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Key Site Parameters

CP COL 2.1(1)
CP COL 2.2(1)
CP COL 2.3(1)
CP COL 2.3(2)
CP COL 2.3(3)
CP COL 2.4(1)
CP COL 2.5(1)

CTS-01125

~~A/B releases (reactor coolant system sample line~~ ~~0-8 hrs~~ ~~8-24 hrs~~ ~~1-4 days~~ ~~4-30 days~~	 ~~4.9×10^{-3} s/m^3~~ ~~2.9×10^{-3} s/m^3~~ ~~1.8×10^{-3} s/m^3~~ ~~8.1×10^{-4} s/m^3~~	~~Dispersion of releases from the reactor coolant sampling line are bounded by the dispersion values for the plant vent.~~
~~Air lock releases in containment~~ ~~0-8 hrs~~ ~~8-24 hrs~~ ~~1-4 days~~ ~~4-30 days~~	 ~~6.4×10^{-3} s/m^3~~ ~~3.8×10^{-3} s/m^3~~ ~~2.4×10^{-3} s/m^3~~ ~~1.1×10^{-3} s/m^3~~	~~χ/Q values for the air lock releases in containment are bounded by the χ/Q for the Containment Shell release.~~

Hydrologic Engineering

Parameter Description	Parameter Value	
	DCD	**CPNPP 3 and 4**
Maximum flood (or tsunami) level	1 ft below plant grade	790.9 ft msl for SCR ~~820.83~~820.90 ft msl for a Local Intense Precipitation at units 3 and 4 site.
Maximum rainfall rate (hourly)	19.4 in/hr for seismic category I/II structures	19.0 in/hr
Maximum rainfall rate (short-term)	6.3 in/5 min for seismic category I/II structures	6.2 in/5 min
Maximum groundwater level	1 ft below plant grade	1 ft below plant grade

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CP COL 2.2(1)

recorded water surface elevation of 636.86 ft msl occurred on April 17, 1908 and corresponded to the maximum recorded discharge of 59,000 cfs (Reference 2.4-225). The annual peak stage and discharge measurements for the period of record are provided in Table 2.4.2-202. The datum for USGS gage (08091500) is reported in NAD27 and NGDV29.

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 NGDV29.

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 NAVD88.

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 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

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. RCOL2_02.04.02-2

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 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.

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 ~~type of~~summary results of the events evaluated to determine the worst potential flood ~~include~~are provided as follows: RCOL2_02.04.02-2

- Probable maximum precipitation (PMP) on the total watershed and critical sub-watersheds, including seasonal variations and potential consequent dam failures, with a corresponding water surface elevation of ~~790.9~~793.66 ft msl (discussed in Subsection 2.4.3). RCOL2_02.04.02-2

- Dam failures, including a postulated domino-type failures of three upstream dams coincident with the Probable Maximum Flood (PMF), with a corresponding water surface level of ~~774.99~~760.71 ft msl (discussed in Subsection 2.4.4). RCOL2_02.04.02-2

- Two year coincident wind waves with a corresponding water surface level of ~~807.87~~810.64 ft msl (discussed in Subsection 2.4.3). RCOL2_02.04.02-2

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

The maximum flood level at CPNPP Units 3 and 4 is elevation ~~790.9~~793.66 ft msl. RCOL2_02.04.02-2 This elevation would result from a PMP on the Squaw Creek watershed, as described in Subsection 2.4.3. Coincident wind waves would create maximum waves of ~~16.97~~16.98 ft resulting in a design basis flood elevation of ~~807.87~~810.87 ft msl. CPNPP Units 3 and 4 safety-related plant elevation is 822 ft msl, providing more than ~~14~~11 ft of freeboard under the worst potential flood considerations. RCOL2_02.04.02-2

2.4.2.3 Effects of Local Intense Precipitation

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

The local intense PMP is defined by Hydrometeorological Report No. 51 (HMR 51) and No. 52 (HMR 52). PMP values for durations from 6-hr. to 72-hr. are determined using the procedures as described in HMR No. 51 for areas of 10-sq

T_t= Sheet flow travel time (hr)

n = Mannings Friction Factor

L = Flow Length of the Runoff which is not greater than 300 (ft)

P_2 = Rainfall Depth of the 2 year 24 hour rainfall event (in)

S = Slope of the Runoff Travel Path (ft/ft)

T_t is calculated using the following equation for shallow concentrated flow and channel flow:

The Technical Release 55 minimum Manning's Friction Factors are 0.011 for smooth concrete surfaces and 0.15 for grass. Sheet flow was evaluated using Manning's Friction Factors of 0.01 smooth concrete surfaces and 0.075 for grass. Using smaller values is conservative because minimizing the friction factor also minimizes the travel time. The shorter travel times result in a greater intensity and peak runoff. RCOL2_02.04.02-2

Shallow concentrated flow and channel flow $T_t = \frac{L}{3600V}$ (Reference 2.4-221)

(Equation 2)

where:

T_t= Shallow concentrated and channel flow travel time (hr)

L = Flow Length (ft)

V= Velocity of flow (fps)

For shallow concentrated flow over paved areas:

$V = 20.328 \cdot S^{0.5}$ (Reference 2.4-221)

S = Slope of the Runoff Travel Path (ft/ft)

For open channel flow, velocity is determined using the Manning's formula:

$V = \frac{1}{n}K \cdot r^{\frac{2}{3}} \cdot s^{\frac{1}{2}}$ (Reference 2.4-221)

where:

V = Velocity of open channel flow (fps)

K = constant = 1.49 for English units

r = hydraulic radius = a/ p_w (ft)

a = cross sectional flow area in (sq ft)

p_w = wetted perimeter (ft)

s = slope of hydraulic grade line (ft/ft)

n = Manning's roughness coefficient for open channel flow

For open channel flow, according to Chow (Reference 2.4-223), the minimum Manning's roughness coefficient for excavated or natural channels is 0.016. Open channel flow was evaluated using a Manning's roughness coefficient of 0.015. Using smaller values is conservative because minimizing the roughness coefficient increases the velocity. Increased velocity minimizes the travel time. The shorter travel times result in a greater intensity and peak runoff. RCOL2_02.04.02-2

The rational method was used to determine peak runoff rates for the drainage sub basins. The rational method is given by the equation:

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

where:

Q = Runoff (cfs)

C = Unitless coefficient of runoff

i = Intensity (in/hr)

A = Drainage area (ac)

No runoff losses were assumed. Therefore, the runoff coefficient was assumed equal to one. The weir equation is used to determine the PMF elevation for the peak runoff rate from the sub basins. A tail water elevation at ~~790.9~~793.66 ft msl from a PMF at the SCR was considered for the local site analysis. RCOL2_02.04.02-2

The equation for weir is given by the equation:

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

where:

Q = runoff (cfs)

C_d= Overtopping discharge coefficient (Reference 2.4-223)

L = Crest length of overflow section (ft)

HW_r= Head water elevation for the weir (ft)

Site drainage area details are tabulated in Table 2.4.2-207. Resulting PMP water surface elevation at the points of discharge from the local site analysis are shown in Table 2.4.2-208. Drainage areas 1, 2, and 3 result in a maximum water surface elevation of ~~820.83~~820.90 ft msl at the point of discharge W1. CPNPP Units 3 and 4 safety-related structures are located above the effects of local intense precipitation at plant elevation 822 ft msl. RCOL2_02.04.02-2

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.

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

Point Of Discharge	Drainage Sub Basins	Peak Runoff at Point of Discharge (cfs)	Crest Length L (ft)	Tailwater Elevation (ft msl)	Discharge Coefficient	Weir Elevation (ft msl)	Over Topping Depth Hw (ft)	Resulting Water Surface Elevation (ft msl)
W1	1+2+3	1,195.44	560	~~790.9~~793.66	~~2.80~~2.50	820	~~0.83~~0.90	~~820.83~~820.90
W2	4+5+6	1,166.70	365	~~790.9~~793.66	~~2.83~~2.50	815	~~1.09~~1.18	~~816.09~~816.18
W3	7+8	2,384.49	490	~~790.9~~793.66	~~2.90~~2.50	810	~~1.44~~1.56	~~811.44~~811.56
W4	9+10+11	4,127.68	315	~~790.9~~793.66	~~3.02~~2.50	814	~~3.14~~3.02	~~817.14~~817.02

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