~0700~~	~~0.11~~	~~0.11~~
~~0800~~	~~0.11~~	~~0.11~~
~~0900~~	~~0.11~~	~~0.11~~
~~1000~~	~~0.11~~	~~0.11~~
~~1100~~	~~0.11~~	~~0.11~~
~~1200~~	~~0.11~~	~~0.11~~
~~1300~~	~~0.12~~	~~0.12~~
~~1400~~	~~0.12~~	~~0.12~~
~~1500~~	~~0.12~~	~~0.12~~
~~1600~~	~~0.12~~	~~0.12~~
~~1700~~	~~0.12~~	~~0.12~~
~~1800~~	~~0.12~~	~~0.12~~
~~1900~~	~~0.14~~	~~0.14~~
~~2000~~	~~0.14~~	~~0.14~~
~~2100~~	~~0.16~~	~~0.15~~
~~2200~~	~~0.16~~	~~0.15~~
~~2300~~	~~0.16~~	~~0.15~~
~~2400~~	~~0.16~~	~~0.15~~
~~2500~~	~~0.16~~	~~0.15~~
~~2600~~	~~0.16~~	~~0.15~~
~~2700~~	~~0.21~~	~~0.21~~
~~2800~~	~~0.21~~	~~0.21~~
~~2900~~	~~0.21~~	~~0.21~~
~~3000~~	~~0.21~~	~~0.21~~

~~CP COL 2.4(1)~~

Table 2.4.3-205 ~~(Sheet 2 of 3)~~
~~Paluxy River Watershed Subbasin Hourly Incremental PMP Estimates~~Not Used

RCOL2_02.04.02-2 S03

	~~Hourly Incremental PMP (in)~~	
~~Time (hr)~~	~~Basin 3~~	~~Basin 4~~
~~3100~~	~~0.21~~	~~0.21~~
~~3200~~	~~0.21~~	~~0.21~~
~~3300~~	~~0.30~~	~~0.29~~
~~3400~~	~~0.31~~	~~0.31~~
~~3500~~	~~0.32~~	~~0.32~~
~~3600~~	~~0.34~~	~~0.34~~
~~3700~~	~~0.36~~	~~0.35~~
~~3800~~	~~0.38~~	~~0.38~~
~~3900~~	~~0.60~~	~~0.60~~
~~4000~~	~~0.65~~	~~0.65~~
~~4100~~	~~0.71~~	~~0.72~~
~~4200~~	~~0.79~~	~~0.80~~
~~4300~~	~~0.89~~	~~0.91~~
~~4400~~	~~1.00~~	~~1.03~~
~~4500~~	~~1.34~~	~~1.43~~
~~4600~~	~~1.99~~	~~2.16~~
~~4700~~	~~3.01~~	~~3.25~~
~~4800~~	~~6.85~~	~~7.27~~
~~4900~~	~~2.54~~	~~2.76~~
~~5000~~	~~1.77~~	~~1.92~~
~~5100~~	~~0.56~~	~~0.56~~
~~5200~~	~~0.51~~	~~0.51~~
~~5300~~	~~0.47~~	~~0.47~~
~~5400~~	~~0.44~~	~~0.44~~
~~5500~~	~~0.42~~	~~0.41~~
~~5600~~	~~0.40~~	~~0.39~~
~~5700~~	~~0.26~~	~~0.26~~
~~5800~~	~~0.26~~	~~0.26~~
~~5900~~	~~0.26~~	~~0.26~~
~~6000~~	~~0.26~~	~~0.26~~

~~CP COL 2.4(1)~~

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Table 2.4.3-205 ~~(Sheet 3 of 3)~~
~~Paluxy River Watershed Subbasin Hourly Incremental PMP Estimates~~Not Used

	~~Hourly Incremental PMP (in)~~	
~~Time (hr)~~	~~Basin 3~~	~~Basin 4~~
~~6100~~	~~0.26~~	~~0.26~~
~~6200~~	~~0.26~~	~~0.26~~
~~6300~~	~~0.18~~	~~0.18~~
~~6400~~	~~0.18~~	~~0.18~~
~~6500~~	~~0.18~~	~~0.18~~
~~6600~~	~~0.18~~	~~0.18~~
~~6700~~	~~0.18~~	~~0.18~~
~~6800~~	~~0.18~~	~~0.18~~
~~6900~~	~~0.14~~	~~0.14~~
~~7000~~	~~0.14~~	~~0.14~~
~~7100~~	~~0.14~~	~~0.14~~
~~7200~~	~~0.14~~	~~0.14~~

CP COL 2.4(1)

Table 2.4.3-207
Watershed Subbasin Characteristics

RCOL2_02.04.02-2 S03

Basin	Area (sq. mi.)	Baseflow (cfs)	L (mi)	L_{ca} (mi)	C_t	C_p
Basin 1a ~~& 1c~~	~~43.9~~37.81	~~42.01~~25.0	~~13.7~~12.98	6.~~5~~09	0.4	0.8
Basin 1b	~~20.3~~21.50	~~42.01~~15.0	5.~~3~~44	2.~~5~~96	0.4	0.8
Basin 1c	4.95	4.0	n/a*	n/a*	n/a*	n/a*
Basin 2	10.~~65~~72	~~6.97~~7.0	4.~~6~~52	~~3.0~~2.79	0.4	0.8
Basin 3	24.31	~~15.90~~16.0	4.~~9~~76	~~5.6~~3.83	0.4	0.8
Basin 4	410.~~0~~52	~~268.28~~269.0	~~59.3~~51.03	~~25.8~~24.60	0.4	0.8

*Rainfall to runoff transformation is instantaneous for the SCR (Basin 1c); unit hydrograph parameters are not applicable

L = length of the main stream from outlet to basin divide; used for Snyder's unit hydrograph development

L_{ca} = length along the main stream from ~~the~~ outlet to a point nearest to the ~~watershed~~basin centroid; used for Snyder's unit hydrograph development

C_t & C_p = Snyder's unit hydrograph coefficients; values resulting in higher water surface elevations at ~~the~~ CPNPP Units 3 and 4 were used.

Table 2.4.3-208

~~Squaw Creek Reservoir Watershed, Basin 1, 6-hr Incremental PMP Estimates~~Not Used

RCOL2_02.04.02-2 S03

~~Duration (hr)~~	~~Incremental PMP (in)~~
~~6~~	~~0.61~~
~~12~~	~~0.74~~
~~18~~	~~0.94~~
~~24~~	~~1.28~~
~~30~~	~~2.02~~
~~36~~	~~5.01~~
~~42~~	~~24.93~~
~~48~~	~~2.87~~
~~54~~	~~1.57~~
~~60~~	~~1.08~~
~~66~~	~~0.82~~
~~72~~	~~0.67~~
~~Total~~	~~42.53~~

~~Note:~~

~~Values derived from HMR 51, HMR 52, and the use of HMR 52 computer software. The critical storm was determined to be 100 sq mi with a 181 degree storm orientation, centered near the centroid of the Squaw Creek watershed.~~

RCOL2_02.04.02-2 S03

Table 2.4.3-209
~~Squaw Creek Reservoir Sub-basin, Basin 1, Hourly Incremental PMP Estimates~~Not Used

~~Time (hr)~~	~~Incremental PMP (in)~~	~~Time (hr)~~	~~Incremental PMP (in)~~
~~0100~~	~~0.10~~	~~3700~~	~~0.36~~
~~0200~~	~~0.10~~	~~3800~~	~~0.38~~
~~0300~~	~~0.10~~	~~3900~~	~~0.63~~
~~0400~~	~~0.10~~	~~4000~~	~~0.69~~
~~0500~~	~~0.10~~	~~4100~~	~~0.76~~
~~0600~~	~~0.10~~	~~4200~~	~~0.86~~
~~0700~~	~~0.11~~	~~4300~~	~~0.97~~
~~0800~~	~~0.11~~	~~4400~~	~~1.10~~
~~0900~~	~~0.11~~	~~4500~~	~~1.51~~
~~1000~~	~~0.11~~	~~4600~~	~~2.33~~
~~1100~~	~~0.11~~	~~4700~~	~~3.84~~
~~1200~~	~~0.11~~	~~4800~~	~~12.11~~
~~1300~~	~~0.12~~	~~4900~~	~~3.12~~
~~1400~~	~~0.12~~	~~5000~~	~~2.03~~
~~1500~~	~~0.12~~	~~5100~~	~~0.58~~
~~1600~~	~~0.12~~	~~5200~~	~~0.53~~
~~1700~~	~~0.12~~	~~5300~~	~~0.49~~
~~1800~~	~~0.12~~	~~5400~~	~~0.45~~
~~1900~~	~~0.14~~	~~5500~~	~~0.42~~
~~2000~~	~~0.14~~	~~5600~~	~~0.40~~
~~2100~~	~~0.16~~	~~5700~~	~~0.26~~
~~2200~~	~~0.16~~	~~5800~~	~~0.26~~
~~2300~~	~~0.16~~	~~5900~~	~~0.26~~
~~2400~~	~~0.16~~	~~6000~~	~~0.26~~
~~2500~~	~~0.16~~	~~6100~~	~~0.26~~
~~2600~~	~~0.16~~	~~6200~~	~~0.26~~
~~2700~~	~~0.21~~	~~6300~~	~~0.18~~
~~2800~~	~~0.21~~	~~6400~~	~~0.18~~
~~2900~~	~~0.21~~	~~6500~~	~~0.18~~
~~3000~~	~~0.21~~	~~6600~~	~~0.18~~
~~3100~~	~~0.21~~	~~6700~~	~~0.18~~
~~3200~~	~~0.21~~	~~6800~~	~~0.18~~
~~3300~~	~~0.30~~	~~6900~~	~~0.14~~
~~3400~~	~~0.31~~	~~7000~~	~~0.14~~
~~3500~~	~~0.33~~	~~7100~~	~~0.14~~
~~3600~~	~~0.34~~	~~7200~~	~~0.14~~

Table 2.4.3-210
Snyder’s Unit Hydrograph Characteristics

RCOL2_02.04.02-2 S03

	Qp (cfs)	~~T~~t_p (hr.)	T_b (hr.)	~~Qp (cfs)~~	W_{75} (hr.)	W_{50} (hr.)	~~Q75 (cfs)~~	~~Q50 (cfs)~~	~~Nonlinear Qp +20%~~
Basin 1a ~~& 1c~~	13,045	1.~~54~~84	4.~~61~~452	~~14,615~~	0.80~~3~~	1.40~~5~~	~~10,961~~	~~7308~~	~~17,538~~
Basin 1b	11,956	0.~~87~~921	2.~~61~~762	~~11,969~~	0.48~~5~~	0.7~~8~~4	~~8977~~	~~5985~~	~~14,363~~
Basin 2	6,415	0.8~~8~~56	2.~~64~~567	~~6203~~	0.44~~5~~	0.77~~9~~	~~4653~~	~~3102~~	~~7444~~
Basin 3	13,024	~~1.08~~ 0.956	~~3.24~~ 2.867	~~11,516~~	0.50~~7~~	0.~~99~~87	~~8367~~	~~5758~~	~~13,820~~
Basin 4	61,790	3.~~61~~402	9.~~23~~70	~~58,156~~	~~2.09~~1.96	3.~~65~~43	~~43,617~~	~~29,078~~	~~69,788~~

~~T~~t_p = ~~basin lag (hr);~~ $C_t\,(LL_{ca})^{0.3}$

where

t_p = basin lag time (hr.)

C_t = Snyder’s lag time coefficient

L = length of the main stream from the outlet to divide (mi.)

L_{ca} = length along the main stream from outlet to a point nearest to the ~~watershed~~basin centroid (mi.)

~~Tb = time base of the unit hydrograph (hr); 3+Tp/8 or 3 to 5 times Tp for small watersheds~~

Qp = ~~peak discharge of the unit hydrograph (cfs);~~ $640C_p\,A/$~~T~~t_p

where

Qp = peak discharge of the unit hydrograph (cfs)

C_p = Snyder’s peaking coefficient

A = drainage area (sq. mi.)

$T_b = 3+t_p/8$, or 3 to 5 times tp for small watersheds

T_b = time base of the unit hydrograph (hr.)

W_{75} = ~~unit hydrograph width at 75 percent;~~ $440(Qp/A)^{-1.08}$

W_{50} = ~~unit hydrograph width at 50 percent;~~ $770(Qp/A)^{-1.08}$

W_{75} = width of the unit hydrograph corresponding to 75 percent of the peak

W_{50} = width of the unit hydrograph corresponding to 50 percent of the peak

~~Q75 = unit hydrograph discharge at W75~~

~~Q50 = unit hydrograph discharge at W50~~

RCOL2_02.04.02-2 S03

Table 2.4.3-211
Overall Watershed, Basin 1, Hourly PMP Estimates

Duration (hours)	Incremental Depth (in)	Duration (hours)	Incremental Depth (in)	Duration (hours)	Incremental Depth (in)
1	0.097	25	0.1500	49	2.4320
2	0.097	26	0.1500	50	1.7420
3	0.097	27	0.2050	51	0.5450
4	0.097	28	0.2050	52	0.5020
5	0.097	29	0.2050	53	0.4640
6	0.097	30	0.2050	54	0.4320
7	0.107	31	0.2050	55	0.4070
8	0.107	32	0.2050	56	0.3870
9	0.107	33	0.2880	57	0.2510
10	0.107	34	0.2990	58	0.2510
11	0.107	35	0.3130	59	0.2510
12	0.107	36	0.3290	60	0.2510
13	0.118	37	0.3470	61	0.2510
14	0.118	38	0.3680	62	0.2510
15	0.118	39	0.5860	63	0.1730
16	0.118	40	0.6320	64	0.1730
17	0.118	41	0.6940	65	0.1730
18	0.118	42	0.7730	66	0.1730
19	0.132	43	0.8680	67	0.1730
20	0.132	44	0.9790	68	0.1730
21	0.150	45	1.3220	69	0.1320
22	0.150	46	1.9490	70	0.1320
23	0.150	47	2.8180	71	0.1320
24	0.150	48	5.6040	72	0.1320

Table 2.4.3-212
Overall Watershed, Basin 2, Hourly PMP Estimates

RCOL2_02.04.02-2 S03

Duration (hours)	Incremental Depth (in)	Duration (hours)	Incremental Depth (in)	Duration (hours)	Incremental Depth (in)
1	0.100	25	0.1530	49	2.4520
2	0.100	26	0.1530	50	1.7410
3	0.100	27	0.2090	51	0.5580
4	0.100	28	0.2090	52	0.5130
5	0.100	29	0.2090	53	0.4740
6	0.100	30	0.2090	54	0.4420
7	0.109	31	0.2090	55	0.4160
8	0.109	32	0.2090	56	0.3960
9	0.109	33	0.2950	57	0.2570
10	0.109	34	0.3060	58	0.2570
11	0.109	35	0.3200	59	0.2570
12	0.109	36	0.3370	60	0.2570
13	0.121	37	0.3550	61	0.2570
14	0.121	38	0.3760	62	0.2570
15	0.121	39	0.6020	63	0.1770
16	0.121	40	0.6500	64	0.1770
17	0.121	41	0.7150	65	0.1770
18	0.121	42	0.7940	66	0.1770
19	0.135	43	0.8890	67	0.1770
20	0.135	44	0.9990	68	0.1770
21	0.153	45	1.3310	69	0.1350
22	0.153	46	1.9520	70	0.1350
23	0.153	47	2.8720	71	0.1350
24	0.153	48	6.0730	72	0.1350

RCOL2_02.04.02-2 S03

Table 2.4.3-213
Overall Watershed, Basin 3, Hourly PMP Estimates

Duration (hours)	Incremental Depth (in)	Duration (hours)	Incremental Depth (in)	Duration (hours)	Incremental Depth (in)
1	0.102	25	0.1570	49	2.7270
2	0.102	26	0.1570	50	1.9000
3	0.102	27	0.2140	51	0.5760
4	0.102	28	0.2140	52	0.5270
5	0.102	29	0.2140	53	0.4860
6	0.102	30	0.2140	54	0.4510
7	0.112	31	0.2140	55	0.4240
8	0.112	32	0.2140	56	0.4040
9	0.112	33	0.3010	57	0.2620
10	0.112	34	0.3130	58	0.2620
11	0.112	35	0.3270	59	0.2620
12	0.112	36	0.3440	60	0.2620
13	0.123	37	0.3630	61	0.2620
14	0.123	38	0.3850	62	0.2620
15	0.123	39	0.6250	63	0.1810
16	0.123	40	0.6800	64	0.1810
17	0.123	41	0.7510	65	0.1810
18	0.123	42	0.8390	66	0.1810
19	0.138	43	0.9430	67	0.1810
20	0.138	44	1.0630	68	0.1810
21	0.157	45	1.4350	69	0.1380
22	0.157	46	2.1410	70	0.1380
23	0.157	47	3.2230	71	0.1380
24	0.157	48	7.2880	72	0.1380

RCOL2_02.04.02-2 S03

Table 2.4.3-214
Overall Watershed, Basin 4, Hourly PMP Estimates

Duration (hours)	Incremental Depth (in)	Duration (hours)	Incremental Depth (in)	Duration (hours)	Incremental Depth (in)
1	0.100	25	0.1540	49	2.7360
2	0.100	26	0.1540	50	1.9190
3	0.100	27	0.2100	51	0.5680
4	0.100	28	0.2100	52	0.5200
5	0.100	29	0.2100	53	0.4780
6	0.100	30	0.2100	54	0.4440
7	0.109	31	0.2100	55	0.4170
8	0.109	32	0.2100	56	0.3970
9	0.109	33	0.2950	57	0.2570
10	0.109	34	0.3070	58	0.2570
11	0.109	35	0.3210	59	0.2570
12	0.109	36	0.3370	60	0.2570
13	0.121	37	0.3560	61	0.2570
14	0.121	38	0.3780	62	0.2570
15	0.121	39	0.6150	63	0.1770
16	0.121	40	0.6680	64	0.1770
17	0.121	41	0.7390	65	0.1770
18	0.121	42	0.8270	66	0.1770
19	0.135	43	0.9320	67	0.1770
20	0.135	44	1.0540	68	0.1770
21	0.154	45	1.4400	69	0.1350
22	0.154	46	2.1600	70	0.1350
23	0.154	47	3.2060	71	0.1350
24	0.154	48	6.9340	72	0.1350

RCOL2_02.04.02-2 S03

Table 2.4.3-215
SCR Watershed, Hourly PMP Estimates

Duration (hours)	Incremental Depth (in)	Duration (hours)	Incremental Depth (in)	Duration (hours)	Incremental Depth (in)
1	0.098	25	0.1530	49	3.0880
2	0.098	26	0.1530	50	2.0530
3	0.098	27	0.2110	51	0.6150
4	0.098	28	0.2110	52	0.5540
5	0.098	29	0.2110	53	0.5040
6	0.098	30	0.2110	54	0.4640
7	0.108	31	0.2110	55	0.4330
8	0.108	32	0.2110	56	0.4120
9	0.108	33	0.3010	57	0.2610
10	0.108	34	0.3140	58	0.2610
11	0.108	35	0.3290	59	0.2610
12	0.108	36	0.3470	60	0.2610
13	0.120	37	0.3680	61	0.2610
14	0.120	38	0.3920	62	0.2610
15	0.120	39	0.6870	63	0.1770
16	0.120	40	0.7720	64	0.1770
17	0.120	41	0.8660	65	0.1770
18	0.120	42	0.9700	66	0.1770
19	0.134	43	1.0820	67	0.1770
20	0.134	44	1.2040	68	0.1770
21	0.153	45	1.5720	69	0.1340
22	0.153	46	2.3400	70	0.1340
23	0.153	47	3.8060	71	0.1340
24	0.153	48	12.1460	72	0.1340