



Serial: NPD-NRC-2009-230
November 19, 2009

10 CFR 52.79

U.S. Nuclear Regulatory Commission
Attention: Document Control Desk
Washington, D.C. 20555-0001

**SHEARON HARRIS NUCLEAR POWER PLANT, UNITS 2 AND 3
DOCKET NOS. 52-022 AND 52-023
SUPPLEMENT 2 TO RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION LETTER
NO. 023 RELATED TO PROBABLE MAXIMUM FLOOD ON STREAMS AND RIVERS**

- References:
1. Letter from Manny Comar (NRC) to James Scarola (PEC), dated September 26, 2008, "Request for Additional Information Letter No. 023 Related to SRP Section 02.04.03 for the Harris Units 2 and 3 Combined License Application"
 2. Letter from James Scarola (PEC) to U. S. Nuclear Regulatory Commission (NRC), dated October 31, 2008, "Response to Request for Additional Information Letter No. 023 Related to Probable Maximum Flood on Streams and Rivers," Serial: NPD-NRC-2008-054
 3. Letter from Garry D. Miller (PEC) to U. S. Nuclear Regulatory Commission (NRC), dated April 1, 2009, "Supplement 1 to Response to Request for Additional Information Letter No. 023 Related to Probable Maximum Flood on Streams and Rivers," Serial: NPD-NRC-2009-056

Ladies and Gentlemen:

Progress Energy Carolinas, Inc. (PEC) hereby submits a supplemental response to the Nuclear Regulatory Commission's (NRC) request for additional information provided in the referenced letter.

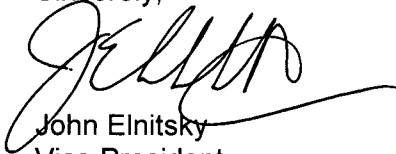
A revised response to NRC question 02.04.03-4 is provided in the enclosure. The enclosure also identifies changes that will be made in a future revision of the Shearon Harris Nuclear Power Plant Units 2 and 3 application.

If you have any further questions, or need additional information, please contact Bob Kitchen at (919) 546-6992, or me at (727) 820-4481.

I declare under penalty of perjury that the foregoing is true and correct.

Executed on November 19, 2009.

Sincerely,

A handwritten signature in black ink, appearing to read "John Elnitsky", with a long horizontal flourish extending to the right.

John Elnitsky
Vice President
Nuclear Plant Development

Enclosure/Attachments

cc : U.S. NRC Region II, Regional Administrator
U.S. NRC Resident Inspector, SHNPP Unit 1
Mr. Brian Hughes, U.S. NRC Project Manager

**Shearon Harris Nuclear Power Plant Units 2 and 3
Supplement 2 to Response to NRC Request for Additional Information Letter No. 023
Related to SRP Section 02.04.03 for the Combined License Application,
Dated September 26, 2008**

<u>NRC RAI #</u>	<u>Progress Energy RAI #</u>	<u>Progress Energy Response</u>
02.04.03-4	H-0485	Revised response enclosed – see following pages

NRC Letter No.: HAR-RAI-LTR-023

NRC Letter Date: September 26, 2008

NRC Review of Final Safety Analysis Report

NRC RAI NUMBER: 02.04.03-4

Text of NRC RAI:

Based on discussions with US Army Corps of Engineers personnel, the NRC staff have determined that ETL 1110-2-221 "Wave Runup and Wind Setup on Reservoir Embankments" has been superseded by guidance found in EM 1110-2-1420 "Hydrologic Engineering Requirements for Reservoirs" and guidance found in the Coastal Engineering Manual (EM 1110-2-1100). The staff requests the applicant to show how their current methodology results in a conservative estimate of wind-wave effects.

PGN RAI ID #: H-0485

PGN Response to NRC RAI:

After Progress Energy Carolina's (PEC's) initial response to this Request for Additional Information (RAI), the U.S. Nuclear Regulatory Commission (NRC) requested a follow-up meeting to clarify its request concerning the probable maximum flood (PMF) estimate. A meeting between PEC and the NRC was held in Raleigh, North Carolina, on February 10 and 11, 2009. During the meeting, PEC summarized the methodology and approach used to develop the PMF estimate for the site and described why that estimate was considered highly conservative. As a result of the meeting, PEC provided a response to this RAI and provided revised responses to RAI 02.04.03-1 and RAI 02.04.03-3 on April 1, 2009, in order to do the following:

- Clarify the methodology used.
- Ensure that the PMF estimate was both conservative and representative of the site.
- Include the use of the U.S. Army Corps of Engineers' (USACE's) *Coastal Engineering Manual* to account for wind setup and wave run-up, which added additional conservatism to the PMF analysis for Shearon Harris Nuclear Power Plant Units 2 and 3 (HAR).

During the course of the analyses used for RAIs 02.04.03-1, 02.04.03-3, and 02.04.03-4, it was noted that the additional incorporated conservatism could result in potential PMF impacts at existing safety-related structures. Accordingly, PEC performed a comprehensive evaluation of potential PMF mitigation strategies and identified the following two strategies that would result in no potential PMF impacts at safety-related structures, regardless of the additional incorporated conservatism. Currently, two water control structures consisting of open spillways with crest elevations at 220 feet National Geodetic Vertical Datum of 1929 (NGVD29) are present at the Main Dam, and the top of the Main Dam is at an elevation of 260 feet NGVD29.

- Option 1: Raise the existing open spillway to 240 feet NGVD29 in both spans and add an emergency spillway with a crest at 243 feet NGVD29.
- Option 2: Raise the existing open spillway to 240 feet NGVD29 in one span and install a Tainter gate in the second span with a spillway crest at 220 feet NGVD29.

These two PMF mitigation strategies were modeled and are incorporated into this revised RAI response. To determine the length of emergency spillway in Option 1, different scenarios using various emergency spillway lengths were evaluated. To determine the upstream water elevation at which the Tainter gate is completely open in Option 2, different scenarios were evaluated using various lake level target elevations to begin opening the Tainter gate.

The following information is intended to supersede and replace the previous response to this RAI, which was submitted to the NRC by letter dated April 1, 2009 (NPD-NRC-2009-056). The PMF stillwater elevations obtained for various scenarios for Option 1 and Option 2 have been summarized in RAI 02.04.03-1 and RAI 02.04.03-3. In order to be conservative, the stillwater PMF elevations corresponding to the 25 percent peaking for the various scenarios have been considered to determine the effect of wind setup and wave runup in the vicinity of safety-related structures at the HAR site, Shearon Harris Nuclear Power Plant Unit 1 (HNP) site, and at the existing Main Dam and Auxiliary Dam. **Table 1** presents the worst case stillwater PMF elevations for Options 1 and 2.

Table 1. Worst Case Stillwater PMF Elevations for Options 1 and 2

Scenario	(Option 1)Emergency Spillway Length (ft) / (Option 2) Water Surface Elevation (ft NGVD29) at which the Tainter Gate is Opened	HEC-HMS Results		HEC-RAS Results	Max of HEC-HMS and HEC-RAS
		PMF Elevation AUX Reservoir (ft NGVD29)	PMF Elevation Main Reservoir (ft NGVD29)	PMF Elevation Main Reservoir (ft NGVD29)	Selected PMF Elevation Main Reservoir (ft NGVD29)
OPTION 1 (with Emergency Spillway)					
1	400	256.53	252.96	253.33	253.33
2	500	256.55	252.42	252.74	252.74
3	600	256.58	251.94	252.26	252.26
OPTION 2 (with Tainter Gate)					
1	243	256.54	252.78	253.18	253.18
2	242	256.56	252.25	252.76	252.76
3	241	256.59	251.73	252.20	252.20

Effects of wind setup and wave runup have been calculated for tabulated Scenarios 1 through 3 for Options 1 and 2. A discussion of each of these is provided below.

Wind Setup and Wave Runup - Impacts in the Vicinity of HAR Safety-Related Structures

As discussed in Final Safety Analysis Report (FSAR) Subsection 2.4.2.2, safety-related structures and facilities for the HAR site are protected against floods and flood waves caused by probable maximum events, such as the PMF and the probable maximum hurricane (PMH). Coincident wind wave activity was evaluated for Shearon Harris Nuclear Power Plant Unit 2 (HAR 2), Shearon Harris Nuclear Power Plant Unit 3 (HAR 3), HNP, the Auxiliary Dam, and the Main Dam. For these locations, the USACE's *Coastal Engineering Manual, Engineer Manual*

1110-2-1100 (Part II) (USACE, 2006) was strictly followed to determine wave runup. It is important to note that the Auxiliary Dam and Main Dam are not safety-related structures in the context of the HAR site. Both of these non-safety-related structures were considered in the analysis because they are important in the context of the HNP site.

In order to determine wind setup and wave runup for a given location, the following data were required:

- Water body bathymetry data
- Critical fetch distances
- Over-wind speed averaged for an appropriate duration
- Site characteristics, such as type and material of protection, and slope

Bathymetry Data

PEC performed detailed bathymetric surveys to establish the current geometry of the Main Reservoir. Thousands of depth-to-bottom measurements were collected during this bathymetric survey study. These data were compiled into single geographic information system (GIS) point coverage. ArcGIS three-dimensional (3-D) analyst Kriging sampling interpolation was used to generate a 3-D surface from the mass point data. For further detail on the processing of bathymetric data, refer to RAI 02.04.03-1.

The locations of interest for determining the wind-wave activity for HAR 2, HAR 3, HNP, the Auxiliary Dam, and the Main Dam are shown on **Figures 1 through 4**. For wind and wave calculation purposes, water depth at various locations was determined using the bottom elevation of the lake near the location of interest and the stillwater PMF elevation for various scenarios of both options. **Table 2** presents the lake bottom elevations that were used in the wind-wave activity analysis.

Table 2. Histogram Data Main Reservoir Depths

Location	Line ID	Lake Bottom Elevation (ft NGVD29)
HAR 2	1	220
	2	220
	3	240
	4	240
	5	240
HAR 3	1	220
	2	220
	3	220
	4	240
	5	240
	6	240
	7	240
HNP	1	240
	2	220
	3	220
Main Dam	1	220
	2	220
Auxiliary Dam	3	240
	4	240

Fetch Distances

Fetch is the length of water surface exposed to wind during the generation of waves. Fetch is an important characteristic of open water because a longer fetch can result in larger wind-generated waves. According to *EM 1110-2-1100* (USACE, 2002), straight line fetch distances were used in wave runup calculations. Thus, straight line fetch distances were determined using the site topographic and reservoir bathymetry data as shown on **Figures 1 through 4**. **Tables 3 through 6** provide the overwater fetch distances for the HAR 2, HAR 3, HNP, Auxiliary Dam, and Main Dam locations, as identified on **Figures 1 through 4**. The critical fetch distances for the HAR 2, HAR 3, HNP, Auxiliary Dam, and Main Dam locations are 0.93, 0.85, 4.33, 4.29, and 4.29 miles, respectively.

Table 3. Fetch Distances for HAR 2

Line ID	Fetch Dist (mi)
1	0.93 ^a
2	0.88
3	0.87
4	0.40
5	0.50
6	0.50
7	0.50

Notes:

^aCritical Fetch Distance = 0.93 mi

Table 4. Fetch Distances for HAR 3

Line ID	Fetch Dist (mi)
1	0.85 ^a
2	0.72
3	0.61
4	0.64
5	0.55

Notes:

^a Critical Fetch Distance = 0.85 mi

Table 5. Fetch Distances for HNP

Line ID	Fetch Dist (mi)
1	0.76
2	4.33 ^a
3	2.73

Notes:

^a Critical Fetch Distance = 4.33 mi

Table 6. Fetch Distances for Auxiliary and Main Dams

Line ID	Fetch Dist (mi)
1	4.29 ^a
2	4.29 ^a
3	1.17
4	1.08

Notes:

^a Critical Fetch Distance = 4.29 mi

Figure 1. Direct Fetch for the HAR 2 Safety-Related Structures



Figure 2. Direct Fetch for the HAR 3 Safety-Related Structures



Figure 3. Direct Fetch for the Main and Auxiliary Dams



Figure 4. Direct Fetch for the HNP Site

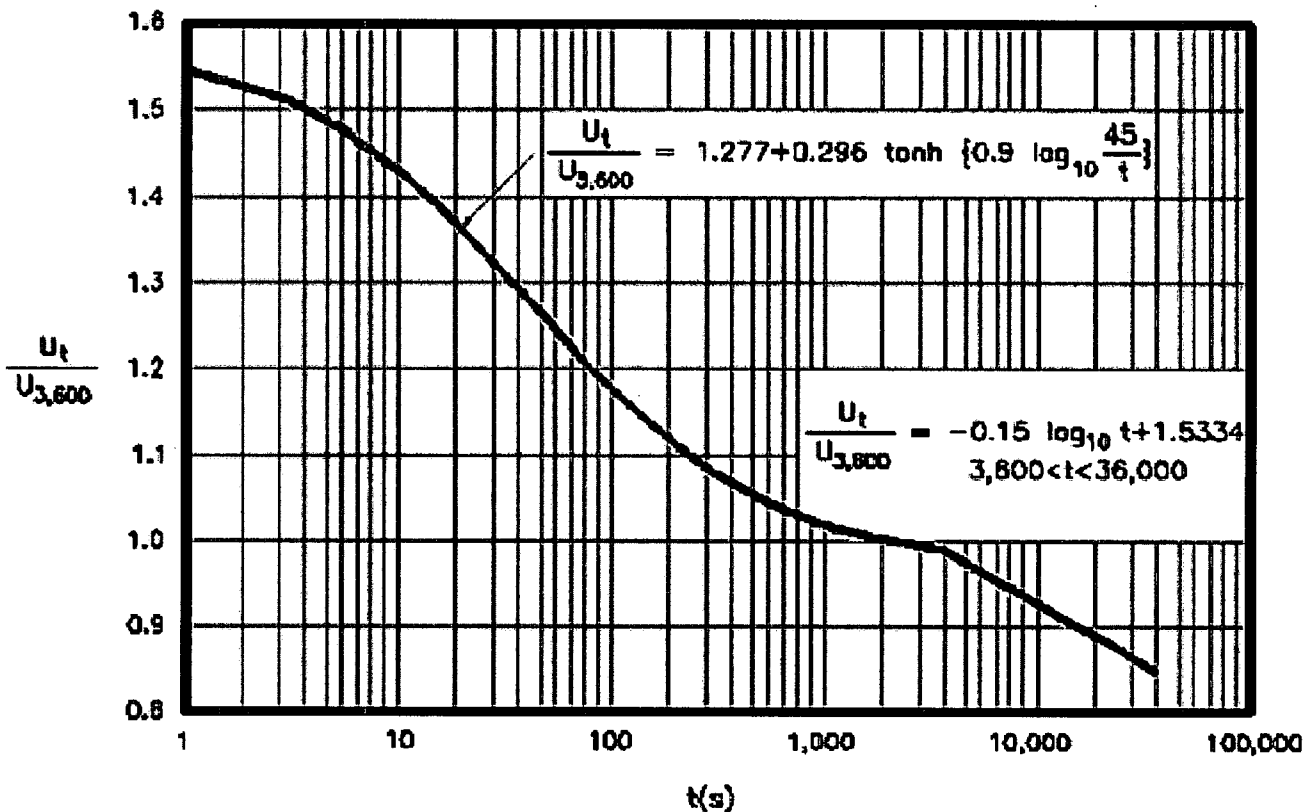


Overwater Wind Speed

According to American National Standards Institute/American Nuclear Society (ANSI/ANS) 2.8-1992, the 2-year wind speed should be used while conducting the coincident wind-wave activity analysis. The 2-year wind speed at the HAR site is 50 miles per hour (mph). Before using this wind speed in the calculation of wave runup, several adjustments were applied following the procedure outlined in the *EM 1110-2-1100* (Part II) (USACE, 2006). The step-by-step procedure is as follows:

1. Standard measurements should be collected at 10 meters above ground surface. Since the wind speed obtained from ANSI/ANS 2.8.1992 was measured at 30 feet (about 10 meters) above ground, no adjustment needed to be applied.
2. Using the 2-year wind speed of 50 mph and Figure II-2-1 (Figure 5) of *EM 1110-2-1100* (Part II), the 1-hour wind speed was calculated as 50 mph.

Figure 5. Ratio of Wind Speed of any Duration, U_t to the 1-Hour Wind Speed U_{3600}



3. Overwater wind speeds at various locations were then determined by applying a correction for transition from land to water. This correction factor was determined using Figure II-2-7 of *EM 1110-2-1100* (Part II) (Figure 6). (Note: The HAR site is approximately 140 miles from the coastal line, hence, Figure II-2-7 of *EM 1110-2-1100* [Part II] is applicable.)

According to Figure II-2-7 (**Figure 6**), the correction factor R_L is given as follows:

$$U_W = R_L * U_L \quad (1)$$

where:

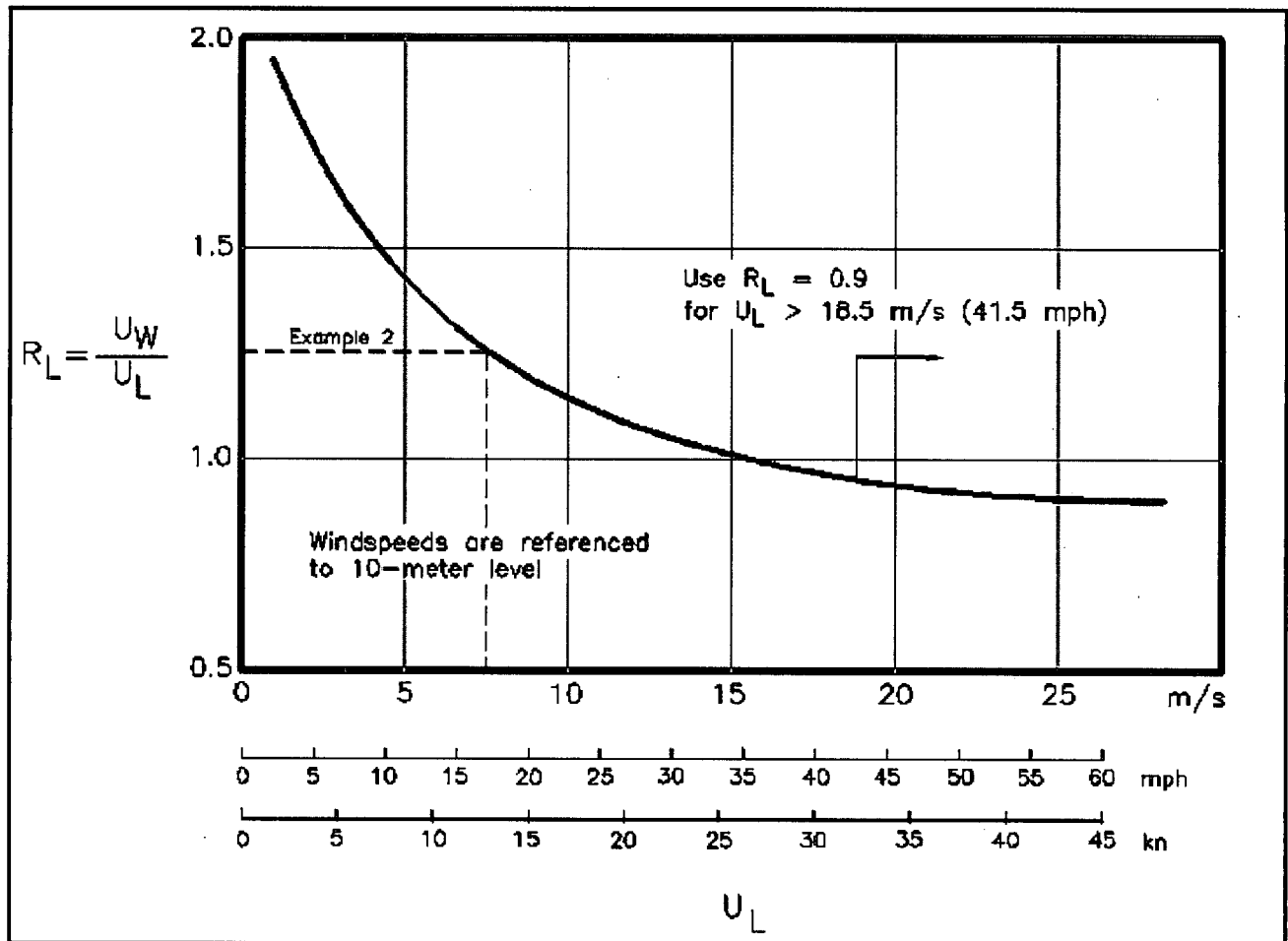
U_W is the overwater wind speed,

U_L is the overland wind speed, and

R_L is a correction factor which is equal to 0.9 for $U_L > 41.5$ mph.

In this case, the overland wind speed, U_L , is 50.0 mph (22.4 meter per second [m/s]). As such, the correction factor, R_L , is equal to 0.9. However, in an effort to be conservative, a correction factor, R_L , equal to 1.0 was used for this analysis.

Figure 6. Ratio R_L of Wind Speed Overwater U_W to Wind Speed Overland U_L as a Function of Wind Speed Overland U_L (after Resio and Vincent [1977])



- The wind speed is now corrected according to the appropriate averaging duration. When wind occurs with essentially constant direction over a fetch for sufficient time to achieve steady-state, fetch-limited values, simplified wave predictions can provide accurate estimates of wave conditions. The time required to accomplish fetch-limited wave development for short fetches was calculated as follows:

$$t_{x,u} = 77.23 \frac{X^{0.67}}{u^{0.34} g^{0.33}} \quad (2)$$

where:

$t_{x,u}$ is the time required for waves crossing a fetch of length X under a wind of velocity u to become fetch-limited.

The resulting averaging time interval, $t_{x,u}$ was then used in conjunction with Figure II-2-1 of *EM 1110-2-1100* (Part II) (**Figure 5**) in order to determine the appropriate wind speed for various locations. **Table 7** presents calculated corrections of wind averaging intervals for various fetch lines directed toward HAR 2, HAR 3, HNP, the Auxiliary Dam, and the Main Dam.

Table 7. Correction for Wind Averaging Interval

Location	Line ID	St. Line Fetch, X (mi)	St. Line Fetch, X (km)	$t_{x,u}$ (s)	Correction Factor	Wind Speed u (m/s)
HAR 2	1	0.85	1.36	1592	1.02	22.56
	2	0.72	1.16	1431	1.02	22.63
	3	0.61	0.98	1280	1.02	22.70
	4	0.64	1.02	1314	1.02	22.68
	5	0.55	0.88	1192	1.02	22.74
HAR 3	1	0.93	1.49	1693	1.01	22.53
	2	0.88	1.41	1632	1.01	22.55
	3	0.87	1.39	1616	1.01	22.55
	4	0.40	0.80	1116	1.03	22.79
	5	0.50	0.80	1118	1.03	22.79
	6	0.50	0.80	1119	1.03	22.79
	7	0.50	0.80	1116	1.03	22.79
HNP	1	0.76	1.22	1482	1.02	22.60
	2	4.33	6.92	4738	0.98	21.82
	3	2.73	4.36	3476	1.00	22.23
Dams	1	4.29	6.87	4713	0.98	21.83
	2	4.29	6.87	4713	0.98	21.83
	3	1.17	1.88	1975	1.01	22.45
	4	1.08	1.73	1871	1.01	22.48

Notes:
km = kilometer
mi = mile
m/s = meter per second
s = second

Site Characteristics

The HAR is surrounded by the Thomas Creek Branch of the Main Reservoir on the east side and by the Auxiliary Reservoir on the west side. FSAR Figure 2.4.1-205 shows the planned site drainage plan and indicates that the HAR will have permeable natural land area between the developed site and the water bodies. Using site-specific topographic and bathymetry data, the land slope adjacent to the Thomas Creek Branch of the Main Reservoir and Auxiliary Reservoir were determined; these slopes were 0.09 and 0.13 for the east and west sides of HAR 2 and HAR 3, respectively.

The upstream faces of the Main Dam and Auxiliary Dam are protected by riprap with slopes of 1(V): 2(H) and 1(V):2.5(H), respectively. On the plant island, the southerly fill (Line # 1, Figure 4, directing toward Plant Island from the Auxiliary Reservoir) and embankment faces of the plant island which face the Main Reservoir (Line # 2 and 3, Figure 4, directing towards Plant Island from the Main Reservoir) are protected by sacrificial spoil fill. The fill directed by Line # 1 has a slope of 1(V):5 (H), whereas the fills directed by Lines 2 and 3 have slope of 1(V):10 (H).

Wave Runup

Having determined the estimate of winds for wave prediction, wave runup for various fetch lines directed toward HAR 2, HAR 3, HNP, the Auxiliary Dam, and the Main Dam were calculated according to the step-by-step procedure given in the *EM 1110-2-1100* (Part VI) (USACE, 2006). The step-by-step procedure is as follows:

1. Using the previously determined fetch lengths and wind speeds, estimates of significant wave heights were obtained using the deepwater nomogram for the fetch limited wave heights given in the *EM 1110-2-1100* (Part II) (**Figure 7**).
2. Similarly, estimates of peak wave periods were obtained using the deepwater nomogram for the fetch limited wave periods given in the *EM 1110-2-1100* (Part II) (**Figure 8**).

Figure 7. Fetch-Limited Wave Heights

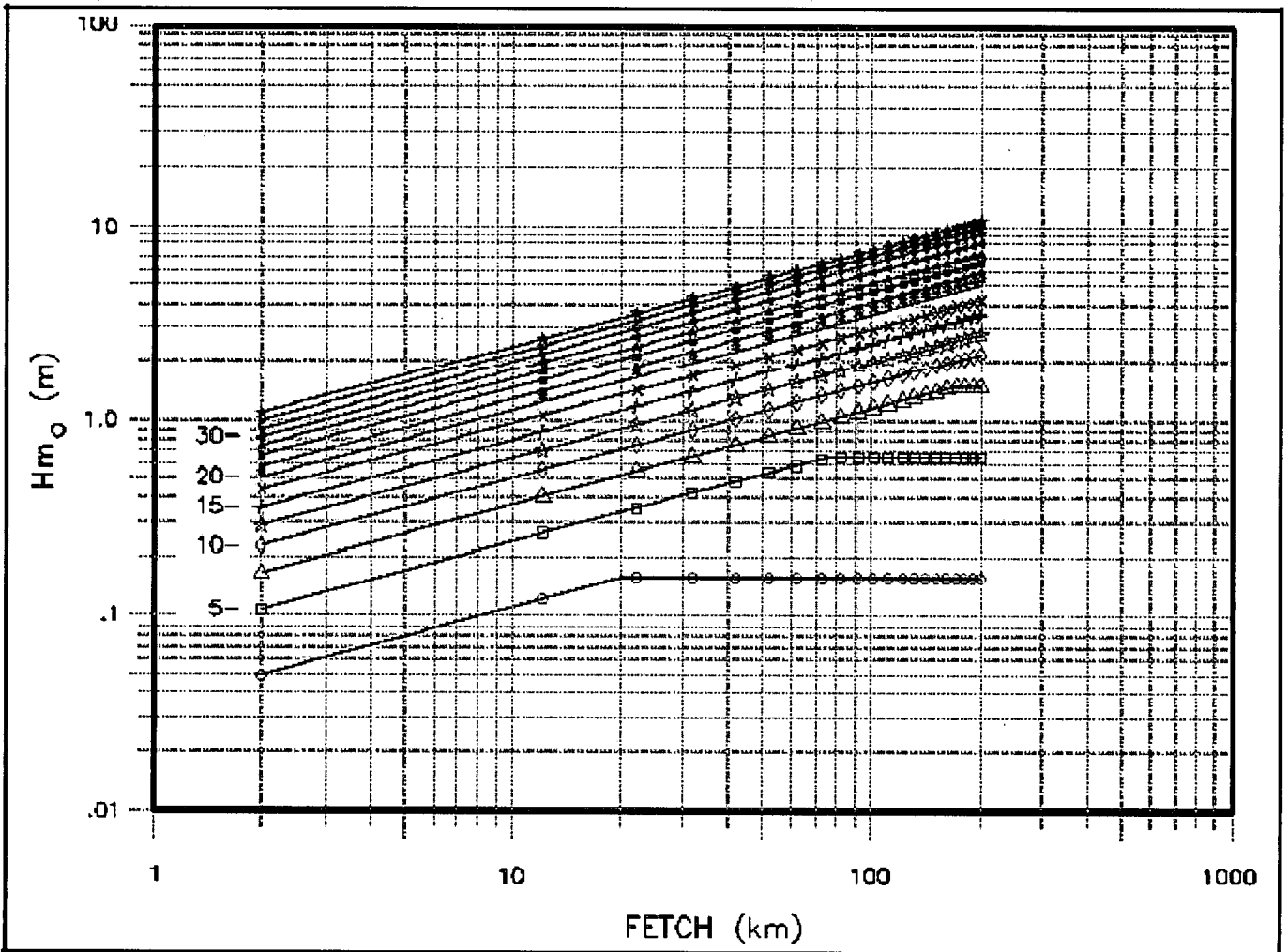
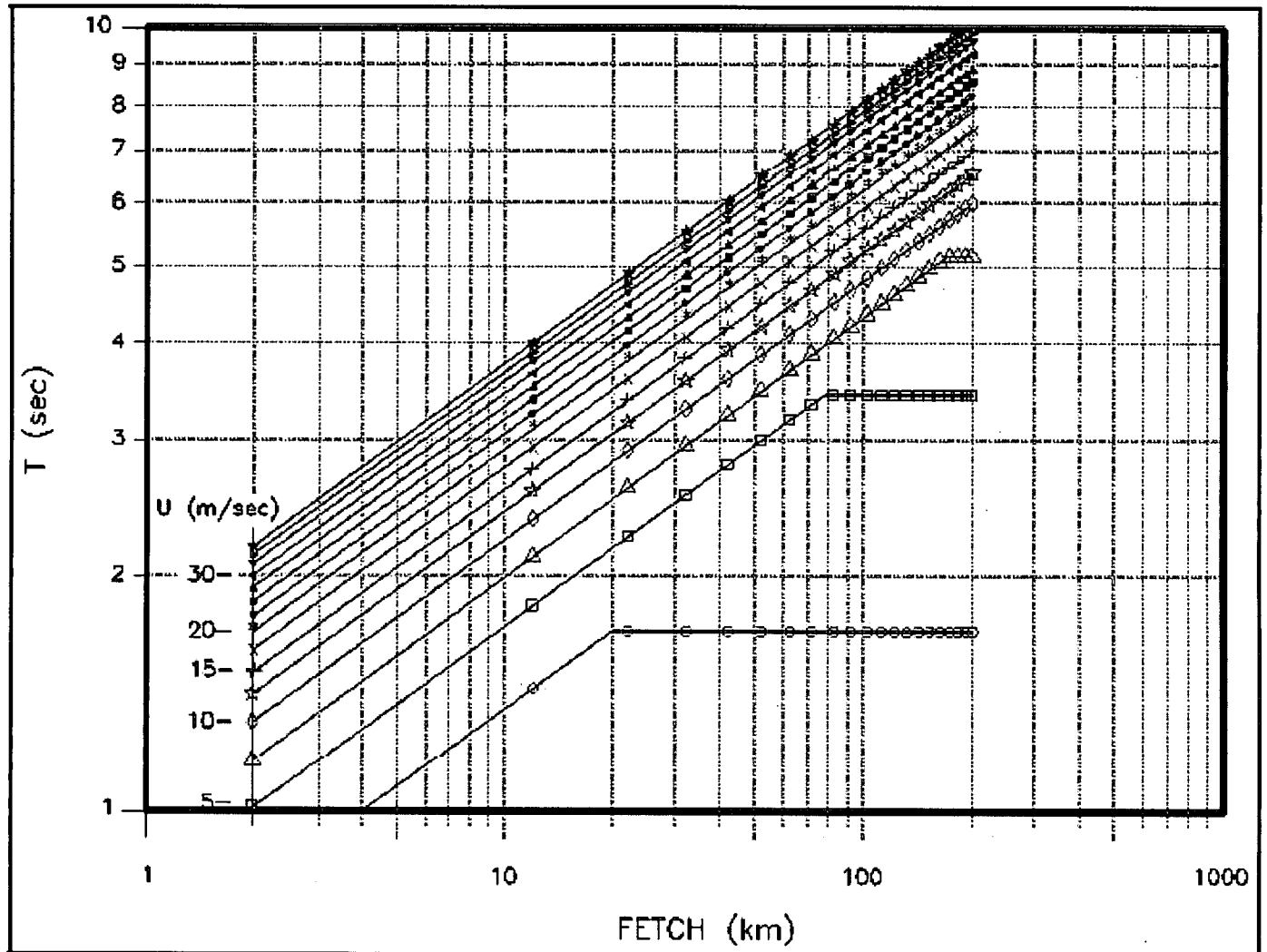


Figure 8. Fetch-Limited Wave Periods (Wind Speed in Increments of 2.5 m/s)



3. The peak wave periods for various fetch lines calculated above were compared with the shallow-water limit. According to *EM 1110-2-1100* (Part II), the shallow-water limit is given by the following equation:

$$T_p \approx 9.78 \left(\frac{d}{g} \right)^{\frac{1}{2}} \quad (1)$$

Where:

T_p = the limiting wave period in seconds,
 d = the water depth in meters, and
 g = the gravitational acceleration in meter/sec².

If the predicted peak wave period for a given fetch line is greater than the limiting value, then the predicted wave period is reduced to the limiting wave period. Conversely, if the

predicted wave period was less than the limiting value, the predicted deepwater wave period was retained and used for further calculations.

4. Wave runup was calculated using the wave runup equation on permeable slopes given in Chapter 5 of *EM 1110-2-1100* (Part VI). According to the Coastal Engineering Manual (CEM) (2006), the runup equation for various levels of percentage exceedances is given as:

$$R_{ui\%}/H_S = A\xi_{om} \text{ for } 1.0 < \xi_{om} \leq 1.5 \quad (2a)$$

$$R_{ui\%}/H_S = B\xi_{om}^C \text{ for } 1.5 < \xi_{om} \leq (D/B)^{1/C} \quad (2b)$$

$$R_{ui\%}/H_S = D \text{ for } (D/B)^{1/C} < \xi_{om} \leq 7.5 \quad (2c)$$

Where, ξ_{om} is the surf-similarity parameter for irregular waves defined as:

$$\xi_{om} = \frac{\tan \alpha}{\sqrt{S_{om}}} \quad (3)$$

In which α is an angle defined by arctangent of the slope of the structure (dam)/embankment or stream or reservoir bank, and S_{om} is the fictitious wave steepness defined as the ratio between the statistical wave height at the structure and representative deepwater wavelengths or as follows:

$$S_{om} = \frac{H_s}{L_{om}} = \frac{2\pi}{g} \frac{H_s}{T_m^2} \quad (4)$$

Where:

H_s = significant wave height of incident waves at the toe of the structure,

L_{om} = deepwater wavelength,

T_m = mean wave period, and

T_p = wave period corresponding to the peak of the wave spectrum.

The coefficients A through D in Equations 2a, 2b, and 2c for runup of irregular head-on waves on impermeable and permeable rock armored slopes are listed in **Table 8**.

Table 8. Coefficients in Equations (2a, 2b, and 2c) for Runup of Irregular Head-On Waves in Impermeable and Permeable Rock Armored Slopes

Percent	A	B	C	D
0.1	1.12	1.34	0.55	2.58
2	0.96	1.17	0.46	1.97
1	1.04	1.26	0.51	2.29
Significant	0.72	0.88	0.41	1.35

Using Steps 1 through 4 for various fetch lines, runup was calculated for various locations considering various scenarios for Options 1 and 2. **Tables 9 through 14** present the runup results.

Table 9. Runup Calculation for the HAR 2, HAR 3, HNP, Auxiliary Dam, and Main Dam Locations, Option 1, Scenario 1

Location	Line ID	Wind Velocity (m/s)	St. Line Fetch, X (km)	Hm0 (m)	Predicted Peak Wave Period, Tp (sec)	Slope = tan(alpha)	Deepwater Wave Steepness, s0	Iribarren Number	Significant Runup, Rus (ft)	Max Runup (0.1%), Ru (ft)
HAR 2	1	22.56	1.36	0.48	1.80	0.09	0.09	0.28	0.32	0.50
	2	22.63	1.16	0.44	1.71	0.09	0.10	0.28	0.29	0.45
	3	22.70	0.98	0.41	1.62	0.13	0.10	0.40	0.39	0.60
	4	22.68	1.02	0.42	1.64	0.13	0.10	0.40	0.39	0.61
	5	22.74	0.88	0.39	1.57	0.13	0.10	0.40	0.36	0.56
HAR 3	1	22.53	1.49	0.50	1.86	0.09	0.09	0.29	0.34	0.52
	2	22.55	1.41	0.49	1.83	0.09	0.09	0.28	0.33	0.51
	3	22.55	1.39	0.48	1.82	0.09	0.09	0.28	0.32	0.50
	4	22.79	0.80	0.33	1.41	0.13	0.11	0.38	0.30	0.47
	5	22.79	0.80	0.37	1.52	0.13	0.10	0.39	0.34	0.54
	6	22.79	0.80	0.37	1.52	0.13	0.10	0.39	0.34	0.54
	7	22.79	0.80	0.37	1.52	0.13	0.10	0.39	0.34	0.53
HNP	1	22.60	1.22	0.45	1.74	0.20	0.10	0.65	0.69	1.08
	2	21.82	6.92	1.03	3.06	0.10	0.07	0.38	0.92	1.43
	3	22.23	4.36	0.84	2.64	0.10	0.08	0.36	0.71	1.11
Dams	1	21.83	6.87	1.03	3.05	0.50	0.07	1.88	3.85	6.41
	2	21.83	6.87	1.03	3.05	0.40	0.07	1.50	3.52	5.67
	3	22.45	1.88	0.56	2.00	0.40	0.09	1.34	1.77	2.75
	4	22.48	1.73	0.54	1.95	0.40	0.09	1.33	1.69	2.62

Table 10. Runup Calculation for the HAR 2, HAR 3, HNP, Auxiliary Dam, and Main Dam Locations, Option 1, Scenario 2

Location	Line ID	Wind Velocity (m/s)	St. Line Fetch, X (km)	Hm0 (m)	Predicted Peak Wave Period, Tp (sec)	Slope = tan(alpha)	Deepwater Wave Steepness, s0	Iribarren Number	Significant Runup, Rus (ft)	Max Runup (0.1%), Ru (ft)
HAR 2	1	22.56	1.36	0.48	1.80	0.09	0.09	0.28	0.32	0.50
	2	22.63	1.16	0.44	1.71	0.09	0.10	0.28	0.29	0.45
	3	22.70	0.98	0.41	1.62	0.13	0.10	0.40	0.39	0.60
	4	22.68	1.02	0.42	1.64	0.13	0.10	0.40	0.39	0.61
	5	22.74	0.88	0.39	1.57	0.13	0.10	0.40	0.36	0.56
HAR 3	1	22.53	1.49	0.50	1.86	0.09	0.09	0.29	0.34	0.52
	2	22.55	1.41	0.49	1.83	0.09	0.09	0.28	0.33	0.51
	3	22.55	1.39	0.48	1.82	0.09	0.09	0.28	0.32	0.50
	4	22.79	0.80	0.33	1.41	0.13	0.11	0.38	0.30	0.47
	5	22.79	0.80	0.37	1.52	0.13	0.10	0.39	0.34	0.54
	6	22.79	0.80	0.37	1.52	0.13	0.10	0.39	0.34	0.54
	7	22.79	0.80	0.37	1.52	0.13	0.10	0.39	0.34	0.53
HNP	1	22.60	1.22	0.45	1.74	0.20	0.10	0.65	0.69	1.08
	2	21.82	6.92	1.03	3.06	0.10	0.07	0.38	0.92	1.43
	3	22.23	4.36	0.84	2.64	0.10	0.08	0.36	0.71	1.11
Dams	1	21.83	6.87	1.03	3.05	0.50	0.07	1.88	3.85	6.41
	2	21.83	6.87	1.03	3.05	0.40	0.07	1.50	3.52	5.67
	3	22.45	1.88	0.56	2.00	0.40	0.09	1.34	1.77	2.75
	4	22.48	1.73	0.54	1.95	0.40	0.09	1.33	1.69	2.62

Table 11. Runup Calculation for the HAR 2, HAR 3, HNP, Auxiliary Dam, and Main Dam Locations, Option 1, Scenario 3

Location	Line ID	Wind Velocity (m/s)	St. Line Fetch, X (km)	Hm0 (m)	Predicted Peak Wave Period, Tp (sec)	Slope = tan(alpha)	Deepwater Wave Steepness, s0	Iribarren Number	Significant Runup, Rus (ft)	Max Runup (0.1%), Ru (ft)
HAR 2	1	22.56	1.36	0.48	1.80	0.09	0.09	0.28	0.32	0.50
	2	22.63	1.16	0.44	1.71	0.09	0.10	0.28	0.29	0.45
	3	22.70	0.98	0.41	1.62	0.13	0.10	0.40	0.39	0.60
	4	22.68	1.02	0.42	1.64	0.13	0.10	0.40	0.39	0.61
	5	22.74	0.88	0.39	1.57	0.13	0.10	0.40	0.36	0.56
HAR 3	1	22.53	1.49	0.50	1.86	0.09	0.09	0.29	0.34	0.52
	2	22.55	1.41	0.49	1.83	0.09	0.09	0.28	0.33	0.51
	3	22.55	1.39	0.48	1.82	0.09	0.09	0.28	0.32	0.50
	4	22.79	0.80	0.33	1.41	0.13	0.11	0.38	0.30	0.47
	5	22.79	0.80	0.37	1.52	0.13	0.10	0.39	0.34	0.54
	6	22.79	0.80	0.37	1.52	0.13	0.10	0.39	0.34	0.54
	7	22.79	0.80	0.37	1.52	0.13	0.10	0.39	0.34	0.53
HNP	1	22.60	1.22	0.45	1.74	0.20	0.10	0.65	0.69	1.08
	2	21.82	6.92	1.03	3.06	0.10	0.07	0.38	0.92	1.43
	3	22.23	4.36	0.84	2.64	0.10	0.08	0.36	0.71	1.11
Dams	1	21.83	6.87	1.03	3.05	0.50	0.07	1.88	3.85	6.41
	2	21.83	6.87	1.03	3.05	0.40	0.07	1.50	3.52	5.67
	3	22.45	1.88	0.56	2.00	0.40	0.09	1.34	1.77	2.75
	4	22.48	1.73	0.54	1.95	0.40	0.09	1.33	1.69	2.62

Table 12. Runup Calculation for the HAR 2, HAR 3, HNP, Auxiliary Dam, and Main Dam Locations, Option 2, Scenario 1

Location	Line ID	Wind Velocity (m/s)	St. Line Fetch, X (km)	Hm0 (m)	Predicted Peak Wave Period, Tp (sec)	Slope = tan(alpha)	Deepwater Wave Steepness, s0	Iribarren Number	Significant Runup, Rus (ft)	Max Runup (0.1%), Ru (ft)
HAR 2	1	22.56	1.36	0.48	1.80	0.09	0.09	0.28	0.32	0.50
	2	22.63	1.16	0.44	1.71	0.09	0.10	0.28	0.29	0.45
	3	22.70	0.98	0.41	1.62	0.13	0.10	0.40	0.39	0.60
	4	22.68	1.02	0.42	1.64	0.13	0.10	0.40	0.39	0.61
	5	22.74	0.88	0.39	1.57	0.13	0.10	0.40	0.36	0.56
HAR 3	1	22.53	1.49	0.50	1.86	0.09	0.09	0.29	0.34	0.52
	2	22.55	1.41	0.49	1.83	0.09	0.09	0.28	0.33	0.51
	3	22.55	1.39	0.48	1.82	0.09	0.09	0.28	0.32	0.50
	4	22.91	0.63	0.33	1.41	0.13	0.11	0.38	0.30	0.47
	5	22.79	0.80	0.37	1.52	0.13	0.10	0.39	0.34	0.54
	6	22.79	0.80	0.37	1.52	0.13	0.10	0.39	0.34	0.54
	7	22.79	0.80	0.37	1.52	0.13	0.10	0.39	0.34	0.53
HNP	1	22.60	1.22	0.45	1.74	0.20	0.10	0.65	0.69	1.08
	2	21.82	6.92	1.03	3.06	0.10	0.07	0.38	0.92	1.43
	3	22.23	4.36	0.84	2.64	0.10	0.08	0.36	0.71	1.11
Dams	1	21.83	6.87	1.03	3.05	0.50	0.07	1.88	3.85	6.41
	2	21.83	6.87	1.03	3.05	0.40	0.07	1.50	3.52	5.67
	3	22.45	1.88	0.56	2.00	0.40	0.09	1.34	1.77	2.75
	4	22.48	1.73	0.54	1.95	0.40	0.09	1.33	1.69	2.62

Table 13. Runup Calculation for the HAR 2, HAR 3, HNP, Auxiliary Dam, and Main Dam Locations, Option 2, Scenario 2

Location	Line ID	Wind Velocity (m/s)	St. Line Fetch, X (km)	Hm0 (m)	Predicted Peak Wave Period, Tp (sec)	Slope = tan(alpha)	Deepwater Wave Steepness, s0	Iribarren Number	Significant Runup, Rus (ft)	Max Runup (0.1%), Ru (ft)
HAR 2	1	22.56	1.36	0.48	1.80	0.09	0.09	0.28	0.32	0.50
	2	22.63	1.16	0.44	1.71	0.09	0.10	0.28	0.29	0.45
	3	22.70	0.98	0.41	1.62	0.13	0.10	0.40	0.39	0.60
	4	22.68	1.02	0.42	1.64	0.13	0.10	0.40	0.39	0.61
	5	22.74	0.88	0.39	1.57	0.13	0.10	0.40	0.36	0.56
HAR 3	1	22.53	1.49	0.50	1.86	0.09	0.09	0.29	0.34	0.52
	2	22.55	1.41	0.49	1.83	0.09	0.09	0.28	0.33	0.51
	3	22.55	1.39	0.48	1.82	0.09	0.09	0.28	0.32	0.50
	4	22.91	0.63	0.33	1.41	0.13	0.11	0.38	0.30	0.47
	5	22.79	0.80	0.37	1.52	0.13	0.10	0.39	0.34	0.54
	6	22.79	0.80	0.37	1.52	0.13	0.10	0.39	0.34	0.54
	7	22.79	0.80	0.37	1.52	0.13	0.10	0.39	0.34	0.53
HNP	1	22.60	1.22	0.45	1.74	0.20	0.10	0.65	0.69	1.08
	2	21.82	6.92	1.03	3.06	0.10	0.07	0.38	0.92	1.43
	3	22.23	4.36	0.84	2.64	0.10	0.08	0.36	0.71	1.11
Dams	1	21.83	6.87	1.03	3.05	0.50	0.07	1.88	3.85	6.41
	2	21.83	6.87	1.03	3.05	0.40	0.07	1.50	3.52	5.67
	3	22.45	1.88	0.56	2.00	0.40	0.09	1.34	1.77	2.75
	4	22.48	1.73	0.54	1.95	0.40	0.09	1.33	1.69	2.62

Table 14. Runup Calculation for the HAR 2, HAR 3, HNP, Auxiliary Dam, and Main Dam Locations, Option 2, Scenario 3

Location	Line ID	Wind Velocity (m/s)	St. Line Fetch, X (km)	Hm0 (m)	Predicted Peak Wave Period, Tp (sec)	Slope = tan(alpha)	Deepwater Wave Steepness , s0	Iribarren Number	Significant Runup, Rus (ft)	Max Runup (0.1%), Ru (ft)
HAR 2	1	22.56	1.36	0.48	1.80	0.09	0.09	0.28	0.32	0.50
	2	22.63	1.16	0.44	1.71	0.09	0.10	0.28	0.29	0.45
	3	22.70	0.98	0.41	1.62	0.13	0.10	0.40	0.39	0.60
	4	22.68	1.02	0.42	1.64	0.13	0.10	0.40	0.39	0.61
	5	22.74	0.88	0.39	1.57	0.13	0.10	0.40	0.36	0.56
HAR 3	1	22.53	1.49	0.50	1.86	0.09	0.09	0.29	0.34	0.52
	2	22.55	1.41	0.49	1.83	0.09	0.09	0.28	0.33	0.51
	3	22.55	1.39	0.48	1.82	0.09	0.09	0.28	0.32	0.50
	4	22.91	0.63	0.33	1.41	0.13	0.11	0.38	0.30	0.47
	5	22.79	0.80	0.37	1.52	0.13	0.10	0.39	0.34	0.54
	6	22.79	0.80	0.37	1.52	0.13	0.10	0.39	0.34	0.54
	7	22.79	0.80	0.37	1.52	0.13	0.10	0.39	0.34	0.53
HNP	1	22.60	1.22	0.45	1.74	0.20	0.10	0.65	0.69	1.08
	2	21.82	6.92	1.03	3.06	0.10	0.07	0.38	0.92	1.43
	3	22.23	4.36	0.84	2.64	0.10	0.08	0.36	0.71	1.11
Dams	1	21.83	6.87	1.03	3.05	0.50	0.07	1.88	3.85	6.41
	2	21.83	6.87	1.03	3.05	0.40	0.07	1.50	3.52	5.67
	3	22.45	1.88	0.56	2.00	0.40	0.09	1.34	1.77	2.75
	4	22.48	1.73	0.54	1.95	0.40	0.09	1.33	1.69	2.62

Wind Setup

When wind blows over a water body, it exerts a horizontal stress on the water surface in the wind direction. In an enclosed water body, this results in a surplus of water at the leeward end and a decrease in water level at the windward end. This effect is called wind setup. According to *EM 1110-2-1420* (USACE, 1997), the wind setup in lakes and reservoirs can reasonably be estimated using the Zeider Zee equation, given as:

$$S = \frac{u^2 X}{1400d} \quad (5)$$

Where:

- S = the setup (ft) above the Stillwater level,
- u = the wind speed (mph),
- X = the fetch length (mile [mi]), and
- d = the water depth corresponding to the PMF level.

Tables 15 through 20 present the setup calculation using Equation 5 for the HAR 2, HAR 3, HNP, Auxiliary Dam, and Main Dam locations for various scenarios of Options 1 and 2.

Table 15. Setup Calculation for the HAR 2, HAR 3, HNP, Auxiliary Dam, and Main Dam Locations, Option 1, Scenario 1

Location	Line ID	Wind Velocity, u (mph)	St. Line Fetch, X (mi)	Depth, d (ft)	Setup, S (ft)
HAR 2	1	50.77	0.85	33.33	0.05
	2	50.91	0.72	33.33	0.04
	3	51.06	0.61	16.53	0.07
	4	51.03	0.64	16.53	0.07
	5	51.17	0.55	16.53	0.06
HAR 3	1	50.69	0.93	33.33	0.05
	2	50.74	0.88	33.33	0.05
	3	50.75	0.87	33.33	0.05
	4	51.27	0.40	16.53	0.05
	5	51.27	0.50	16.53	0.06
	6	51.27	0.50	16.53	0.06
	7	51.27	0.50	16.53	0.06
HNP	1	50.86	0.76	16.53	0.09
	2	49.10	4.33	33.33	0.22
	3	50.01	2.73	33.33	0.15
Dams	1	49.12	4.29	33.33	0.22
	2	49.12	4.29	33.33	0.22
	3	50.51	1.17	16.53	0.13
	4	50.57	1.08	16.53	0.12

Table 16. Setup Calculation for the HAR 2, HAR 3, HNP, Auxiliary Dam, and Main Dam Locations, Option 1, Scenario 2

Location	Line ID	Wind Velocity, u (mph)	St. Line Fetch, X (mi)	Depth, d (ft)	Setup, S (ft)
HAR 2	1	50.77	0.85	32.74	0.05
	2	50.91	0.72	32.74	0.04
	3	51.06	0.61	16.55	0.07
	4	51.03	0.64	16.55	0.07
	5	51.17	0.55	16.55	0.06
HAR 3	1	50.69	0.93	32.74	0.05
	2	50.74	0.88	32.74	0.05
	3	50.75	0.87	32.74	0.05
	4	51.27	0.40	16.55	0.04
	5	51.27	0.50	16.55	0.06
	6	51.27	0.50	16.55	0.06
	7	51.27	0.50	16.55	0.06
HNP	1	50.86	0.76	16.55	0.09
	2	49.10	4.33	32.74	0.23
	3	50.01	2.73	32.74	0.15
Dams	1	49.12	4.29	32.74	0.23
	2	49.12	4.29	32.74	0.23
	3	50.51	1.17	16.55	0.13
	4	50.57	1.08	16.55	0.12

Table 17. Setup Calculation for the HAR 2, HAR 3, HNP, Auxiliary Dam, and Main Dam Locations, Option 1, Scenario 3

Location	Line ID	Wind Velocity, u (mph)	St. Line Fetch, X (mi)	Depth, d (ft)	Setup, S (ft)
HAR 2	1	50.77	0.85	32.26	0.05
	2	50.91	0.72	32.26	0.04
	3	51.06	0.61	16.58	0.07
	4	51.03	0.64	16.58	0.07
	5	51.17	0.55	16.58	0.06
HAR 3	1	50.69	0.93	32.26	0.05
	2	50.74	0.88	32.26	0.05
	3	50.75	0.87	32.26	0.05
	4	51.27	0.40	16.58	0.04
	5	51.27	0.50	16.58	0.06
	6	51.27	0.50	16.58	0.06
	7	51.27	0.50	16.58	0.06
HNP	1	50.86	0.76	16.58	0.09
	2	49.10	4.33	32.26	0.23
	3	50.01	2.73	32.26	0.15
Dams	1	49.12	4.29	32.26	0.23
	2	49.12	4.29	32.26	0.23
	3	50.51	1.17	16.58	0.13
	4	50.57	1.08	16.58	0.12

Table 18. Setup Calculation for the HAR 2, HAR 3, HNP, Auxiliary Dam, and Main Dam Locations, Option 2, Scenario 1

Location	Line ID	Wind Velocity, u (mph)	St. Line Fetch, X (mi)	Depth, d (ft)	Setup, S (ft)
HAR 2	1	50.77	0.85	33.18	0.05
	2	50.91	0.72	33.18	0.04
	3	51.06	0.61	16.54	0.07
	4	51.03	0.64	16.54	0.07
	5	51.17	0.55	16.54	0.06
HAR 3	1	50.69	0.93	33.18	0.05
	2	50.74	0.88	33.18	0.05
	3	50.75	0.87	33.18	0.05
	4	51.54	0.40	16.54	0.05
	5	51.27	0.50	16.54	0.06
	6	51.27	0.50	16.54	0.06
	7	51.27	0.50	16.54	0.06
HNP	1	50.86	0.76	16.54	0.09
	2	49.10	4.33	33.18	0.22
	3	50.01	2.73	33.18	0.15
Dams	1	49.12	4.29	33.18	0.22
	2	49.12	4.29	33.18	0.22
	3	50.51	1.17	16.54	0.13
	4	50.57	1.08	16.54	0.12

Table 19. Setup Calculation for the HAR 2, HAR 3, HNP, Auxiliary Dam, and Main Dam Locations, Option 2, Scenario 2

Location	Line ID	Wind Velocity, u (mph)	St. Line Fetch, X (mi)	Depth, d (ft)	Setup, S (ft)
HAR 2	1	50.77	0.85	32.76	0.05
	2	50.91	0.72	32.76	0.04
	3	51.06	0.61	16.56	0.07
	4	51.03	0.64	16.56	0.07
	5	51.17	0.55	16.56	0.06
HAR 3	1	50.69	0.93	32.76	0.05
	2	50.74	0.88	32.76	0.05
	3	50.75	0.87	32.76	0.05
	4	51.54	0.40	16.56	0.05
	5	51.27	0.50	16.56	0.06
	6	51.27	0.50	16.56	0.06
	7	51.27	0.50	16.56	0.06
HNP	1	50.86	0.76	16.56	0.09
	2	49.10	4.33	32.76	0.23
	3	50.01	2.73	32.76	0.15
Dams	1	49.12	4.29	32.76	0.23
	2	49.12	4.29	32.76	0.23
	3	50.51	1.17	16.56	0.13
	4	50.57	1.08	16.56	0.12

Table 20. Setup Calculation for the HAR 2, HAR 3, HNP, Auxiliary Dam, and Main Dam Locations, Option 2, Scenario 3

Location	Line ID	Wind Velocity, u (mph)	St. Line Fetch, X (mi)	Depth, d (ft)	Setup, S (ft)
HAR 2	1	50.77	0.85	32.20	0.05
	2	50.91	0.72	32.20	0.04
	3	51.06	0.61	16.59	0.07
	4	51.03	0.64	16.59	0.07
	5	51.17	0.55	16.59	0.06
HAR 3	1	50.69	0.93	32.20	0.05
	2	50.74	0.88	32.20	0.05
	3	50.75	0.87	32.20	0.05
	4	51.54	0.40	16.59	0.05
	5	51.27	0.50	16.59	0.06
	6	51.27	0.50	16.59	0.06
	7	51.27	0.50	16.59	0.06
HNP	1	50.86	0.76	16.59	0.09
	2	49.10	4.33	32.20	0.23
	3	50.01	2.73	32.20	0.15
Dams	1	49.12	4.29	32.20	0.23
	2	49.12	4.29	32.20	0.23
	3	50.51	1.17	16.59	0.13
	4	50.57	1.08	16.59	0.12

Overall PMF Elevation

In order to determine the PMF elevation coincident with wind-wave activity at the HAR 2, HAR 3, HNP, Auxiliary Dam, and Main Dam locations, the obtained values of stillwater elevation, wave runup, and wind setup for various fetch lines were added together. **Tables 21 through 26** present the overall PMF elevations for various scenarios of Options 1 and 2.

Table 21. Overall PMF Elevation at the HAR 2, HAR 3, HNP, Auxiliary Dam, and Main Dam Locations, Option 1, Scenario 1

Location	Line ID	Stillwater EL. (ft. NGVD29)	Max. Runup (0.1%), Ru (ft.)	Setup (ft.)	Overall PMF (ft. NGVD29)
HAR 2	1	253.33	0.50	0.05	253.87
	2	253.33	0.45	0.04	253.82
	3	256.53	0.60	0.07	257.20
	4	256.53	0.61	0.07	257.21
	5	256.53	0.56	0.06	257.16
HAR 3	1	253.33	0.52	0.05	253.91
	2	253.33	0.51	0.05	253.89
	3	253.33	0.50	0.05	253.88
	4	256.53	0.47	0.05	257.04
	5	256.53	0.54	0.06	257.12
	6	256.53	0.54	0.06	257.12
	7	256.53	0.53	0.06	257.12
HNP	1	256.53	1.08	0.09	257.69
	2	253.33	1.43	0.22	254.98
	3	253.33	1.11	0.15	254.59
Dams	1	253.33	6.41	0.22	259.96
	2	253.33	5.67	0.22	259.22
	3	256.53	2.75	0.13	259.41
	4	256.53	2.62	0.12	259.27

Note: 253.33 ft and 256.53 ft are the maximum Stillwater PMF elevations for the Main and Auxiliary Reservoirs, respectively..

Table 22. Overall PMF Elevation at the HAR 2, HAR 3, HNP, Auxiliary Dam, and Main Dam Locations, Option 1, Scenario 2

Location	Line ID	Stillwater EL. (ft. NGVD29)	Max Runup (0.1%), Ru (ft.)	Setup (ft.)	Overall PMF (ft. NGVD29)
HAR 2	1	252.74	0.50	0.05	253.29
	2	252.74	0.45	0.04	253.24
	3	256.55	0.60	0.07	257.22
	4	256.55	0.61	0.07	257.24
	5	256.55	0.56	0.06	257.18
HAR 3	1	252.74	0.52	0.05	253.32
	2	252.74	0.51	0.05	253.30
	3	252.74	0.50	0.05	253.29
	4	256.55	0.47	0.04	257.07
	5	256.55	0.54	0.06	257.14
	6	256.55	0.54	0.06	257.15
	7	256.55	0.53	0.06	257.14
HNP	1	256.55	1.08	0.09	257.71
	2	252.74	1.43	0.23	254.40
	3	252.74	1.11	0.15	254.00
Dams	1	252.74	6.41	0.23	259.37
	2	252.74	5.67	0.23	258.63
	3	256.55	2.75	0.13	259.43
	4	256.55	2.62	0.12	259.29

Note: 252.74 ft and 256.55 ft are the maximum Stillwater PMF elevations for the Main and Auxiliary reservoirs, respectively..

Table 23. Overall PMF Elevation at the HAR 2, HAR 3, HNP, Auxiliary Dam, and Main Dam Locations, Option 1, Scenario 3

Location	Line ID	Stillwater EL. (ft. NGVD29)	Max Runup (0.1%), Ru (ft.)	Setup (ft.)	Overall PMF (ft. NGVD29)
HAR 2	1	252.26	0.50	0.05	252.81
	2	252.26	0.45	0.04	252.76
	3	256.58	0.60	0.07	257.24
	4	256.58	0.61	0.07	257.26
	5	256.58	0.56	0.06	257.20
HAR 3	1	252.26	0.52	0.05	252.84
	2	252.26	0.51	0.05	252.82
	3	252.26	0.50	0.05	252.81
	4	256.58	0.47	0.04	257.09
	5	256.58	0.54	0.06	257.17
	6	256.58	0.54	0.06	257.17
	7	256.58	0.53	0.06	257.17
HNP	1	256.58	1.08	0.09	257.74
	2	252.26	1.43	0.23	253.92
	3	252.26	1.11	0.15	253.52
Dams	1	252.26	6.41	0.23	258.90
	2	252.26	5.67	0.23	258.16
	3	256.58	2.75	0.13	259.45
	4	256.58	2.62	0.12	259.32

Note: 252.26 ft and 256.58 ft are the maximum Stillwater PMF elevations for the Main and Auxiliary reservoirs, respectively..

Table 24. Overall PMF Elevation at the HAR 2, HAR 3, HNP, Auxiliary Dam, and Main Dam Locations, Option 2, Scenario 1

Location	Line ID	Stillwater EL. (ft. NGVD29)	Max Runup (0.1%), Ru (ft.)	Setup (ft.)	Overall PMF (ft. NGVD29)
HAR 2	1	253.18	0.50	0.05	253.72
	2	253.18	0.45	0.04	253.68
	3	256.54	0.60	0.07	257.21
	4	256.54	0.61	0.07	257.22
	5	256.54	0.56	0.06	257.16
HAR 3	1	253.18	0.52	0.05	253.76
	2	253.18	0.51	0.05	253.74
	3	253.18	0.50	0.05	253.73
	4	256.54	0.47	0.05	257.05
	5	256.54	0.54	0.06	257.13
	6	256.54	0.54	0.06	257.13
	7	256.54	0.53	0.06	257.13
HNP	1	256.54	1.08	0.09	257.70
	2	253.18	1.43	0.22	254.83
	3	253.18	1.11	0.15	254.44
Dams	1	253.18	6.41	0.22	259.81
	2	253.18	5.67	0.22	259.07
	3	256.54	2.75	0.13	259.41
	4	256.54	2.62	0.12	259.28

Note: 253.18 ft and 256.54 ft are the maximum Stillwater PMF elevations for the Main and Auxiliary reservoirs, respectively..

Table 25. Overall PMF Elevation at the HAR 2, HAR 3, HNP, Auxiliary Dam, and Main Dam Locations, Option 2, Scenario 2

Location	Line ID	Stillwater EL. (ft. NGVD29)	Max Runup (0.1%), Ru (ft.)	Setup (ft.)	Overall PMF (ft. NGVD29)
HAR 2	1	252.76	0.50	0.05	253.31
	2	252.76	0.45	0.04	253.26
	3	256.56	0.60	0.07	257.23
	4	256.56	0.61	0.07	257.24
	5	256.56	0.56	0.06	257.19
HAR 3	1	252.76	0.52	0.05	253.34
	2	252.76	0.51	0.05	253.32
	3	252.76	0.50	0.05	253.31
	4	256.56	0.47	0.05	257.07
	5	256.56	0.54	0.06	257.15
	6	256.56	0.54	0.06	257.15
	7	256.56	0.53	0.06	257.15
HNP	1	256.56	1.08	0.09	257.72
	2	252.76	1.43	0.23	254.42
	3	252.76	1.11	0.15	254.02
Dams	1	252.76	6.41	0.23	259.39
	2	252.76	5.67	0.23	258.65
	3	256.56	2.75	0.13	259.44
	4	256.56	2.62	0.12	259.30

Note: 252.76 ft and 256.56 ft are the maximum Stillwater PMF elevations for the Main and Auxiliary reservoirs, respectively..

Table 26. Overall PMF Elevation at the HAR 2, HAR 3, HNP, Auxiliary Dam, and Main Dam Locations, Option 2, Scenario 3

Location	Line ID	Stillwater EL. (ft. NGVD29)	Max Runup (0.1%), Ru (ft.)	Setup (ft.)	Overall PMF (ft. NGVD29)
HAR 2	1	252.20	0.50	0.05	252.75
	2	252.20	0.45	0.04	252.70
	3	256.59	0.60	0.07	257.26
	4	256.59	0.61	0.07	257.27
	5	256.59	0.56	0.06	257.21
HAR 3	1	252.20	0.52	0.05	252.78
	2	252.20	0.51	0.05	252.76
	3	252.20	0.50	0.05	252.75
	4	256.59	0.47	0.05	257.10
	5	256.59	0.54	0.06	257.18
	6	256.59	0.54	0.06	257.18
	7	256.59	0.53	0.06	257.18
HNP	1	256.59	1.08	0.09	257.75
	2	252.20	1.43	0.23	253.86
	3	252.20	1.11	0.15	253.46
Dams	1	252.20	6.41	0.23	258.84
	2	252.20	5.67	0.23	258.10
	3	256.59	2.75	0.13	259.46
	4	256.59	2.62	0.12	259.33

Note: 252.20 ft and 256.59 ft are the maximum Stillwater PMF elevations for the Main and Auxiliary reservoirs, respectively.

Maximum PMF Elevation due to Coincident Wind-Wave Activity

To determine the critical PMF elevation coincident with wind-wave activity at the HAR 2, HAR 3, HNP, Auxiliary Dam, and Main Dam locations, the overall PMF elevations for various scenarios of Options 1 and 2 were calculated and are summarized in **Table 27**. Based on these summary results, all of the PMF elevations obtained in various scenarios for Options 1 and 2 are below 260 ft NGVD29. Therefore, any of the engineering solutions considered in the various scenarios for Options 1 and 2 can be used to ensure protection of safety-related structures against external flooding and dynamic effects of wave action due to wind-generated activity.

Table 27. Maximum PMF Elevation at the HAR 2, HAR 3, HNP, Auxiliary, and Main Dam

Location	Maximum PMF Elevation due to Coincident Wind-Wave Activity (feet NGVD29)					
	Option 1			Option 2		
	Scenario 1	Scenario 2	Scenario 3	Scenario 1	Scenario 2	Scenario 3
HAR 2	257.21	257.24	257.26	257.22	257.24	257.27
HAR 3	257.12	257.15	257.17	257.13	257.15	257.18
HNP	259.96	259.37	258.90	259.81	259.39	258.84
Dams	259.96	259.43	259.45	259.81	259.44	259.46

Associated HAR COL Application Revisions:

The following changes will be made to the HAR COLA in a future revision:

Revisions to the HAR ER include:

Second paragraph (p. 2-24) of HAR ER Rev. 1, Subsection 2.3.1.2.1.1, Dam and Appurtenances:

From:

“The Main Dam currently includes a concrete spillway with an ogee-shaped crest on the west abutment of the dam to pass floods as the only flow component. The spillway is uncontrolled and has a crest net length of 15.2 m (50 ft.) with a pier at mid-length. The crest of the current spillway is at an elevation of 67.1 m (220 ft.) NGVD29 and will be increased to a proposed elevation of 73.1 m (240 ft.) NGVD29. The proposed spillway will also have an uncontrolled, ogee-shaped crest with a net length of 15.2 m (50 ft.) and a pier at mid-length.”

To:

“The Main Dam currently includes a concrete spillway with an ogee-shaped crest on the west abutment of the dam to pass floods as the only flow component. The spillway is uncontrolled and has a crest net length of 15.2 m (50 ft.) with a pier at mid-length. The crest of the current spillway is at an elevation of 67.1 m (220 ft.) NGVD29. The proposed spillway design for the Main Dam will include raising the existing uncontrolled, ogee-shaped crest to 73.1 m (240 ft.) NGVD29 in one span and installing a Tainter gate in the second span with a spillway crest at 67.1 m (220 ft.) NGVD29.”

Second set of bullets (p. 4-3) in HAR ER Rev. 1, Section 4.0, Environmental Impacts of Construction:

From:

- “Modifications to the Main Dam at Harris Reservoir. The Main Dam currently includes a concrete service spillway with an ogee-shaped crest on the west abutment of the dam. The spillway is uncontrolled and has a crest net length of 15.2 m (50 ft.) with a pier at mid-length. The crest of the current spillway is at an

elevation of 67.1 m (220 ft.) NGVD29 and will be increased to a proposed elevation of 73.2 m (240 ft.) NGVD29. The proposed spillway will also have an uncontrolled, ogee-shaped crest with a net length of 15.2 m (50 ft.) and a pier at mid-length.”

To:

- “Modifications to the Main Dam at Harris Reservoir. The Main Dam currently includes a concrete service spillway with an ogee-shaped crest on the west abutment of the dam. The spillway is uncontrolled and has a crest net length of 15.2 m (50 ft.) with a pier at mid-length. The crest of the current spillway is at an elevation of 67.1 m (220 ft.) NGVD29. The proposed spillway design for the Main Dam will include raising the existing uncontrolled, ogee-shaped crest to 73.1 m (240 ft.) NGVD29 in one span and installing a Tainter gate in the second span with a spillway crest at 67.1 m (220 ft.) NGVD29.”

Second set of bullets (p. 5-3) in HAR ER Rev. 1, Section 5.0, Environmental Effects of Station Operation:

From:

- “Operation of the modified Main Dam at Harris Reservoir with a modified concrete service spillway with an ogee-shaped crest increased to an elevation of 73.2 m (240 ft.) NGVD29. The spillway will also have an uncontrolled, ogee-shaped crest with a net length of 15.2 m (50.0 ft.) and a pier at mid-length.”

To:

- “Operation of the modified Main Dam at Harris Reservoir with a modified concrete service spillway with an ogee-shaped crest increased to an elevation of 73.2 m (240 ft.) NGVD29 in one span and a Tainter gate in the second span with a spillway crest at 67.1 m (220 ft.) NGVD29. The spillway will also have a net length of 15.2 m (50.0 ft.) and a pier at mid-length.”

Revisions to the HAR FSAR include:

Fourteenth paragraph (p. 2.4-4) of HAR FSAR Rev. 1, Subsection 2.4.1.1, Site and Facilities:

From:

“The Main Dam currently includes a concrete spillway with an ogee-shaped crest on the west abutment to pass floods as the only flow component. The spillway is uncontrolled and has a crest net length of 15.2 m (50 ft.) with a pier at mid-length. The crest of the current spillway is at an elevation of 67.1 m (220 ft.) NGVD29 and will be raised to a proposed elevation of 73.2 m (240 ft.) NGVD29. The proposed spillway will also have an uncontrolled, ogee-shaped crest with a net length of 15.2 m (50 ft.) and a pier at mid-length.”

To:

“The Main Dam currently includes a concrete spillway with an ogee-shaped crest on the west abutment to pass floods as the only flow component. The spillway is uncontrolled and has a crest net length of 15.2 m (50 ft.) with a pier at mid-length. The crest of the current spillway is at an elevation of 67.1 m (220 ft.) NGVD29. The proposed spillway design for the Main Dam will include raising the existing uncontrolled, ogee-shaped crest to 73.1 m (240 ft.) NGVD29 in one span and installing a Tainter gate in the second span with a spillway crest at 67.1 m (220 ft.) NGVD29. The spillway will continue to have a net length of 15.2 m (50.0 ft.) and a pier at mid-length.”

The text and tables of HAR FSAR Rev. 1, Subsection 2.4.3 will be revised as presented below. Revisions to associated figures are as follows:

1. HAR FSAR Rev. 1, Figures 2.4.3-201 through 2.4.3-208 remain unchanged.
2. HAR FSAR Rev. 1, Figures 2.4.3-209 through 2.4.3-224 are revised.
3. Thirteen new figures (Figures 2.4.3-225 through 2.4.3-237) will be added to HAR FSAR Rev. 2, Subsection 2.4.3.

2.4.3 Probable Maximum Flood on Streams and Rivers

STD DEP 1.1-1

The PMF has been defined as an estimate of the hypothetical flood (peak discharge, volume, and hydrograph shape) that is considered to be the most severe and reasonably possible at a particular location, based on comprehensive hydrometeorological application of PMP and other hydrologic factors favorable for maximum flood runoff (Reference 2.4-222). The PMF represents an estimated upper bound on the maximum runoff potential for a given watershed. Thus, the objective of this study is to obtain a PMF hydrograph and estimation of the reservoir flood level to ensure the plant's safety.

Using the previous definition as a guide, the PMFs for the HAR site were developed using the following steps:

- a. The crest of the uncontrolled ogee spillway on the Main Dam is 67.1 m (220 ft.) NGVD29. It is proposed that this crest will be elevated to 73.2 m (240 ft.) NGVD29 in one span of the existing spillway and a Tainter gate will be installed in the other span at 67.1 m (220 ft.) NGVD29. Therefore, the first step for determining the PMF is to delineate the sub-basins of the Buckhorn Creek drainage basin above the Main Dam. Considering these proposed modifications in the spillway of the Main Dam, the drainage basin above the dam is 182.1 km² (70.3 mi.²) (Figure 2.4.3-201), wherein the area inundated is about 30.8 km² (11.9 mi.²), or about 16 percent of the entire basin. The Buckhorn Creek drainage basin above the Main Dam was divided into seven sub-basins (Sub-basin IV through Sub-basin X).
- b. The unit-hydrograph theory was used as the runoff model for developing runoff hydrographs for each sub-basin, except the Auxiliary Reservoir and Main Reservoir pool surfaces. Therefore, various parameters required for

developing unit hydrographs for the sub-basins were determined. Using these parameters, unit hydrographs were developed for each sub-basin. For the Auxiliary Reservoir and Main Reservoir pool surfaces, the direct rainfall was assumed to be equal to the runoff without any loss and lag.

- c. The PMP storm hyetograph was determined for the Buckhorn Creek drainage basin using criteria and step-by-step instructions given in HMR 51 (Reference 2.4-223) and HMR 52 (Reference 2.4-218). The developed PMP storm hyetograph was applied to the unit hydrographs with the appropriate infiltration losses to develop the estimated flood hydrographs for each sub-basin, as well as for the entire drainage basin.
- d. Based on the requirements of American National Standards Institute/American Nuclear Society (ANSI/ANS)-2.8-1992, Section 9.2.1.1, an antecedent 72-hour storm having a volume of 40 percent of the PMP was assumed (Reference 2.4-222). This antecedent storm was assumed to be followed by 72 hours of no rain. Then the full 72-hour PMP storm followed. This was the complete PMP storm that was applied to the unit hydrographs with appropriate infiltration losses to develop the estimated flood hydrograph for each sub-basin. These flood hydrographs were used to estimate the PMF stillwater level in the Main Reservoir and in the Auxiliary Reservoir.
- e. Inflow hydrographs from various sub-basins upstream of the Main Dam were added together without conducting reach routing using the Hydrologic Engineering Center-Hydrologic Modeling System (HEC-HMS) model to determine the combined inflow to the Main Dam (Reference 2.4-224).
- f. After obtaining the combined inflow hydrograph, the PMF hydrograph was routed through the reservoir, spillway, and outlet works using the level pool reservoir routing method to estimate the maximum PMF stillwater level in the reservoirs.

The following discussions are based on the guidance presented in Regulatory Guide 1.206, Revision 0.

2.4.3.1 Probable Maximum Precipitation

The PMP is theoretically the greatest depth of precipitation for a given duration that is physically possible over a given size storm area at a particular geographical location at a certain time of the year (Reference 2.4-225). In other words, the PMP is the estimated depth of precipitation for which there is virtually no risk of exceedance (Reference 2.4-222). The PMP depths used in this study were calculated using the criteria and step-by-step instructions given in HMR 51 (Reference 2.4-223) and HMR 52 (Reference 2.4-218).

Generally, a three-step process is followed for determining PMP in nonorographic regions: moisture maximization, transposition, and envelopment:

- a. Moisture maximization consists of increasing storm precipitation measured in a major historical event by a factor that reflects the maximum amount of moisture that could have existed in the atmosphere for the storm location and time of year.

- b. Transposition refers to the process of moving a storm (that is, isohyetal pattern) from the location where it occurred to another location of interest. Transposition is carried out only within a region that is homogeneous with respect to terrain and meteorology.
- c. Envelopment involves construction of smooth curves that envelope precipitation maxima for various durations and area sizes to compensate for data gaps. In addition, geographic smoothing is performed to ensure regional consistency.

Using these principles, estimates of all-season PMPs for various-sized areas and storm durations are available in the form of generalized plots on Figures 18 through 47 in HMR 51 (Reference 2.4-223).

The drainage area for the Buckhorn Creek watershed above the Main Dam (Figure 2.4.3-201) is 182.1 km² (70.3 mi.²), and the location of the centroid of the basin is approximately 35°38'00" N, 78°57'22" W. Using HMR 52 as a guide, the PMP for the Buckhorn Creek drainage basin was developed using the following steps (Reference 2.4-218):

- a. Determination of 6-hour Incremental PMP.
- b. Determination of 6-hour Incremental PMP Isohyetal Pattern.
- c. Maximization of Precipitation Volume.
- d. Distribution of Storm-Area Averaged PMP over the Drainage Basin.
- e. Development of Design Storm for Basin above the Main Dam.
- f. Development of Design Storm for Drainage Basin above the Auxiliary Dam.

2.4.3.1.1 Determination of 6-Hour Incremental PMP

The generalized estimates of all-season PMP depths available from Figures 18 through 47 of HMR 51 (Reference 2.4-223) were obtained for various-sized areas, both larger and smaller than the drainage area under study for the Buckhorn Creek watershed. Table 2.4.3-201 provides the 6-hour incremental depth-area-duration data taken from Figures 18 through 47 of HMR 51 (Reference 2.4-223). Using the data presented in Table 2.4.3-201, the smooth depth-area-duration curves for the Buckhorn Creek drainage basin above the Main Dam were plotted on Figure 2.4.3-202.

This initial plotting of the basic input data serves the following two functions:

- It eliminates reader errors from basic misinterpretation of values in the figures in HMR 51 (Reference 2.4-223).
- It applies initial important smoothing of the basic precipitation data.

From the smooth curves of Figure 2.4.3-202, the PMP depths for various durations were read as tabulated in Table 2.4.3-202. Using the depth-area-duration graph of Figure 2.4.3-202, depth-area-duration values for a set of standard isohyet area sizes, both larger and smaller than the size of the drainage area under study, were read. The selected standard isohyet area sizes for the current study are 10, 25, 50,

100, 175, 300, and 450 mi.². Table 2.4.3-203 tabulates the depth-area-duration values for the selected standard isohyet areas.

The depth-area-duration data for the selected standard areas from Table 2.4.3-203 were plotted on a linear paper, and smooth curves were fitted as shown on Figure 2.4.3-203. From Figure 2.4.3-203, the PMP values corresponding to an 18-hour duration were read as tabulated in Table 2.4.3-204. Incremental differences for the first three 6-hour periods were obtained by successive subtraction of the values contained in Tables 2.4.3-203 and 2.4.3-204. Table 2.4.3-205 shows the incremental PMP values obtained for the first three periods. Each set of 6-hour values was plotted against the corresponding area values, and smooth lines were fitted through these points, as shown on Figure 2.4.3-204. Using the smooth curves from Figure 2.4.3-204, the data in Table 2.4.3-206 were tabulated for the 6-hour incremental PMP differences.

2.4.3.1.2 Determination of 6-Hour Incremental PMP Isohyetal Pattern

There is a preferred orientation for storms at a particular geographic location. That orientation is related to the general movement of storm systems and the direction of moisture-bearing winds. Based on contours of preferred orientation shown on Figure 8 of HMR 52, the preferred orientation for storms at the location having its latitude 35°38'00" N and longitude 78°57'22" W was 200 degrees (Reference 2.4-218). The orientation of the storm pattern to produce maximum precipitation volume in the watershed was found to be approximately 215 degrees. The angular difference in the orientations is 15 degrees, which is less than 40 degrees. This indicates that no adjustment is required in the incremental storm pattern given in Table 2.4.3-206.

2.4.3.1.3 Maximization of Precipitation Volume

The maximum precipitation volume for the three largest 6-hour incremental periods resulting from placement of the storm pattern given in Table 2.4.3-206 over the Buckhorn Creek drainage basin above the Main Dam was determined. To do this, it was necessary to obtain the value to be assigned to each isohyet in the pattern that occurs over the drainage basin during each period. Tables 2.4.3-207, 2.4.3-208, and 2.4.3-209 present the computations based on the HMR 52 procedure (Reference 2.4-218) for the first, second, and third increments, respectively.

Based on the calculations presented in Tables 2.4.3-207, 2.4.3-208, and 2.4.3-209, the pattern area size that maximizes the volume of precipitation for the three largest 6-hour incremental periods was found to be 259 km² (100 mi.²).

2.4.3.1.4 Distribution of Storm-Area Averaged PMP over the Drainage Basin

It was concluded that the maximum volume occurs for a PMP pattern near 259 km² (100 mi.²) when placed over the Buckhorn Creek watershed. With this information, the values for each isohyet for all 12 six-hour increments can be determined. Table 2.4.3-210 provides the incremental average depths for each 6-hour period of the 72-hour storm. With this information, the isohyet values were obtained for all 12 increments (Table 2.4.3-211).

The values in Table 2.4.3-211 represent the incremental isohyet values for the Buckhorn Creek watershed with a 259 km² (100 mi.²) PMP pattern. To obtain incremental average depths for this drainage, it was necessary to compute the incremental volumes as determined in Tables 2.4.3-207, 2.4.3-208, and 2.4.3-209 and then divide each incremental volume by the drainage area. The computations were performed in the tabular format as shown in Tables 2.4.3-212 and 2.4.3-213.

Based on the previous calculations, Table 2.4.3-214 provides the 72-hour total drainage-averaged PMP. After obtaining the drainage-averaged PMP storm depths, they were distributed according to ANSI/ANS-2.8-1992 guidelines, as provided in Table 2.4.3-215 (Reference 2.4-222). Total rainfall for the 72-hour duration was found to be 99.7 cm (39.24 in.). The resulting hourly PMP rainfall distribution is tabulated in Table 2.4.3-216 and plotted on Figure 2.4.3-205.

2.4.3.1.5 Development of Design Storm for Basin above the Main Dam

Using the PMP rainfall distribution shown on Figure 2.4.3-205, a design storm was developed. The design storm was developed by accounting for the antecedent rainfall that precedes the PMP storm based on ANSI/ANS-2.8-1992 guidelines (Reference 2.4-222). This design storm, which was used as the rainfall input in the hydrologic modeling, has the following components:

- An antecedent 72-hour storm that comprises 40 percent of the PMP volume.
- A 72-hour dry period following the antecedent 72-hour storm.
- The full 72-hour PMP following the 72-hour no-rain period.

Combining the above three components, Figure 2.4.3-206 shows the resulting design storm rainfall data that were developed for the basin above the Main Dam.

2.4.3.1.6 Development of Design Storm for Drainage Basin above the Auxiliary Dam

The total drainage area of the Auxiliary Reservoir watershed is 7.8 km² (3 mi.²). The smallest area considered in HMR 52 is 26.0 km² (10 mi.²), with a 72-hour PMP of about 119.6 cm (47.10 in.) (Reference 2.4-218). Extrapolating depth-area-duration curves of Figure 2.4.3-202 for a drainage area of 7.8 km² (3 mi.²), the 72-hour PMP for the drainage basin above the Auxiliary Dam was found to be 126.62 cm (49.85 in.). Using the temporal distribution of the design storm above the Main Dam (Figure 2.4.3-206), the design storm for the drainage basin above the Auxiliary Dam was determined. The resulting hourly PMP rainfall distribution for the Auxiliary Dam is presented in Table 2.4.3-216 and plotted on Figure 2.4.3-207. Combining the three components described in Subsection 2.4.3.1.5, Figure 2.4.3-208 depicts the resulting design storm rainfall input that was developed.

2.4.3.2 Precipitation Losses

This subsection describes the methodology used to assign precipitation loss rates in the PMF hydrologic model. The amount of rainfall loss (the portion that does not contribute to runoff) is a function of the type of soil, the ground cover (vegetated, bare, or paved), and the soil moisture prior to the storm. The amount of rainfall loss

can be characterized by various methods; the loss methods and their parameters are selected in accordance with recognizable characteristics of the drainage basin under study. The HEC-HMS model offers several methods for estimating precipitation losses. However, the exponential loss rate method that was used in the HNP FSAR is not included among the available loss methods. Thus, the optimized loss parameters used in the HNP FSAR could not be used in the present study.

The traditional initial and constant loss rate method for PMF computations was selected from the HEC-HMS model precipitation loss methods based on Federal Energy Regulatory Commission (FERC) recommendations (Reference 2.4-226). The following assumptions were made:

- Saturated antecedent conditions existed in the entire watershed prior to the start of the PMP.
- The initial loss for the sub-basins was zero inches (conservative assumption).
- Infiltration occurs at the minimum rate (for consistency with saturated soil conditions).

To determine the minimum infiltration rate, the average soil type for each sub-basin was determined. The land use in the study basin is primarily forested game lands throughout the watershed, with some transitional and urban areas well beyond the major watershed. FSAR Subsection 2.4.12.1.2.1 provides a detailed description of soil types in the study basin. Table 2.4.12-201 briefly summarizes the soil types in the Buckhorn Creek watershed. The study basin contains primarily three soil types: Creedmoor, Mayodan, and White Store. The U.S. Department of Agriculture (USDA) soil texture can be described approximately as sandy clay loam that falls into hydraulic soil group "C." The HAR site is classified as heavy industrial, and the remaining area of the Buckhorn Creek watershed can be classified as approximately 85 percent forest and 15 percent transitional lands.

The USDA soil texture at the HAR site can be described as approximately sandy clay loam that falls into hydraulic soil group "C." Based on TR-55 (References 2.4-227 and 2.4-228), the range of infiltration rates for the hydrologic soil group "C" is 0.05 – 0.15 in/hr. To ensure that the PMF estimate is both conservative and representative of the site, the PMF analysis is performed by taking credit for an initial infiltration loss of 0.15 in/hr, which then decreases linearly to zero at the end of 72 hours of the antecedent storm. During the hours of 72 through 144 when there is no rainfall but soil is still saturated and depressions are full, the infiltration loss rate is assumed to be zero. During the full PMP event, which includes the hours of 72 and after, the infiltration loss rate is assumed to be zero.

Given these conditions, the maximum potential loss rate due to infiltration, A_{loss} , is described by the following formula:

$$A_{loss} = 0.15 \left(1 - \frac{t}{72} \right) \text{ for } 0 \leq t \leq 72$$

$$A_{loss} = 0.0 \text{ for } t > 72$$

where

A_{loss} = infiltration loss rate (in/hr), and

t = time (hr).

The above equation gives the maximum potential infiltration loss rate. Further, the actual rate of loss due to infiltration during the 72-hour antecedent storm is then calculated as the minimum of (1) the maximum potential infiltration rate given by the above formula, and (2) the rate of rainfall during a given hour of the 72-hour antecedent storm:

$$Loss_{actual} = \min[A_{loss}(t), \text{rainfall}(t)]$$

Figure 2.4.3-209 depicts the actual infiltration rate during the 72-hour antecedent storm and the two succeeding 72-hour periods of the PMP event for both the design storms associated with the Main Reservoir and the Auxiliary Reservoir drainage basins. It can be noted from Figure 2.4.3-209 that the actual infiltration rate is constrained only by precipitation levels for the first 24 hours of the antecedent storm. However, from hours 24 through 72, the infiltration rate is constrained by the potential maximum rate, as previously described. The infiltration rate is assumed to be zero after hour 72.

The infiltration rate represents a precipitation loss. The traditional initial and constant loss rate method was used as the precipitation loss method while using the HEC-HMS model for conducting the PMF computations. However, it is difficult to incorporate the infiltration loss given by the above equations in a single continuous run of the HEC-HMS model. Therefore, the effective rainfall has been calculated outside the HEC-HMS model. Effective precipitation values are determined by subtracting infiltration from actual precipitation as follows:

$$\text{Effective Precipitation} = \text{Actual Precipitation} - \text{Infiltration Loss}$$

Actual and effective precipitations, calculated using the above procedure, for both the Main and Auxiliary drainage basins are shown on Figures 2.4.3-210 and 2.4.3-211. Note that actual and effective precipitations are equal when the infiltration rate is zero, which is the case for hour 72 and after. Therefore, only the 72-hour antecedent storm is shown on Figures 2.4.3-210 and 2.4.3-211. Table 2.4.3-217 shows a tabular list of the effective rainfall values for both the Main and Auxiliary Reservoirs. Total losses due to infiltration as a percentage of precipitation were 22.6 percent (3.55 inches) and 19.5 percent (3.88 inches) for the Main and Auxiliary Reservoirs, respectively.

This procedure was not used for the pool areas of the Main and Auxiliary Reservoirs where 100 percent of the rainfall was converted into direct runoff.

As previously described, the infiltration loss rate was applied outside the HEC-HMS model. The input infiltration loss rate parameters for various sub-basins were assumed to be zero in the HEC-HMS model, as shown in Table 2.4.3-218. In addition, Table 2.4.3-218 lists loss parameters for various sub-basins of the Buckhorn Creek watershed above the Main Dam.

2.4.3.3 Runoff and Stream Course Models

A runoff model is used to transform excess precipitation into surface runoff. For the purpose of this analysis, runoff was modeled using two different methods: one for rain falling on land surfaces, and a second for rain falling directly on reservoir pool surfaces. The runoff modeling approach is generally described as follows:

Land Surface Areas - Unit hydrographs were applied to transform excess rainfall over land surface areas into runoff.

- Reservoir Pool Surface Areas - Precipitation falling directly over reservoir pool areas was converted into runoff without considering any infiltration loss or lag time.
- No reach routing was used; traveling time of runoff from land areas into the reservoir was neglected.
- Level pool routing was used to determine the PMF elevations in both the Main and Auxiliary Reservoirs.

2.4.3.3.1 Runoff Model

An overland runoff model is generally represented in the form of a unit hydrograph. A unit hydrograph is defined as the direct runoff hydrograph produced by one unit (inch) of effective rain uniformly distributed over a sub-basin. Unit hydrographs are combined with precipitation data to determine the direct runoff hydrograph for a given storm event in a particular basin. Thus, separate unit hydrographs are developed for each sub-basin using their specific hydrologic parameters.

Several different methods can be used to develop a unit hydrograph for a given sub-basin. Selection of an appropriate method depends on knowledge of its hydrologic response characteristics. Based on the hydrologic characteristics of the Buckhorn Creek drainage basin, the Snyder hydrograph method was selected as acceptable. The required hydrologic parameters for developing the Snyder's synthetic unit hydrographs were readily available. The HNP FSAR calculated the required generalized values of the shape coefficients that are empirical in nature. The other parameters of the Snyder's method can be determined from the geometry of each sub-basin.

The following information summarizes the Snyder's synthetic hydrograph method. The Snyder unit hydrograph relationships define only the unit hydrograph peak discharge (Q_P) and the lag time (t_L) that are defined as (Reference 2.4-226):

$$t_L = CC_t(LLC)^{0.3} \quad (1)$$

$$Q_P = \frac{640C_P A}{t_L} \quad (2)$$

where

L = flow path length from outlet to the hydraulically farthest point (basin divide),

L_C = flow path length from outlet to sub-basin centroid,

A = drainage area in square miles,

C = unit conversion factor (equal to 1.0 when English units are used),

C_t = Snyder basin lag coefficient, and

C_p = Snyder peaking coefficient.

The parameters C_t and C_p are strictly empirical values often recommended as applicable to a specific region. C_t accounts for storage and shape of the watershed, and C_p is a function of flood-wave velocity and storage. The generalized values of C_t and C_p as given in the HNP FSAR are 3.91 and 0.75, respectively.

To apply the unit hydrograph approach to the Buckhorn Creek drainage basin, unit hydrographs were developed for three surfaces: (1) Main Reservoir pool surface, (2) Auxiliary Reservoir pool surface, and (3) Residual Land Surface around the Main Reservoir and the seven sub-basins in the Buckhorn Creek drainage basin above the Main Dam. Figure 2.4.3-201 shows Buckhorn Creek drainage sub-basin areas above the Main Dam. This figure illustrates that Sub-basins I, II, and III fall below the Main Dam spillway. Therefore, these sub-basins were not considered in the drainage area at the Main Dam. Excluding these sub-basins, the total drainage area at the Main Dam is 182.1 km² (70.3 mi.²). This area also includes the drainage area at the Auxiliary Reservoir. Table 2.4.3-219 lists the drainage areas of the Auxiliary Reservoir Surface, Main Reservoir Surface, Residual Land Surface, and Sub-basins IV, V, VI, VII, VIII, IX, and X.

A unit hydrograph has meaning only in connection with a specific duration of runoff. A sub-basin may have many different unit hydrographs, each associated with a different duration of runoff. Haan et al. recommend that the duration D of a unit hydrograph should be between $T_p/5$ and $T_p/3$, where T_p is the time to peak (Reference 2.4-229). Further, T_p is a function of D and catchment lag time T_L , defined as $T_p = T_L + D/2$ (Reference 2.4-259). However, for the Snyder's synthetic unit hydrograph, $D = T_L/5.5$. The catchment lag is a parameter used in unit hydrograph theory to provide a global measure of the response time of a catchment area. Since this global parameter incorporates various basin characteristics, such as hydraulic length, gradient, drainage density, and drainage patterns to determine these characteristics, it is necessary to delineate the sub-basins according to their drainage pattern as shown on Figure 2.4.3-201. Table 2.4.3-220 lists various watershed parameters, along with the Snyder Hydrograph parameters used in the HEC-HMS model.

More conservative alternate parameters were used for the residual area. A lag time of 10.6 hours was obtained by substituting the geometric characteristics associated with the land area surrounding the Main Reservoir in the Snyder's unit hydrograph

equations. To increase conservatism, the calculated lag time was reduced from 10.6 hours to 1.7 hours by assuming a coefficient of $L = 0.4$ and of $L_C = 0.15$ in Equation (1). By decreasing the lag time, the peak flow increases from 796 cfs to 4,992 cfs within the residual area.

Using the standard Snyder hydrograph parameters and the more conservative lag time and peak flow parameters for the residual area presented in Table 2.4.3-220 as input in HEC-HMS model, 1-hour unit hydrographs were developed, as shown on Figure 2.4.3-212. The parameters associated with the 1-hour hydrographs for each basin are provided in Table 2.4.3-221.

2.4.3.3.2 Hydrograph Peaking

In order to ensure safety of the HAR site against flooding, the degree of conservatism associated with the peak flow calculation was determined. For this purpose, several storm events smaller than the PMP storm, but sufficient to cause out-of-bank flooding, were used. For these storm events, both rainfall and expected runoff are known. Using the hydrographs of these storm events, along with unit hydrograph parameters used in the PMF inflow calculation as presented in Table 2.4.3-221, peak flows were determined using the developed HEC-HMS model used for the PMF analysis. The obtained peak flows were compared with peak flows determined using the peak flow equations developed by the USGS for rural basins in North Carolina (Reference 2.4-260). The predictive error associated with these equations is known. In order to produce the most conservative estimate, the peak flows generated by these equations have been corrected by adding the known predictive errors (that is, erring in the positive direction). The resulting peak flow values were then compared with the results generated by the HEC-HMS model for various storm events (Table 2.4.3-222) without making any change in the HEC-HMS parameters used for the PMP storm event.

As shown in Table 2.4.3-222, the estimated magnitude of peak flow events generated by the HEC-HMS model exceeds the corrected peak flows predicted using the USGS equations by more than 30 percent in all cases. This comparison serves to emphasize the degree to which the HEC-HMS computed peak flows are conservative. In other words, the difference between the magnitude of peak flows generated by the HEC-HMS model and peak flows obtained using USGS flow equations (Reference 2.4-260) can be considered as the implicit peaking factors.

In order to comply with the recommendation of ER 1110-8-2(FR) (Reference 2.4-231), the 1-hour base unit hydrographs that were developed for the FSAR analysis using the Snyder method were peaked. That is, the unit hydrographs (Figure 2.4.3-212) were adjusted such that the peak flows were increased by 25 percent, while the unit volume of each unit hydrograph was maintained. Given these adjustments, the appropriate time base and lag times of the peaked unit hydrographs were determined. The revised parameters associated with the peaked unit hydrographs are listed in Table 2.4.3-223. (Refer to Table 2.4.3-221 for the 1-hour base unit hydrograph parameters.)

A comparison of the base unit hydrograph to the peaked unit hydrograph for Sub-basin X is shown on Figure 2.4.3-213, while the peaked unit hydrographs for all sub-basins are shown on Figure 2.4.3-214.

Using the PMP storms above the Main Dam (Figure 2.4.3-206) and Auxiliary Dam (Figure 2.4.3-208), along with the developed unit hydrographs for various sub-basins for both cases considering peaking and no peaking, inflow hydrographs to the Auxiliary and Main Reservoirs were determined using the HEC-HMS model. Figures 2.4.3-215 and 2.4.3-216 show inflow hydrographs to the Auxiliary and Main Reservoirs for both cases with and without considering hydrograph peaking. Table 2.4.3-224 provides a summary of inflow results for both the Auxiliary and Main Reservoirs considering 25 percent peaking and no peaking for the PMP event.

2.4.3.3.3 Basin Data

Basin data include the elements of the basin, their connectivity, runoff, storage, and discharge relationships of hydraulic structures, and routing parameters of stream reaches and reservoirs. Figure 2.4.3-217 presents a schematic of the Buckhorn Creek drainage basin above the Main Dam and its elements, along with their connectivity.

Figures 2.4.1-206 and 2.4.1-207 present the stage-storage-area curves for the Main Reservoir and the Auxiliary Reservoir, respectively. Survey data from the Topographic Maps and Digital Ortho Photos from Barton Aerial Technologies (BAT) were used to develop the stage-storage-area curves for the Main Reservoir. The HNP FSAR provided the stage-storage-area curves for the Auxiliary Reservoir.

Both the Main Dam and the Auxiliary Dam have uncontrolled ogee spillways. The crest of the Main Dam spillway is at elevation 67.1 m (220 ft.) NGVD29, and the crest of the Auxiliary Dam spillway is at elevation 76.8 m (252 ft.) NGVD29. The elevation of the top of both dams is 79.2 m (260 ft.) NGVD29. The spillway crest at the Main Dam has a net length of 15.2 m (50 ft.) with a pier at its mid-length, while the spillway crest at the Auxiliary Dam has a length of 51.8 m (170 ft.). Both spillways are ogee-shaped and designed with a design head (H_0) and the upstream dam height (P) of 3.0 m (10 ft.) and 9.1 m (30 ft.), respectively, for the Main Dam spillway, while the corresponding values for the Auxiliary Dam spillway are 1.5 m (5 ft.) and 2.1 m (7 ft.), respectively.

It is proposed that the normal water level (NWL) of the Main Reservoir be raised to an elevation of 73.2 m (240 ft.) NGVD29 from the existing NWL elevation of 67.1 m (220 ft.) NGVD29. In order to ensure protection of safety-related structures against external flooding and dynamic effects of wave action due to wind-generated activity, several engineering solutions were considered to modify the spillway design of the Main Dam so that the maximum water level due to coincidental effects of wind-generated setup and wave activity superimposed on PMF stillwater elevation will be below an elevation of 79.2 m (260 ft.) NGVD29. These modifications are considered as Option 1 and Option 2, as follows:

- Option 1: The existing open spillway will be raised to an elevation of 73.2 m (240 ft.) NGVD29 in both spans, and a 500-ft.-wide emergency spillway will be constructed west of the existing Main Dam spillway with its crest at an elevation of 74.1 m (243 ft.) NGVD29.
- Option 2: One 25-ft. span of the existing uncontrolled ogee spillway will be raised to an elevation of 73.2 m (240 ft.) NGVD29, and a Tainter gate will be installed in the second 25-ft. span of the existing Main Dam spillway with a crest elevation of

67.1 m (220 ft.) NGVD29. The Tainter gate will be completely opened when the water elevation in the Main Reservoir reaches an elevation of 73.8 m (242 ft.) NGVD29.

For both of these options, the PMF level and the maximum water level coincident with wind-wave action were evaluated. The results show that the maximum water level due to PMF and coincident wind wave action at HAR 2, HAR 3, HNP, the Main Dam, and the Auxiliary Dam are below the elevation of 79.2 m (260 ft.) NGVD29. Out of these two options, Option 2 has been selected as the preferred option by PEC. Therefore, only Option 2 is described further.

The discharge over an ogee crest is given by the following equation (Reference 2.4-230):

$$Q = CLH_e^{3/2} \quad (3)$$

where

Q = discharge (cfs),

L = effective length of crest (ft.),

H_e = total head on the spillway crest including velocity of approach (ft.), and

C = variable discharge coefficient.

The effective length of the spillway is determined by taking contraction effects from piers and abutments into account. The effective length of the spillway (L) is determined using the following relationship:

$$L = L' - 2(NK_p + K_a)H_e \quad (4)$$

where

L' is the net length of the spillway,

N is the number of piers, and

K_p and K_a are pier and abutment contraction coefficients, respectively.

For the Main Dam, K_p = K_a = 0.01 and N = 0 when the Tainter gate is completely closed and N = 1 when the Tainter gate is completely opened. Further, K_p = K_a = 0.01 and N = 0 for the Auxiliary Dam.

The discharge coefficient C varies with the ratio of upstream dam height P to water depth above the spillway crest H₀ and with the ratio of total head H_e to design head H₀. Figures 9.23 and 9.24 in Section 9.12 of *Design of Small Dams* provide discharge coefficient curves (Reference 2.4-230). To determine the discharge

coefficients, the following relationships were developed and used in the calculations:

$$C = C_0 \left[0.86242043 + 0.13731086 \sqrt{\frac{H_e}{H_0}} \right]^2 \quad (5)$$

where, C_0 is the discharge coefficient when $H_e = H_0$ and is given as:

$$C_0 = \frac{3.115674587 + 5.584120225 \left(\frac{P}{H_0} \right) - 37.803292 \left(\frac{P}{H_0} \right)^2 + 59.93051634 \left(\frac{P}{H_0} \right)^3}{1 + 0.542416723 \left(\frac{P}{H_0} \right) - 8.21524481 \left(\frac{P}{H_0} \right)^2 + 14.50132694 \left(\frac{P}{H_0} \right)^3 + 0.102735247 \left(\frac{P}{H_0} \right)^4} \quad (6)$$

Using Equation (6), the corresponding values of P/H_0 and C_0 for the Main Dam are 3 and 3.95, respectively; the corresponding values of P/H_0 and C_0 for the Auxiliary Dam are 1.4 and 3.92, respectively. The discharge coefficient C was determined by substituting values of C_0 and H_e/H_0 in Equation (5). Figures 2.4.3-218 and 2.4.3-219 show the rating curves that were developed for spillways of the Main Dam and the Auxiliary Dam.

During the PMF event, the rate of discharge from the Auxiliary Dam may be impacted due to tail water effects from the Main Reservoir. The magnitude of impact in the discharging capacity of an outlet structure is governed by the degree of submergence from tail water. In order to correct the discharge coefficient of the Auxiliary Dam ogee spillway, the ratios of discharge coefficients at various tail water submergence levels were obtained from Figure 9.28 in Section 9.13 of *Design of Small Dams* (Reference 2.4-230). Using these data, the following function was developed and used for correcting the discharge from the Auxiliary Dam:

$$C_s = C_0 \frac{0.03 + 13.6 \frac{h_d + d}{H_e}}{1 + 10.98 \frac{h_d + d}{H_e} + 1.91 \left(\frac{h_d + d}{H_e} \right)^2} \quad (7)$$

where

H_e = actual head on the crest,

d = tailwater depth,

$h_d = H_e - d$, and

C_s = submergence correction factor.

2.4.3.3.4 Backwater Analysis

In order to assess the impact of the PMF event, including backwater effects at the HAR site, an unsteady state HEC-RAS model was developed (Reference 2.4-263). The geometric data necessary to develop such a model includes the following:

- Stream system connectivity between the Auxiliary and Main Reservoirs and their tributaries.
- Cross-section data for various tributaries.
- Hydraulic structure data for the Main Dam.

The stream system data for the Main Reservoir were determined using Geographic Information System (GIS)-based data. The reach lengths were determined by demarcating the uppermost and lowermost points on the stream and then measuring the length along the stream between two successive demarcated points. For calculating average width, stream width was determined using GIS-based data at several locations on the stream. These values were averaged to get the representative stream width. This information was used to define the river system on a reach-by-reach basis and to establish junctions at the intersections of two or more reaches. Figure 2.4.3-220 shows an overview of the study area and various sub-basins, along with various stream segments assessed in the model. The location of the HAR plant site is indicated by the shaded square shown in the inset.

PEC performed detailed bathymetric surveys to establish the current geometry of the Main Reservoir. These data were compiled into a single GIS point coverage. ArcGIS 3-D analyst Kriging sampling interpolation was used to generate a 3-D surface from the mass point data. This surface was further processed using ArcGIS to generate 1-ft. contour lines from an elevation of 67.1 m (220 ft.) NGVD29 to the approximate bottom of the Main Reservoir at 46.9 m (154 ft.) NGVD29. Two-ft. contours above the 67.1-m (220-ft.) NGVD29 lake level elevation were extracted from the Chatham and Wake County GIS databases. Using ArcGIS Append, the contour features greater than 67.1 m (220 ft.) NGVD29 located within the Buckhorn Creek drainage basin were combined with the existing contours below 67.1 m (220 ft.) NGVD29 to generate a comprehensive elevation file for the basin.

Since profiles were required, the contours were converted into a 3-D shapefile to facilitate the conversion to a 3-D computer aided design (CAD) format. In addition, an overlap analysis was performed to determine where any contour line might cross another. The resulting contour files were then post-processed in the CAD environment to generate the ground surface profiles and cross-sections in each of the basin's stream reaches.

The CAD-generated data were imported into the HEC-RAS model to define the geometric data of the Main Reservoir. Each cross-section was defined by a series of points that consist of an X-value, which establishes distance from the left bank (looking downstream), and a Y-value for elevation. Once each cross-section was

established, characteristics describing the downstream channel (the stream reach between the current cross-section and the next downstream cross-section) were defined, including the following:

- Manning's n values (Left Overbank [LOB] = 0.1, Main Channel = 0.045, Right Overbank [ROB] = 0.1)
- Main Channel Bank Stations (most were defined to an elevation of up to 67.1 m [220 ft.] NGVD29)
- Contraction and Expansion Coefficients (contraction = 0.3 and expansion = 0.6)

Following the determination of downstream channel characteristics, data describing hydraulic structures were incorporated. In this case, the only hydraulic structure to be defined was the Main Dam, which is located at the southwestern-most downstream portion of the Main Reservoir.

In addition to geometric data, unsteady flow data consisting of the boundary and initial conditions are required input for the HEC-RAS model. For this requirement, PMF inflow hydrographs generated from each sub-basin of the Main Reservoir by considering the 25 percent peaking factor were used. Figure 2.4.3-221 shows various PMF inflow hydrographs used as boundary conditions within the HEC-RAS model.

In addition to the 25 percent peaking factor, the HEC-RAS model was initiated with other conservative assumptions, such as an initial stillwater elevation of 73.3 m (240.36 ft.) NGVD29 instead of 73.2 m (240 ft.) NGVD29. This initial stillwater elevation is 0.11 m (0.36 ft.) above the proposed uncontrolled ogee crest of the Main Dam spillway.

2.4.3.4 Probable Maximum Flood Flow

PMF hydrographs for the various sub-basins and the entire Buckhorn Creek drainage basin were developed using the HEC-HMS model incorporating: (1) application of the 1-hour incremental effective PMP values tabulated in Table 2.4.3-217 to the unit hydrographs of various sub-basins considering 25 percent peaking, as presented in Figure 2.4.3-214 and Table 2.4.3-223, and (2) values of initial loss and infiltration parameters, listed in Table 2.4.3-218. The HEC-HMS model is flexible and offers many options to input precipitation, to estimate runoff hydrographs, and to manipulate and route hydrographs. HEC-HMS has been used extensively throughout the U.S. to predict stream flows in both gauged and non-gauged watersheds (Reference 2.4-224).

Based on the HNP FSAR, the base flow of the Buckhorn Creek watershed is insignificant (1.1 cfs per square mile of the drainage area) in comparison to the PMF flow. Thus, no base flow was considered for this study. Further, it was assumed that both the Main Reservoir and the Auxiliary Reservoir were completely full. In order to determine the most conservative PMF elevation, the most critical

rainfall scenario was selected. The following two PMP storms were considered from the PMP storms tabulated in Table 2.4.3-217:

- Case 1: Using the PMP corresponding to the entire basin (that is, the PMP storm given on Figure 2.4.3-206 for the entire basin).
- Case 2: Using two different PMP storms. Figure 2.4.3-206 was used as the PMP for the Main Dam watershed, and Figure 2.4.3-208 was used as the PMP for the Auxiliary Dam watershed.

In Case 1, the PMP storm corresponds to the Main Reservoir and Auxiliary Reservoir drainage basins. In this case, it was assumed that the same storm would be occurring over the entire drainage area. In Case 2, the PMP storm was considered as a mixture of two different PMP storms: one for the Auxiliary Reservoir drainage basin and a second for the Main Reservoir drainage basin. The most critical PMP storm scenario, which generates the higher peak and inflow volume, was selected. Table 2.4.3-225 provides model results in terms of both the peak flow and total inflow volume corresponding to the Case 1 and Case 2 PMP storm scenarios. Based on the results presented in Table 2.4.3-225, Case 2 was selected as the most critical rainfall scenario and has been used in the PMF analysis.

2.4.3.4.1 Probable Maximum Flood Flow from Drainage Basin above the Auxiliary Dam

The PMP storm corresponding to the Auxiliary Reservoir drainage area of 7.8 km² (3.0 mi.²), as shown in Figure 2.4.3-208, was used to estimate the PMF flow for the Auxiliary Reservoir. Figure 2.4.3-222 presents the PMF inflow and outflow hydrographs for the Auxiliary Reservoir. The peak inflow and outflow for the Auxiliary Reservoir are 197.1 m³/s (6,961.3 cfs) and 176.5 m³/s (6,234.6 cfs), respectively.

2.4.3.4.2 Probable Maximum Flood Flow from Drainage Basin above the Main Dam

The drainage area that contributes runoff to the Main Reservoir is 182.1 km² (70.3 mi.²). This area includes the 7.8-km² (3.0-mi.²) drainage area above the Auxiliary Dam, as described in FSAR Subsection 2.4.3.4.1. As described in FSAR Subsection 2.4.3.4.1, to determine the PMF inflow and outflow hydrographs, the Case 2 PMP storm was considered. Case 2 uses two different PMP storms: Figure 2.4.3-206 was used as the PMP for the Main Reservoir watershed, and Figure 2.4.3-208 was used as the PMP for the Auxiliary Reservoir watershed. Figure 2.4.3-223 presents the inflow and outflow hydrographs for the Main Reservoir. The peak inflow and outflow for the Main Reservoir are 3,539.7 m³/s (125,020.2 cfs) and 596.0 m³/s (21,050.1 cfs), respectively.

2.4.3.5 Water Level Determinations

Using the level pool routing technique, along with the stage-storage curve and storage-out flow curve of dam outlet works for the Auxiliary and Main Reservoirs within the HEC-HMS model, PMF stillwater elevations were determined for both the Auxiliary and Main Reservoirs. Figures 2.4.3-224 and 2.4.3-225 present the obtained stillwater elevations for the Auxiliary Reservoir and the Main Reservoir, respectively. The peak stillwater elevations in the Auxiliary Reservoir and Main Reservoir were found to be 78.2 m (256.56 ft.) NGVD29 and 76.9 m (252.25 ft.) NGVD29, respectively.

As discussed in FSAR Subsection 2.4.3.3.4, the HEC-RAS model was initiated with conservative initial conditions. The results of the HEC-RAS model run for the PMF event are shown on Figures 2.4.3-226, 2.4.3-227, 2.4.3-228, and 2.4.3-229. Figure 2.4.3-226 presents time series of stage (ft. NGVD29) and flow (cfs) values for Thomas Creek, a tributary of the Main Reservoir adjacent to the HAR plant site. Based on this plot, the stage peaked on day 9 at an elevation of 77.0 m (252.76 ft.) NGVD29.

Figures 2.4.3-227 and 2.4.3-228 show cross-sections depicting the maximum stage at the two locations immediately upstream and downstream of the HAR plant site. Again, the maximum stage at these locations during the PMF event is 77.0 m (252.76 ft.) NGVD29. Similarly, Figure 2.4.3-229 presents the maximum stage for all cross-section locations from the downstream end of the Main Reservoir (Main Dam) to the upstream end (Thomas Creek, upstream of the HAR plant site). The maximum stage during the PMF event for all locations, including backwater effects, is 77.0 m (252.76 ft.) NGVD29. These values are provided in tabular format in Table 2.4.3-226.

The final stillwater PMF elevations obtained using HEC-RAS and HEC-HMS models are summarized in Table 2.4.3-227. The maximum stillwater elevations in the Auxiliary Reservoir and Main Reservoir are 78.2 m (256.56 ft.) NGVD29 and 77.0 m (252.76 ft.) NGVD29, respectively.

2.4.3.6 Coincident Wind-Wave Activity

As discussed in FSAR Subsection 2.4.2.2, safety-related structures and facilities for the HAR site are protected against floods and flood waves caused by probable maximum events, such as the PMF and the PMH. Coincident wind-wave activity was evaluated at HAR 2, HAR 3, HNP, the Auxiliary Dam, and the Main Dam. In the context of the HAR site, the Auxiliary Dam and Main Dam are not safety-related structures. However, in the context of HNP, the Auxiliary Dam is a safety-related structure, whereas the Main Dam is not a safety-related structure. In this analysis, however, both the Auxiliary and Main Dams have been considered for evaluating the coincident wind-wave activity. For the wind-wave activity analyses, the USACE's *Coastal Engineering Manual, Engineer Manual 1110-2-1100* (Part II) was strictly followed (Reference 2.4-261):

In order to determine wind setup and wave runup for a given site, the following data were required:

- Water body bathymetry data
- Critical fetch distances
- Over-wind speed averaged for an appropriate duration
- Site characteristics, such as protection type and material and slope

2.4.3.6.1 Bathymetry Data

PEC performed detailed bathymetric surveys to establish the current geometry of the Main Reservoir with thousands of depth-to-bottom measurements collected during the study. These data were compiled into a single GIS point coverage. ArcGIS 3-D analyst Kriging sampling interpolation was used to generate a 3-D surface from the mass point data.

The locations of interest for determining the wind-wave activity for HAR 2, HAR 3, HNP, the Auxiliary Dam, and the Main Dam are shown on Figures 2.4.3-230, 2.4.3-231, 2.4.3-232, and 2.4.3-233. For wind and wave calculation purposes, water depths at various locations were determined using: (1) the bottom elevation of the lake near the location at which wind-wave activity had been determined, and (2) the stillwater PMF elevation for a given scenario and option. Table 2.4.3-228 presents the lake bottom elevations that were used in the wind-wave activity analysis.

2.4.3.6.2 Determination of Fetch for the Main Reservoir and the Auxiliary Reservoir

Fetch is the length of water surface exposed to wind during the generation of waves. Fetch is an important characteristic of open water because longer fetch can result in larger wind-generated waves. For this analysis, straight line fetch distances were used in the wave runup calculations, as detailed in the EM 1110-2-1100 (Part II) (Reference 2.4-261). Selected straight line fetches associated with HAR 2, HAR 3, HNP, the Auxiliary Dam, and the Main Dam are shown on Figures 2.4.3-230, 2.4.3-231, 2.4.3-232, and 2.4.3-233. Tables 2.4.3-229, 2.4.3-230, 2.4.3-231, and 2.4.3-232 provide the over water fetch distances. The critical fetch distances for the HAR 2, HAR 3, HNP, Auxiliary Dam, and Main Dam locations are 0.93, 0.85, 4.33, 4.29, and 4.29 mi., respectively.

2.4.3.6.3 Over Water Wind Speed

According to ANSI/ANS 2.8-1992 (Reference 2.4-222), a 2-year wind speed should be used while conducting the coincident wind-wave activity analysis. Therefore, the 2-year wind speed was obtained from ANSI/ANS 2.8-1992 (Reference 2.4-222) and used for conducting the coincident wind-wave activity analysis at the HAR site. The 2-year wind speed at the HAR site is 50 mph. Before using this wind speed in the calculation of wave runup, several adjustments were applied following the procedure outlined in the EM 1110-2-1100 (Part II) (Reference 2.4-261). The step-by-step procedure is as follows:

1. Standard measurements should be collected at 10 meters above ground surface. Since the wind speed obtained from the ANSI/ANS 2.8.1992 (Reference 2.4-222)

was measured at 10 m (30 ft.) above ground, no adjustment was applied.

2. The averaging duration associated with the 2-year wind speed obtained from ANSI/ANS 2.8-1992 is assumed to be 1 hour (Reference 2.4-222). Using a 2-year wind speed of 50 mph, its averaging duration, and Figure II-2-1 of EM 1110-2-1100 (Part II) (Reference 2.4-261) (Figure 2.4.3-234), the 1-hour wind speed was calculated as 50.0 mph.

3. Overwater wind speeds at various locations were then determined by applying a correction for transition from land to water. This correction factor was determined using Figure II-2-7 of EM 1110-2-1100 (Part II) (Reference 2.4-261) (Figure 2.4.3-235). (Note: The HAR site is approximately 140 mi. from the coast line.)

According to this figure (Figure 2.4.3-235), the correction factor R_L is given as follows (Reference 2.4-261):

$$U_w = R_L * U_L \quad (8)$$

where

U_w is the over water wind speed,

U_L is the overland wind speed, and

R_L is a correction factor which is equal to 0.9 for $U_L > 41.5$ mph.

In this case, the overland wind speed, U_L , is 50.0 mph (20.1 m/s). As such, the correction factor, R_L , is equal to 0.9. However, in an effort to be conservative, a correction factor, R_L , equal to 1.0 was used for this analysis.

4. Finally, the wind speed was corrected according to the appropriate averaging duration. When a sustained wind with essentially constant direction over a fetch for sufficient time achieves steady-state, fetch-limited values, simplified wave predictions can provide accurate estimates of wave conditions. The time required to accomplish fetch-limited wave development for short fetches was calculated as follows (Reference 2.4-261):

$$t_{x,u} = 77.23 \frac{X^{0.67}}{u^{0.34} g^{0.33}} \quad (9)$$

where

$t_{x,u}$ is the time required for waves crossing a fetch of length x under a wind of velocity u to become fetch-limited.

The resulting averaging time interval, $t_{x,u}$ was then used in conjunction with Figure II-2-1 of EM 1110-2-1100 (Part II) (Figure 2.3.4-234) in order to determine the appropriate wind speed for various locations. Table 2.3.4-233 presents calculated corrections of wind averaging intervals for various fetch lines directed toward HAR 2, HAR 3, HNP, the Auxiliary Dam, and the Main Dam.

2.4.3.6.4 Site Characteristics Such as Type and Material of Protection and Slope

The HAR is surrounded by the Thomas Creek Branch of the Main Reservoir on the East side and by the Auxiliary Reservoir on the West side. Figure 2.4.1-205 shows the planned site drainage plan and indicates that the HAR will have permeable natural land area between the developed site and the water bodies. Using site-specific topographic and bathymetry data, the land slope adjacent to the Thomas Creek Branch of the Main Reservoir and Auxiliary Reservoir were determined; these slopes are 0.09 and 0.13 for the east and west sides of the HAR, respectively.

The upstream faces of the Main Dam and Auxiliary Dam are protected by riprap with slopes of 1(V): 2(H) and 1(V):2.5(H), respectively. On the plant island, the embankments located on the Auxiliary Reservoir (Line 1 as shown on Figure 2.3.4-233, which is directed toward the plant island from the Auxiliary Reservoir) and the Main Reservoir (Lines 2 and 3 as shown on Figure 2.3.4-233, which are directed towards the plant island from the Main Reservoir) are protected by sacrificial spoil fill. The embankment intersected by Line 1 has a slope of 1(V):10(H) whereas the embankment intersected by Lines 2 and 3 have a slope of 1(V):10(H).

2.4.3.6.5 Wave Runup

Having determined the estimate of winds for wave prediction, wave runup for various fetch lines directed toward HAR 2, HAR 3, HNP, the Auxiliary Dam, and the Main Dam were calculated according to the step-by-step procedure given in the EM 1110-2-1100 (Part VI) (Reference 2.4-261). The step-by-step procedure is as follows:

1. Using the previously determined fetch lengths and wind speeds, estimates of significant wave heights were obtained using the deepwater nomogram for the fetch limited wave heights given in the EM 1110-2-1100 (Part II) (Figure 2.3.4-236).
2. Similarly, estimates of peak wave periods were obtained using the deepwater nomogram for the fetch limited wave periods given in the EM 1110-2-1100 (Part II) (Figure 2.3.4-237).
3. The peak wave periods for various fetch lines calculated above were compared with the shallow-water limit. According to the CEM (Reference 2.4-261), the shallow-water limit is given by the following equation:

$$T_p \approx 9.78 \left(\frac{d}{g} \right)^{\frac{1}{2}} \quad (10)$$

where

T_p = the limiting wave period in sec,
 d = the water depth in meter, and
 g = the gravitational acceleration in meter/sec².

If the predicted peak wave period for a given fetch line is greater than the limiting value, then the predicted wave period was reduced to the limiting wave period.

Conversely, if the predicted wave period was less than the limiting value, the predicted deepwater wave period was retained and used for further calculations.

4. Wave runup was calculated using the wave runup equation on permeable slopes given in Chapter 5 of *EM 1110-2-1100* (Part VI) (Reference 2.4-261). According to the CEM, the runup equation for various levels of percentage exceedances is given as:

$$R_{ui\%}/H_S = A\xi_{om} \text{ for } 1.0 < \xi_{om} \leq 1.5 \quad (11a)$$

$$R_{ui\%}/H_S = B\xi_{om}^C \text{ for } 1.5 < \xi_{om} \leq (D/B)^{1/C} \quad (11b)$$

$$R_{ui\%}/H_S = D \text{ for } (D/B)^{1/C} < \xi_{om} \leq 7.5 \quad (11c)$$

where, ξ_{om} is the surf-similarity parameter for irregular waves defined as:

$$\xi_{om} = \frac{\tan \alpha}{\sqrt{s_{om}}} \quad (12)$$

In which α is an angle defined by arctangent of the slope of structure (dam)/embankment or stream or reservoir bank, and S_{om} is the fictitious wave steepness. S_{om} is the ratio between the statistical wave height at the structure and representative deepwater wavelengths and is defined as:

$$s_{om} = \frac{H_s}{L_{om}} = \frac{2\pi}{g} \frac{H_s}{T_m^2} \quad (13)$$

where

H_s = significant wave height of incident waves at the toe of the structure,

L_{om} = deepwater wavelength,

T_m = mean wave period, and

T_p = wave period corresponding to the peak of the wave spectrum.

The coefficients A through D in Equations 11a, 11b, and 11c for runup of irregular head-on waves on impermeable and permeable rock armored slopes are given in Table 2.4.3-234 (Reference 2.4-261). Using Steps 1 through 4 for various fetch lines, runup was calculated. Table 2.4.3-235 presents the runup results.

2.4.3.6.6 Wind Setup

Sustained wind over a water body exerts a horizontal stress on the water surface in the wind direction. In an enclosed water body, this wind effect results in a surplus of water at the leeward end and a decrease in water level at the windward end. This effect is called wind setup. According to *EM 1110-2-1420* (Reference 2.4-262), the

wind setup in lakes and reservoirs can be estimated using the Zeider Zee equation given as:

$$S = \frac{U^2 X}{1400d} \quad (14)$$

where

S = the setup (ft.) above the stillwater level,

U = the wind speed (mph),

X = the fetch length (mi.), and

d = the water depth corresponding to the PMF level.

Table 2.4.3-236 presents the setup calculation results using Equation (14) for the HAR 2, HAR 3, HNP, Auxiliary Dam, and Main Dam Locations.

2.4.3.6.7 Overall PMF Elevation

The values of stillwater elevation, wave runup, and wind setup for various fetch lines were added together to determine the PMF elevation coincident with wind-wave activity at HAR 2, HAR 3, HNP 1, the Auxiliary Dam, and the Main Dam. Table 2.4.3-237 presents the overall PMF elevations for these locations.

2.4.3.6.8 Maximum PMF Elevation due to Coincident Wind-Wave Activity

The maximum PMF elevations for HAR 2, HAR 3, HNP 1, the Auxiliary Dam, and the Main Dam are summarized in Table 2.4.3-238. None of the PMF elevations exceeded the target elevation of 79.2 m (260 ft.) NGVD29. Therefore, no potential hazard exists to the plant's safety-related facilities as a result of the effect of the PMF.

2.4.16 References

In this section, only the references for HAR FSAR Subsection 2.4.3 that need to be deleted or added are shown. All other references in HAR FSAR Rev. 1 remain unchanged.

- 2.4-232 U.S. Army Corps of Engineers (USACE), Engineering and Design, "Wave Runup and Wind Setup on Reservoir Embankments," Engineer Technical Letter No. 1110-2-221, November 1976.
- 2.4-233 Fouad, H. F., and A.C. Elizabeth, "Evaluating the Design Safety of Highway Structural Support," August 2001, University Transportation Center for Alabama, utca.eng.ua.edu/projects/final_reports/00218report.htm.
- 2.4-259 Viessman, W., and G. L. Lewis, "Introduction to Hydrology," Fourth Edition, 1996.
- 2.4-260 U.S. Geological Survey, Estimating the Magnitude and Frequency of Floods in Rural Basins of North Carolina—Revised. Water-Resources Investigations Report 01-4207, Raleigh, North Carolina, 2001.
- 2.4-261 U.S. Army Corps of Engineers' Coastal Engineering Manual, Engineer Manual 1110-2-1100 (Part II) (U.S. Army Corps of Engineers. 2006.
- 2.4-262 U.S. Army Corps of Engineers (USACE), Engineering and Design - Hydrologic Engineering Requirements for Reservoirs, Washington, DC, EM 1110-2-1420, October, 1997.
- 2.4-263 U.S. Army Corps of Engineers (USACE), Hydrologic Engineering Center, "HEC-RAS River Analysis System," Version 4.0, March 2008.

HAR COL 2.4-2

Table 2.4.3-201
6-Hour Incremental PMP Depths (HMR 51)

Area (mi. ²)	PMP Depths (inches) for Various Durations				
	6-Hr.	12-Hr.	24-Hr.	48-Hr.	72-Hr.
10	29.8	35.5	41	45	47.4
200	21.8	26	31	35.5	37.0
1000	15.8	20.8	26	30	31.5
5000	9.25	13.25	18	22	23.6

HAR COL 2.4-2

Table 2.4.3-202
6-Hour Incremental PMP Depths (after Smoothing)

Area (mi. ²)	PMP Depths (inches) for Various Durations				
	6-Hr.	12-Hr.	24-Hr.	48-Hr.	72-Hr.
10	29.9	35.5	41.0	45.0	47.1
200	21.8	26.1	31.2	35.5	37.1
1000	15.8	20.5	25.7	30.0	31.7
5000	9.3	13.4	18.1	22.0	23.6

HAR COL 2.4-2

Table 2.4.3-203
Depth-Area-Duration Values for the Selected Standard Areas at 35°38'00" N, 78°57'22" W

Duration (hr.)	PMP Values (inches) for Selected Standard Areas						
	10 mi. ²	25 mi. ²	50 mi. ²	100 mi. ²	175 mi. ²	300 mi. ²	450 mi. ²
6	29.86	27.20	25.57	23.84	22.25	20.49	19.02
12	35.49	31.65	29.75	28.01	26.52	24.92	23.58
24	40.99	36.71	34.70	32.96	31.52	29.99	28.71
48	45.00	40.90	38.94	37.24	35.83	34.33	33.04
72	47.05	42.66	40.61	38.87	37.45	35.95	34.69

HAR COL 2.4-2

Table 2.4.3-204
Interpolated PMP Values for 18-Hour Duration

Area (mi. ²)	PMP Depth (inch)
10	38.24
25	34.18
50	32.23
100	30.48
175	29.02
300	27.46
450	26.14

HAR COL 2.4-2

Table 2.4.3-205
Incremental Differences for the First Three 6-Hour Periods

Area (mi. ²)	Incremental Difference (inch)		
	1st 6-hr. period	2nd 6-hr. period	3rd 6-hr. period
10	29.9	5.63	2.75
25	27.2	4.46	2.53
50	25.6	4.19	2.48
100	23.8	4.17	2.47
175	22.2	4.28	2.50
300	20.5	4.43	2.54
450	19.0	4.56	2.57

HAR COL 2.4-2

Table 2.4.3-206
Incremental Differences for the First Three 6-Hour Periods Based on
Smooth Curves of Figure 2.4.3-204

Area (mi. ²)	Incremental Difference (inch)		
	1st 6-hr. period	2nd 6-hr. period	3rd 6-hr. period
10	29.9	5.63	2.75
25	27.2	5.31	2.69
50	25.6	5.08	2.65
100	23.8	4.84	2.60
175	22.2	4.64	2.56
300	20.5	4.46	2.53
450	19.0	4.32	2.50

HAR COL 2.4-2

Table 2.4.3-207
Computation Sheet for First 6-Hour Duration

	I	II	III	IV	V	VI		I	II	III	IV	V	VI
Area Size (mi. ²)	Iso	Nomo	Amt. 27.2	Avg. Depth	Delta A	Delta V	Area Size (mi. ²)	Iso	Nomo	Amt. 22.2	Avg. Depth	Delta A	Delta V
25	A	102	27.74	27.74	10	277.40	175	A	119	26.47	26.47	10	264.73
	B	95	25.84	26.79	15	401.82		B	111	24.69	25.58	15	383.75
	C	67	18.22	22.03	25	550.72		C	103	22.91	23.80	25	595.08
	D	52	14.14	16.59	20.3	336.77		D	96	21.36	22.29	20.3	452.50
		316			Sum =	1566.71						Sum =	1696.06
Area Size (mi. ²)	Iso	Nomo	Amt. 25.6	Avg. Depth	Delta A	Delta V	Area Size (mi. ²)	Iso	Nomo	Amt. 20.5	Avg. Depth	Delta A	Delta V
50	A	106	27.10	27.10	10	270.99	300	A	126	25.81	25.81	10	258.15
	B	99	25.31	26.20	15	393.07		B	118	24.18	25.00	15	374.93
	C	92	23.52	24.41	25	610.37		C	110	22.54	23.36	25	583.91
	D	66	16.87	20.86	20.3	423.48		D	103	21.10	21.96	20.3	445.85
					Sum =	1697.91						Sum =	1662.84
Area Size (mi. ²)	Iso	Nomo	Amt. 23.8	Avg. Depth	Delta A	Delta V	Area Size (mi. ²)	Iso	Nomo	Amt. 19.0	Avg. Depth	Delta A	Delta V
100	25.10	112	26.70	26.70	10	267.01	450	A	132	25.10	25.10	10	251.03
	23.58	105	25.03	25.87	15	388.00		B	124	23.58	24.34	15	365.14
	22.06	98	23.36	24.20	25	604.94		C	116	22.06	22.82	25	570.53
	20.54	90	21.46	22.60	20.3	458.79		D	108	20.54	21.45	20.3	435.47
					Sum =	1718.74						Sum =	1622.16

Note:
* = Weighting factor (HMR 52 Section 7.1 Step C6)

HAR COL 2.4-2

Table 2.4.3-208
Computation Sheet for Second 6-Hour Duration

	I	II	III	IV	V	VI		I	II	III	IV	V	VI
Area Size (mi. ²)	Iso	Nomo	Amt. 5.31	Avg. Depth	Delta A	Delta V	Area Size (mi. ²)	Iso	Nomo	Amt. 4.64	Avg. Depth	Delta A	Delta V
25	A	103	5.47	5.47	10	54.73	175	A	110	5.11	5.11	10	51.09
	B	98	5.21	5.34	15	80.10		B	105	4.88	4.99	15	74.90
	C	72	3.83	4.52	25	112.91		C	101.5	4.71	4.80	25	119.89
	D	59	3.13	3.55	20.3	72.05		D	97.5	4.53	4.64	20.3	94.20
					Sum =	319.78						Sum =	340.08
Area Size (mi. ²)	Iso	Nomo	Amt. 5.08	Avg. Depth	Delta A	Delta V	Area Size (mi. ²)	Iso	Nomo	Amt. 4.46	Avg. Depth	Delta A	Delta V
50	A	105.5	5.35	5.35	10	53.54	300	A	111.5	4.97	4.97	10	49.72
	B	100.5	5.10	5.23	15	78.41		B	107	4.77	4.87	15	73.08
	C	96.5	4.90	5.00	25	124.98		C	103.5	4.62	4.69	25	117.34
	D	76	3.86	4.48	20.3	90.97		D	100	4.46	4.55	20.3	92.43
					Sum =	347.90						Sum =	332.58
Area Size (mi. ²)	Iso	Nomo	Amt. 4.84	Avg. Depth	Delta A	Delta V	Area Size (mi. ²)	Iso	Nomo	Amt. 4.32	Avg. Depth	Delta A	Delta V
100	A	108	5.22	5.22	10	52.24	450	A	113	4.88	4.88	10	48.82
	B	103	4.98	5.10	15	76.55		B	109	4.71	4.80	15	71.93
	C	99	4.79	4.89	25	122.14		C	105	4.54	4.62	25	115.57
	D	95	4.60	4.71	20.3	95.64		D	102	4.41	4.48	20.3	91.04
					Sum =	346.56						Sum =	327.36

Note:
* = Weighting factor (HMR 52 Section 7.1 Step C6)

HAR COL 2.4-2

Table 2.4.3-209
Computation Sheet for Third 6-Hour Duration

	I	II	III	IV	V	VI		I	II	III	IV	V	VI
Area Size (mi. ²)	Iso	Nomo	Amt. 27.2	Avg. Depth	Delta A	Delta V	Area Size (mi. ²)	Iso	Nomo	Amt. 22.2	Avg. Depth	Delta A	Delta V
25	A	101	2.72	2.72	10	27.17	175	A	102.8	2.64	2.64	10	26.36
	B	99	2.66	2.69	15	40.35		B	101.3	2.60	2.62	15	39.25
	C	74.5	2.00	2.33	25	58.34		C	100	2.56	2.58	25	64.52
	D	60.5	1.63	1.85	20.3	37.62		D	99.2	2.54	2.56	20.3	51.88
					Sum =	163.48						Sum =	182.01
Area Size (mi. ²)	Iso	Nomo	Amt. 2.65	Avg. Depth	Delta A	Delta V	Area Size (mi. ²)	Iso	Nomo	Amt. 2.53	Avg. Depth	Delta A	Delta V
50	A	101.6	2.69	2.69	10	26.87	300	A	103.4	2.62	2.62	10	26.15
	B	99.8	2.64	2.66	15	39.95		B	101.9	2.58	2.60	15	38.94
	C	98.5	2.61	2.62	25	65.56		C	100.7	2.55	2.56	25	64.05
	D	78.5	2.08	2.39	20.3	48.59		D	99.8	2.52	2.54	20.3	51.52
					Sum =	180.99						Sum =	180.66
Area Size (mi. ²)	Iso	Nomo	Amt. 2.60	Avg. Depth	Delta A	Delta V	Area Size (mi. ²)	Iso	Nomo	Amt. 2.50	Avg. Depth	Delta A	Delta V
100	A	102.3	2.66	2.66	10	26.60	450	A	103.8	2.60	2.60	10	25.98
	B	100.7	2.62	2.64	15	39.59		B	102.4	2.56	2.58	15	38.71
	C	99.3	2.58	2.60	25	65.01		C	101.2	2.53	2.55	25	63.70
	D	98.6	2.56	2.57	20.3	52.27		D	100.3	2.51	2.52	20.3	51.24
					Sum =	183.46						Sum =	179.63

Note:
* = Weighting factor (HMR 52 Section 7.1 Step C6)

HAR COL 2.4-2

Table 2.4.3-210
Incremental Average Depths for Each 6-Hour Period
for 100-mi.² Drainage Area

Increment	Duration (hr.)	Cumulative PMP (inches)	Incremental PMP (inches)
1	6	23.90	23.90
2	12	27.86	3.97
3	18	30.70	2.84
4	24	32.80	2.10
5	30	34.39	1.58
6	36	35.60	1.21
7	42	36.54	0.94
8	48	37.27	0.73
9	54	37.84	0.57
10	60	38.28	0.44
11	66	38.61	0.33
12	72	38.86	0.25

HAR COL 2.4-2

Table 2.4.3-211
72-Hour Drainage Isohyet Values

Isohyet	6-Hr. Periods											
	1	2	3	4	5	6	7	8	9	10	11	12
A	26.70	5.22	2.66	2.10	1.58	1.21	0.94	0.73	0.57	0.44	0.33	0.25
B	25.03	4.98	2.62	2.10	1.58	1.21	0.94	0.73	0.57	0.44	0.33	0.25
C	23.36	4.79	2.58	2.10	1.58	1.21	0.94	0.73	0.57	0.44	0.33	0.25
D	21.46	4.60	2.56	1.71	1.29	0.99	0.77	0.59	0.46	0.36	0.27	0.20

HAR COL 2.4-2

Table 2.4.3-212
Computation of Drainage Average Depths (Increments 1 to 6)

Increment #1							Increment #4						
	I	II	III	IV	V	VI		I	II	III	IV	V	VI
Area Size (mi. ²)	Iso	Nomo	Amt.	Avg. Depth	ΔA	ΔV	Area Size (mi. ²)	Iso	Nomo	Amt.	Avg. Depth	ΔA	ΔV
100	A	112	26.76	26.76	10	267.65	100	A	100	2.10	2.10	10	21.00
	B	105	25.09	25.93	15	388.93		B	100	2.10	2.10	15	31.49
	C	98	23.42	24.26	25	606.39		C	100	2.10	2.10	25	52.49
	D	90	21.51	22.46	20.3	456.01		D	78.5	1.65	1.87	20.3	38.04
					Sum =	1718.98						Sum =	143.02
					Average Depth =	24.45						Average Depth =	2.03
Increment #2							Increment #5						
Area Size (mi. ²)	Iso	Nomo	Amt.	Avg. Depth	ΔA	ΔV	Area Size (mi. ²)	Iso	Nomo	Amt.	Avg. Depth	ΔA	ΔV
100	A	108	4.28	4.28	10	42.83	100	A	100	1.58	1.58	10	15.85
	B	103	4.08	4.18	15	62.75		B	100	1.58	1.58	15	23.77
	C	99	3.93	4.01	25	100.13		C	100	1.58	1.58	25	39.62
	D	95	3.77	3.85	20.3	78.08		D	78.5	1.24	1.41	20.3	28.71
					Sum =	283.79						Sum =	107.96
					Average Depth =	4.04						Average Depth =	1.54
Increment #3							Increment #6						
Area Size (mi. ²)	Iso	Nomo	Amt.	Avg. Depth	ΔA	ΔV	Area Size (mi. ²)	Iso	Nomo	Amt.	Avg. Depth	ΔA	ΔV
100	A	102.3	2.91	2.91	10	29.07	100	A	100	1.21	1.21	10	12.14
	B	100.7	2.86	2.88	15	43.26		B	100	1.21	1.21	15	18.21
	C	99.3	2.82	2.84	25	71.04		C	100	1.21	1.21	25	30.35
	D	98.6	2.80	2.81	20.3	57.08		D	78.5	0.95	1.08	20.3	22.00
					Sum =	200.45						Sum =	82.70
					Average Depth =	2.85						Average Depth =	1.18

HAR COL 2.4-2

Table 2.4.3-213
Computation of Drainage Average Depths (Increments 7 to 12)

Increment #7							Increment #10							
	I	II	III	IV	V	VI		I	II	III	IV	V	VI	
Area Size (mi. ²)	Iso	Nomo	Amt.	Avg. Depth	ΔA	ΔV	Area Size (mi. ²)	Iso	Nomo	Amt.	Avg. Depth	ΔA	ΔV	
100	A	100	0.94	0.94	10	9.39	100	A	100	0.44	0.44	10	4.38	
	B	100	0.94	0.94	15	14.08		B	100	0.44	0.44	15	6.57	
	C	100	0.94	0.94	25	23.47		C	100	0.44	0.44	25	10.95	
	D	78.5	0.74	0.84	20.3	17.01		D	78.5	0.34	0.39	20.3	7.93	
			Sum =			63.95					Sum =			29.83
			Average Depth =			0.91					Average Depth =			0.42
Increment #8							Increment #11							
Area Size (mi. ²)	Iso	Nomo	Amt.	Avg. Depth	ΔA	ΔV	Area Size (mi. ²)	Iso	Nomo	Amt.	Avg. Depth	ΔA	ΔV	
100	A	100	0.73	0.73	10	7.29	100	A	100	0.33	0.33	10	3.35	
	B	100	0.73	0.73	15	10.94		B	100	0.33	0.33	15	5.02	
	C	100	0.73	0.73	25	18.23		C	100	0.33	0.33	25	8.37	
	D	78.5	0.57	0.65	20.3	13.21		D	78.5	0.26	0.30	20.3	6.07	
			Sum =			49.67					Sum =			22.81
			Average Depth =			0.71					Average Depth =			0.32
Increment #9							Increment #12							
Area Size (mi. ²)	Iso	Nomo	Amt.	Avg. Depth	ΔA	ΔV	Area Size (mi. ²)	Iso	Nomo	Amt.	Avg. Depth	ΔA	ΔV	
100	A	100	0.57	0.57	10	5.66	100	A	100	0.25	0.25	10	2.51	
	B	100	0.57	0.57	15	8.50		B	100	0.25	0.25	15	3.77	
	C	100	0.57	0.57	25	14.16		C	100	0.25	0.25	25	6.28	
	D	78.5	0.44	0.51	20.3	10.26		D	78.5	0.20	0.22	20.3	4.55	
			Sum =			38.59					Sum =			17.11
			Average Depth =			0.55					Average Depth =			0.24

HAR COL 2.4-2

Table 2.4.3-214
72-Hour Total Drainage – Averaged PMP

Increment	Duration (hr.)	Drainage Averaged PMP (inches)	Incremental Drainage Averaged PMP (inches)
1	6	24.45	24.45
2	12	28.49	4.04
3	18	31.34	2.85
4	24	33.37	2.03
5	30	34.91	1.54
6	36	36.09	1.18
7	42	37.00	0.91
8	48	37.70	0.71
9	54	38.25	0.55
10	60	38.68	0.42
11	66	39.00	0.32
12	72	39.24	0.24

HAR COL 2.4-2

Table 2.4.3-215
Distribution of PMP According to ANSI/ANS-2.8-1992

6-hour Period	Time (hours)	Incremental Average PMP	ANSI Sequence	Sequence No.	ANSI Storm Distribution	Storm Pattern	Cumulative PMP
1	6	24.45	4	1 st -day	2.03	0.71	0.71
2	12	4.04	2		4.04	0.91	1.62
3	18	2.85	1		24.45	1.18	2.79
4	24	2.03	3		2.85	1.54	4.33
5	30	1.54	8	2 nd -day	0.71	2.03	6.36
6	36	1.18	6		1.18	4.04	10.40
7	42	0.91	5		1.54	24.45	34.85
8	48	0.71	7		0.91	2.85	37.70
9	54	0.55	12	3 rd -day	0.24	0.55	38.25
10	60	0.42	10		0.42	0.42	38.68
11	66	0.32	9		0.55	0.32	39.00
12	72	0.24	11		0.32	0.24	39.24

Notes:
PMP depths are in inches.

HAR COL 2.4-2

Table 2.4.3-216
Incremental Probable Maximum Precipitation
for the Main Dam and the Auxiliary Dam

Time (hr.)	Incremental PMP for the Main Dam (inches)	Incremental PMP for the Auxiliary Dam (inches)	Time (hr.)	Incremental PMP for the Main Dam (inches)	Incremental PMP for the Auxiliary Dam (inches)
1	0.08	0.10	37	1.53	1.95
2	0.08	0.10	38	2.37	3.01
3	0.09	0.11	39	3.77	4.79
4	0.09	0.11	40	10.45	13.27
5	0.09	0.11	41	3.12	3.97
6	0.09	0.11	42	2.08	2.64
7	0.10	0.12	43	0.54	0.69
8	0.10	0.12	44	0.48	0.61
9	0.10	0.12	45	0.44	0.56
10	0.10	0.12	46	0.40	0.51
11	0.11	0.14	47	0.38	0.49
12	0.11	0.14	48	0.35	0.45
13	0.13	0.16	49	0.23	0.29
14	0.13	0.16	50	0.23	0.29
15	0.13	0.16	51	0.23	0.29
16	0.14	0.17	52	0.23	0.29
17	0.14	0.17	53	0.23	0.29
18	0.14	0.17	54	0.23	0.29
19	0.18	0.22	55	0.16	0.20
20	0.18	0.22	56	0.16	0.20
21	0.18	0.22	57	0.16	0.20
22	0.19	0.24	58	0.15	0.19
23	0.19	0.24	59	0.15	0.19
24	0.19	0.24	60	0.15	0.19
25	0.26	0.32	61	0.12	0.15
26	0.27	0.35	62	0.12	0.15
27	0.28	0.36	63	0.12	0.15
28	0.30	0.39	64	0.12	0.15
29	0.31	0.40	65	0.12	0.15
30	0.34	0.44	66	0.11	0.14
31	0.61	0.77	67	0.10	0.12
32	0.68	0.86	68	0.10	0.12
33	0.77	0.97	69	0.10	0.12
34	0.87	1.11	70	0.09	0.11
35	0.98	1.25	71	0.09	0.11
36	1.11	1.41	72	0.09	0.11

HAR COL 2.4-2

Table 2.4.3-217
Incremental Effective Probable Maximum Precipitation
for the Main Dam and the Auxiliary Dam

Time (hr.)	Incremental PMP for the Main Dam (inches)	Incremental PMP for the Auxiliary Dam (inches)	Time (hr.)	Incremental PMP for the Main Dam (inches)	Incremental PMP for the Auxiliary Dam (inches)
1	0.00	0.00	37	0.54	0.71
2	0.00	0.00	38	0.88	1.13
3	0.00	0.00	39	1.44	1.85
4	0.00	0.00	40	4.11	5.24
5	0.00	0.00	41	1.18	1.52
6	0.00	0.00	42	0.77	1.00
7	0.00	0.00	43	0.16	0.21
8	0.00	0.00	44	0.13	0.19
9	0.00	0.00	45	0.12	0.17
10	0.00	0.00	46	0.11	0.15
11	0.00	0.00	47	0.10	0.14
12	0.00	0.00	48	0.09	0.13
13	0.00	0.00	49	0.04	0.07
14	0.00	0.00	50	0.04	0.07
15	0.00	0.00	51	0.05	0.07
16	0.00	0.00	52	0.05	0.07
17	0.00	0.00	53	0.05	0.08
18	0.00	0.00	54	0.05	0.08
19	0.00	0.00	55	0.03	0.04
20	0.00	0.00	56	0.03	0.05
21	0.00	0.00	57	0.03	0.05
22	0.00	0.00	58	0.03	0.05
23	0.00	0.00	59	0.03	0.05
24	0.00	0.00	60	0.03	0.05
25	0.00	0.03	61	0.02	0.04
26	0.01	0.04	62	0.03	0.04
27	0.02	0.05	63	0.03	0.04
28	0.03	0.06	64	0.03	0.04
29	0.04	0.07	65	0.03	0.05
30	0.05	0.09	66	0.03	0.04
31	0.16	0.22	67	0.03	0.04
32	0.19	0.26	68	0.03	0.04
33	0.23	0.31	69	0.03	0.04
34	0.27	0.36	70	0.03	0.04
35	0.32	0.42	71	0.03	0.04
36	0.37	0.49	72	0.04	0.04

Notes:

Total Actual Precipitation = (Main) 15.70 inches ; (Auxiliary) 19.94 inches
 Total Infiltration = (Main) 3.55 inches; (Auxiliary) 3.88 inches
 Total Effective Precipitation = (Main) 12.14 inches. ; (Auxiliary) 16.06 inches

HAR COL 2.4-2

Table 2.4.3-218
Sub-Basin Loss Parameters

Sub-Basin	Initial Loss I_a (inch)	Constant rate (in/hr)	% Impervious
Auxiliary Reservoir Surface	0	0	100
Main Reservoir Surface	0	0	100
Residual Land Surface	0	0	30
Sub-basin IV	0	0	30
Sub-basin V	0	0	30
Sub-basin VI	0	0	30
Sub-basin VII	0	0	30
Sub-basin VIII	0	0	30
Sub-basin IX	0	0	30

Note:

It was assumed that the watershed was in a saturated condition, (i.e., $I_a = 0$). Further, the hydrologic soil is classified as "HSG-C" based on soil group in the watershed as indicated by Appendix-A of TR-55 corresponding to watershed soil names Creedmoor, Mayodan, and White Store (Table 2.4.12-201) (Reference 2.4-227). Based on TR-55, the loss rates for soil group C range between 0.05 and 0.15 inches per hour (in/hr). To be on the conservative side, 0.0 in/hr was selected as the loss rate for all sub-basins.

HAR COL 2.4-2

**Table 2.4.3-219
Sub-Basin Areas**

Basin ID (Figure 2.4.3-201)	Area (mi.²)	Notes
Sub-Basins above the Main Dam		
Sub-basin IV	12.46	Land area
Sub-basin V	3.60	Land area
Sub-basin VI	3.38	Land area
Sub-basin VII	13.16	Land area
Sub-basin VIII	4.02	Land area
Sub-basin IX	1.14	Land area
Residual Land Surface	17.60	Land area around the Main Reservoir
Main Reservoir Surface	11.94	Water surface area
Auxiliary Reservoir		
Sub-basin X	2.47	Land area
Auxiliary Reservoir Surface	0.53	Water surface area
Total	70.29	

HAR COL 2.4-2

Table 2.4.3-220
Sub-Basin Unit Hydrograph Characteristics

Item	Sub-Basin IV	Sub-Basin V	Sub-Basin VI	Sub-Basin VII	Sub-Basin VIII	Sub-Basin IX	Sub-Basin X	Residual Area
A (mi. ²)	12.46	3.60	3.38	13.16	4.02	1.14	2.47	17.60
L (mi.)	5.61	3.22	2.93	5.25	2.98	1.14	2.45	9.07
L _c (mi.)	2.28	2.02	1.64	1.92	1.02	0.41	1.37	3.08
C _t	3.91	3.91	3.91	3.91	3.91	3.91	3.91	3.91
C _p	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
t _L	8.39	6.86	6.27	7.82	5.46	3.12	5.63	10.61 (1.7)
Q _p (cfs)	712	252	259	808	353	175	211	796 (4,992)

Notes:
 Aux. Res. = Auxiliary Reservoir surface
 Main Res. = Main Reservoir surface
 Residual Area = residual land surface
 (x) = alternate conservative parameters were used for the residual area

HAR COL 2.4-2

Table 2.4.3-221
1-Hour Unit Hydrograph Parameters

Parameter	Sub-basin - IV	Sub-basin - V	Sub-basin - VI	Sub-basin - VII	Sub-basin - VIII	Sub-basin - IX	Sub-basin - X	Residual Land Main Reservoir
Q _p (cfs)	750	264	271	847	369	181	220	4992
t _p (hr)	8.5	7.1	6.5	8	5.7	3.5	5.9	2.2
D/2 (hr)	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
t _L (hr)	8	6.6	6	7.5	5.2	3	5.4	1.7

Table 2.4.3-222
Comparison of Peak Flows determined using the USGS Equations and the HEC-HMS Model

Storm Return Period (year) (Col-1)	USGS Equation Based Peak Flow (cfs) (Col-2)	USGS Equation Prediction Error (%) (Col-3)	USGS Equation Based Peak Flow Corrected for Prediction Error (cfs) (Col-4)	FSAR HEC-HMS Model Based Peak Flow (cfs) (Col-5)	Peak Flow Over-prediction by FSAR HEC-HMS Model (%) (Col-6)
100	10,628	±47.00	15,624	22,488	44%
200	12,467	±48.90	18,564	24,271	31%
500	15,199	±51.60	23,042	31,329	36%

HAR COL 2.4-2

Table 2.4.3-223
1-Hour Unit Hydrograph Parameters with Peaking

Item	Sub-Basin IV	Sub-Basin V	Sub-Basin VI	Sub-Basin VII	Sub-Basin VIII	Sub-Basin IX	Sub-Basin X	Residual Area
Time to Peak, t_p (hr)	6.80	5.68	5.20	6.40	4.56	2.80	4.72	1.76
Peak Flow, Q_p (cfs)	937	330	339	1059	462	226	275	6240
Volume Check (in)	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Lag time, t_L (hr)	6.30	5.18	4.70	5.90	4.06	2.30	4.22	1.26

HAR COL 2.4-2

Table 2.4.3-224
Summary of PMF Inflow to Auxiliary and Main Reservoirs, With Peaking vs. Without Peaking

Metric	Auxiliary Reservoir		Main Reservoir	
	Without Peaking	Without Peaking	Without Peaking	With Peaking
Peak Inflow (cfs)	6,242	6,242	110,597	125,020
Peak Outflow (cfs)	5,581	5,581	20,677	21,050
Total Inflow (IN)	66.06	66.06	52.16	52.16
Total Outflow (IN)	66.06	66.06	48.45	48.53
Peak Storage (AF)	6,677	6,677	279,228	281,990
Peak Elevation (ft NGVD29)	256.26	256.26	251.99	252.25

HAR COL 2.4-2

Table 2.4.3-225
Selection of Critical Storm

Case	PMP Storm	Peak Inflow (cfs)	Total Inflow (ac-ft)
Case-1	Using a single PMP storm that corresponds to the Main Reservoir drainage basin (25% Peaking)	124,380	193,664
Case-2	Using two different PMP storms: PMP storm for the Main Reservoir drainage basin and a more severe PMP storm for the drainage basin above the Auxiliary Dam (25% Peaking)	125,065	195,572

HAR COL 2.4-2

Table 2.4.3-226 (Sheet 1 of 2)
HEC-RAS Computed Maximum Water Surface Profile in the Main Reservoirs

Reach			Q Total	Min Ch El	W.S. Elev	E.G. Elev
	River Sta	Profile	(cfs)	(ft. NGVD29)	(ft. NGVD29)	(ft. NGVD29)
Tom Jack Ck	10000	Max WS	985.08	218.27	252.76	252.76
Tom Jack Ck	8800	Max WS	981.12	203	252.76	252.76
Tom Jack Ck	7700	Max WS	979.26	198	252.76	252.76
Tom Jack Ck	6600	Max WS	971.61	194	252.76	252.76
Tom Jack Ck	4500	Max WS	948.25	192.41	252.76	252.76
Tom Jack Ck	2661.272	Max WS	894.57	189	252.76	252.76
LittleWhiteOak-C	17000	Max WS	326.64	208	252.76	252.76
LittleWhiteOak-C	14800	Max WS	322.6	198	252.76	252.76
LittleWhiteOak-C	12600	Max WS	322.6	200.81	252.76	252.76
LittleWhiteOak-C	9800	Max WS	309.23	202	252.76	252.76
LittleWhiteOak-C	7200	Max WS	300.92	199.49	252.76	252.76
Thomas Ck	11242.48	Max WS	67.3	225.71	252.76	252.76
Thomas Ck	9991.82	Max WS	65.26	222.27	252.76	252.76
Thomas Ck	8539.703	Max WS	71.28	220.09	252.76	252.76
Thomas Ck	6905.036	Max WS	44.39	220	252.76	252.76
Thomas Ck	5660.595	Max WS	78.76	208	252.76	252.76
Thomas Ck	4400	Max WS	73.05	198	252.76	252.76
Thomas Ck	3500	Max WS	71.26	200.81	252.76	252.76
Thomas Ck	2600	Max WS	70.9	202	252.76	252.76
Thomas Ck	2205.884	Max WS	69.28	199.49	252.76	252.76
Thomas -White CK	5118.695	Max WS	370.2	194	252.76	252.76
Thomas -White CK	4600	Max WS	363.69	195.72	252.76	252.76
Thomas -White CK	3578.48	Max WS	352.64	196	252.76	252.76
Thomas -White CK	2732.033	Max WS	373.45	192.24	252.76	252.76
White Oak	41200	Max WS	8385.01	232	252.76	252.78
White Oak	38801.01	Max WS	8369.43	224	252.75	252.76
White Oak	35800	Max WS	8343.22	222	252.75	252.76
White Oak	32600	Max WS	8276.31	215.75	252.76	252.76
White Oak	29600	Max WS	8266.04	206	252.76	252.76
White Oak	26400	Max WS	8248.99	195.39	252.76	252.76
White Oak	22600	Max WS	8229.78	192.56	252.76	252.76
Main Res Reach-1	19200	Max WS	8603.24	185	252.76	252.76
Main Res Reach-1	16800	Max WS	8616.16	184.77	252.76	252.76
Main Res Reach-1	14200	Max WS	17151.38	179.26	252.76	252.76
Main Res Reach-2	12800	Max WS	18045.95	179.21	252.76	252.76
Main Res Reach-2	11600	Max WS	18084.99	176.05	252.76	252.76
A-3	5000	Max WS	2	198.72	252.76	252.76
A-3	3800	Max WS	1.18	195.78	252.76	252.76

HAR COL 2.4-2

Table 2.4.3-226 (Sheet 2 of 2)
HEC-RAS Computed Maximum Water Surface Profile in the Main Reservoirs

Reach			Q Total	Min Ch El	W.S. Elev	E.G. Elev
	River Sta	Profile	(cfs)	(ft. NGVD29)	(ft. NGVD29)	(ft. NGVD29)
A-3	3289.593	Max WS	0.94	186.35	252.76	252.76
A-3	2778.718	Max WS	-0.29	188	252.76	252.76
Main Res Reach-3	9800	Max WS	18084.7	170	252.76	252.76
Main Res Reach-3	7800	Max WS	18114.94	175	252.76	252.76
Buckhorn Ck-1	22200	Max WS	3850.57	198	252.76	252.76
Buckhorn Ck-1	20600	Max WS	3854.61	200.81	252.76	252.76
Buckhorn Ck-1	19400	Max WS	3860.89	202	252.76	252.76
Buckhorn Ck-1	17600	Max WS	3854.98	199.49	252.76	252.76
Buckhorn Ck-1	16000	Max WS	3860.54	194	252.76	252.76
Cary Ck	10400	Max WS	268.22	198	252.76	252.76
Cary Ck	8100	Max WS	266.49	200.81	252.76	252.76
Cary Ck	5800	Max WS	257.36	202	252.76	252.76
Cary Ck	4100	Max WS	254.49	199.49	252.76	252.76
Cary Ck	2600	Max WS	247.17	194	252.76	252.76
Buckhorn Ck -2	13200	Max WS	4107.71	195.72	252.76	252.76
Buckhorn Ck -2	9015.458	Max WS	4092	196	252.76	252.76
Buckhorn Ck -2	7400	Max WS	4086.07	192.24	252.76	252.76
Buckhorn Ck -2	5800	Max WS	4087.61	179.26	252.76	252.76
Buckhorn Ck -2	4200	Max WS	4100.11	179.21	252.76	252.76
Buckhorn Ck -2	3070.306	Max WS	4091.5	176.05	252.76	252.76
A-2	5200	Max WS	2	207	252.76	252.76
A-2	4400	Max WS	1.05	200	252.76	252.76
A-2	3200	Max WS	-0.04	190.49	252.76	252.76
A-2	2600	Max WS	2.66	183	252.76	252.76
A-2	2089.779	Max WS	1.97	179	252.76	252.76
Main Res Reach-4	5900	Max WS	18116.91	170	252.76	252.76
Main Res Reach-4	5400	Max WS	18122.03	170	252.76	252.76
Main Res Reach-5	3800	Max WS	22213.52	169	252.76	252.76
Main Res Reach-5	2200	Max WS	22211.21	167.67	252.75	252.76
Main Res Reach-5	1000	Max WS	22211.14	213.34	252.75	252.76
Main Res Reach-5	900		Inl Struct			
Main Res Reach-5	800	Max WS	22211.07	165	167.72	168.65
Main Res Reach-5	600	Max WS	22211.15	162.5	164.34	166.66
Main Res Reach-5	400	Max WS	22211.04	160	162.06	163.88

HAR COL 2.4-2

Table 2.4.3-227
Maximum PMF Stillwater Elevation in the Auxiliary and Main Reservoirs

Water Surface Elevation (ft. NGVD29) at which the Tainter Gate is Opened	HEC-HMS Results		HEC-RAS Results	Max of HEC-HMS and -RAS
	PMF Elevation AUX Reservoir (ft. NGVD29)	PMF Elevation Main Reservoir (ft. NGVD29)	PMF Elevation Main Reservoir (ft. NGVD29)	Selected PMF Elevation Main Reservoir (ft. NGVD29)
242	256.56	252.25	252.76	252.76

HAR COL 2.4-2

Table 2.4.3-228
Reservoir Bottom Elevation

Location	Line ID	Lake Bottom Elevation (ft. NGVD29)
HAR-2	1	220
	2	220
	3	240
	4	240
	5	240
HAR-3	1	220
	2	220
	3	220
	4	240
	5	240
	6	240
	7	240
HNP	1	240
	2	220
	3	220
Main DAM	1	220
Main DAM	2	220
Auxiliary DAM	3	240
Auxiliary DAM	4	240

HAR COL 2.4-2

Table 2.4.3-229
Fetch Distances for HAR 2

Id	Fetch Dist (mile)
1	0.93*
2	0.88
3	0.87
4	0.40
5	0.50
6	0.50
7	0.50

*Critical Fetch Distance = 0.93 mi

HAR COL 2.4-2

Table 2.4.3-230
Fetch Distances for HAR 3

Id	Fetch Dist (mile)
1	0.85*
2	0.72
3	0.61
4	0.64
5	0.55

*Critical Fetch Distance = 0.85 mi

HAR COL 2.4-2

Table 2.4.3-231
Fetch Distances for HNP

Line ID	Fetch Dist (mi)
1	0.76
2	4.33*
3	2.73

*Critical Fetch Distance = 4.33 mi

HAR COL 2.4-2

Table 2.4.3-232
Fetch Distances for Auxiliary and Main Dams

Line ID	Fetch Dist (mi)
1	4.29*
2	4.29*
3	1.17
4	1.08

*Critical Fetch Distance = 4.29 mi

HAR COL 2.4-2

Table 2.4.3-233
Correction for Wind Averaging Interval

Location	Line ID	St. Line Fetch, X (mi)	St. Line Fetch, X (km)	T(X,U) (s)	Correction Factor	Wind Speed Ut (m/s)
HAR 2	1	0.85	1.36	1592	1.02	22.56
	2	0.72	1.16	1431	1.02	22.63
	3	0.61	0.98	1280	1.02	22.70
	4	0.64	1.02	1314	1.02	22.68
	5	0.55	0.88	1192	1.02	22.74
HAR 3	1	0.93	1.49	1693	1.01	22.53
	2	0.88	1.41	1632	1.01	22.55
	3	0.87	1.39	1616	1.01	22.55
	4	0.40	0.80	1116	1.03	22.79
	5	0.50	0.80	1118	1.03	22.79
	6	0.50	0.80	1119	1.03	22.79
	7	0.50	0.80	1116	1.03	22.79
HNP	1	0.76	1.22	1482	1.02	22.60
	2	4.33	6.92	4738	0.98	21.82
	3	2.73	4.36	3476	1.00	22.23
DAMs	1	4.29	6.87	4713	0.98	21.83
	2	4.29	6.87	4713	0.98	21.83
	3	1.17	1.88	1975	1.01	22.45
	4	1.08	1.73	1871	1.01	22.48

HAR COL 2.4-2

Table 2.4.3-234
Coefficients in Equations (11a, 11b, and 11c) for Runup of irregular Head-On Waves in Impermeable and Permeable Rock Armored Slopes

Percent	A	B	C	D
0.1	1.12	1.34	0.55	2.58
2	0.96	1.17	0.46	1.97
1	1.04	1.26	0.51	2.29
Significant	0.72	0.88	0.41	1.35

HAR COL 2.4-2

Table 2.4.3-235
Wave Runup Computation at HAR2, HAR3, HNP, and Auxiliary and Main Dams

Reservoir		PMF Elevation (ft. NGVD29)	Bottom Elevation (ft. NGVD29)		Water Depth (m)		Limiting Wave Period (sec)			
Auxiliary		256.56	240.00		5.05		7.02			
Main		252.76	220.00		9.98		9.87			
Location	Line ID	Wind Velocity (m/s)	St. Line Fetch, X (km)	Hm0 (m)	Predicted Peak Wave Period, Tp (sec)	Slope = tan(alpha)	Deepwater Wave Steepness, s0	Iribarren Number	Significant Runup, Rus (ft)	Max Runup (0.1%), Ru (ft)
HAR 2	1	22.56	1.36	0.48	1.80	0.09	0.09	0.28	0.32	0.50
	2	22.63	1.16	0.44	1.71	0.09	0.10	0.28	0.29	0.45
	3	22.70	0.98	0.41	1.62	0.13	0.10	0.40	0.39	0.60
	4	22.68	1.02	0.42	1.64	0.13	0.10	0.40	0.39	0.61
	5	22.74	0.88	0.39	1.57	0.13	0.10	0.40	0.36	0.56
HAR 3	1	22.53	1.49	0.50	1.86	0.09	0.09	0.29	0.34	0.52
	2	22.55	1.41	0.49	1.83	0.09	0.09	0.28	0.33	0.51
	3	22.55	1.39	0.48	1.82	0.09	0.09	0.28	0.32	0.50
	4	22.91	0.63	0.33	1.41	0.13	0.11	0.38	0.30	0.47
	5	22.79	0.80	0.37	1.52	0.13	0.10	0.39	0.34	0.54
	6	22.79	0.80	0.37	1.52	0.13	0.10	0.39	0.34	0.54
	7	22.79	0.80	0.37	1.52	0.13	0.10	0.39	0.34	0.53
HNP	1	22.60	1.22	0.45	1.74	0.20	0.10	0.65	0.69	1.08
	2	21.82	6.92	1.03	3.06	0.10	0.07	0.38	0.92	1.43
	3	22.23	4.36	0.84	2.64	0.10	0.08	0.36	0.71	1.11
DAMs	1	21.83	6.87	1.03	3.05	0.50	0.07	1.88	3.85	6.41
	2	21.83	6.87	1.03	3.05	0.40	0.07	1.50	3.52	5.67
	3	22.45	1.88	0.56	2.00	0.40	0.09	1.34	1.77	2.75
	4	22.48	1.73	0.54	1.95	0.40	0.09	1.33	1.69	2.62

HAR COL 2.4-2

Table 2.4.3-236
Wave Setup Computation at HAR2, HAR3, HNP, and Auxiliary and Main Dams

Location	Line ID	Wind Velocity (mph)	St. Line Fetch, X (mi)	Depth (ft)	Setup (ft)
HAR 2	1	50.77	0.85	32.76	0.05
	2	50.91	0.72	32.76	0.04
	3	51.06	0.61	16.56	0.07
	4	51.03	0.64	16.56	0.07
	5	51.17	0.55	16.56	0.06
HAR 3	1	50.69	0.93	32.76	0.05
	2	50.74	0.88	32.76	0.05
	3	50.75	0.87	32.76	0.05
	4	51.54	0.40	16.56	0.05
	5	51.27	0.50	16.56	0.06
	6	51.27	0.50	16.56	0.06
	7	51.27	0.50	16.56	0.06
HNP	1	50.86	0.76	16.56	0.09
	2	49.10	4.33	32.76	0.23
	3	50.01	2.73	32.76	0.15
DAMs	1	49.12	4.29	32.76	0.23
	2	49.12	4.29	32.76	0.23
	3	50.51	1.17	16.56	0.13
	4	50.57	1.08	16.56	0.12

HAR COL 2.4-2

Table 2.4.3-237
Overall PMF Elevation at HAR2, HAR3, HNP, and Auxiliary and Main Dams

Location	Line ID	Stillwater EL. (ft. NGVD29)	Max Runup (0.1%), Ru (ft)	Setup (ft)	Overall PMF (ft. NGVD29)
HAR 2	1	252.76	0.50	0.05	253.31
	2	252.76	0.45	0.04	253.26
	3	256.56	0.60	0.07	257.23
	4	256.56	0.61	0.07	257.24
	5	256.56	0.56	0.06	257.19
HAR 3	1	252.76	0.52	0.05	253.34
	2	252.76	0.51	0.05	253.32
	3	252.76	0.50	0.05	253.31
	4	256.56	0.47	0.05	257.07
	5	256.56	0.54	0.06	257.15
	6	256.56	0.54	0.06	257.15
	7	256.56	0.53	0.06	257.15
HNP	1	256.56	1.08	0.09	257.72
	2	252.76	1.43	0.23	254.42
	3	252.76	1.11	0.15	254.02
DAMs	1	252.76	6.41	0.23	259.39
	2	252.76	5.67	0.23	258.65
	3	256.56	2.75	0.13	259.44
	4	256.56	2.62	0.12	259.30

Notes:
252.76 ft and 256.56 ft are the maximum stillwater PMF elevations for the Main and Auxiliary reservoirs.

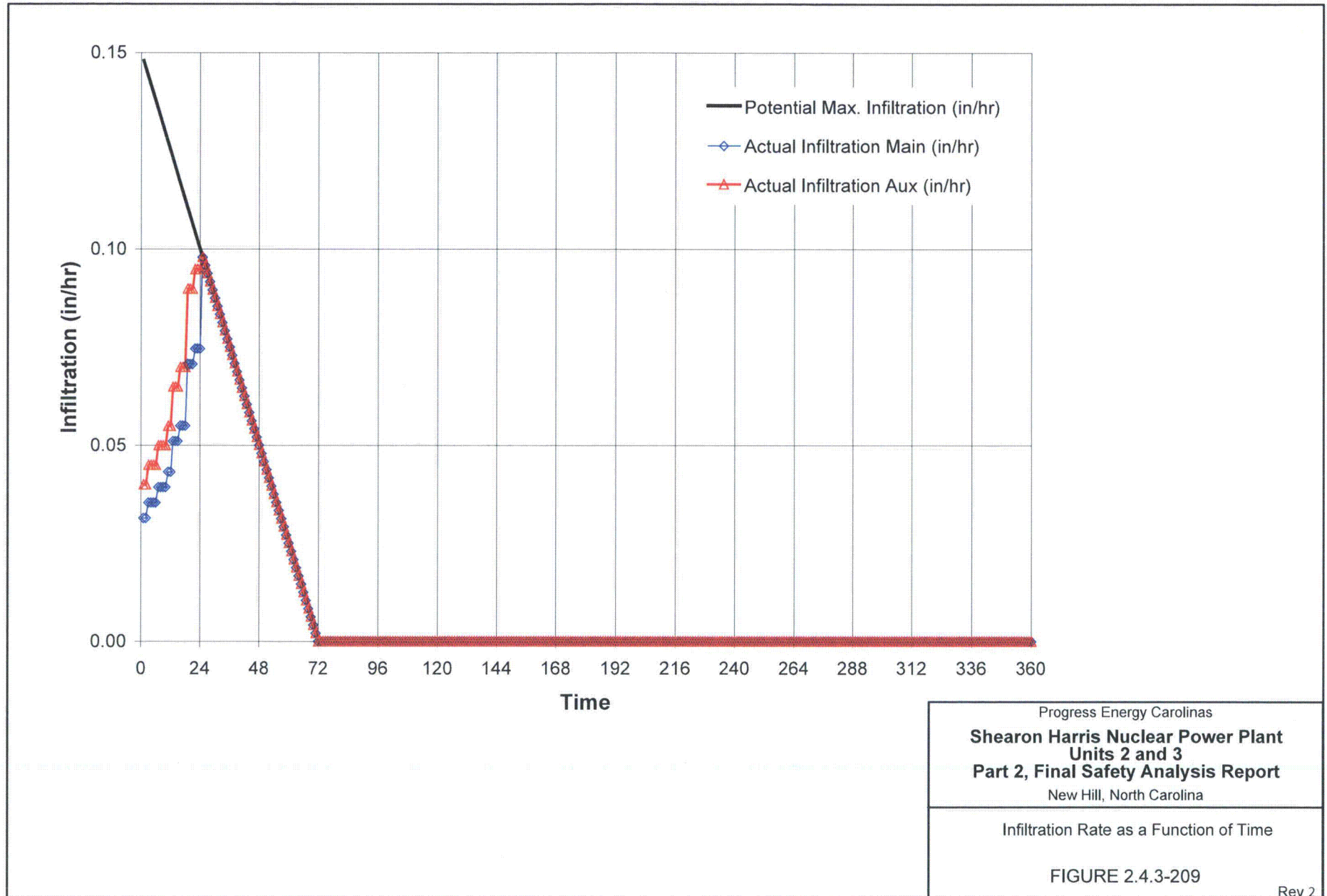
HAR COL 2.4-2

Table 2.4.3-238
Maximum PMF Elevation at HAR2, HAR3, HNP, and Auxiliary and Main Dams

Location	Maximum PMF Elevation (ft. NGVD29)
HAR-2	257.24
HAR-3	257.15
HNP	257.72
DAMs	259.44

Attachments/Enclosures:

Revised Figure 2.4.3-209 (1 page)
Revised Figure 2.4.3-210 (1 page)
Revised Figure 2.4.3-211 (1 page)
Revised Figure 2.4.3-212 (1 page)
Revised Figure 2.4.3-213 (1 page)
Revised Figure 2.4.3-214 (1 page)
Revised Figure 2.4.3-215 (1 page)
Revised Figure 2.4.3-216 (1 page)
Revised Figure 2.4.3-217 (1 page)
Revised Figure 2.4.3-218 (1 page)
Revised Figure 2.4.3-219 (1 page)
Revised Figure 2.4.3-220 (1 page)
Revised Figure 2.4.3-221 (1 page)
Revised Figure 2.4.3-222 (1 page)
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New Figure 2.4.3-226 (1 page)
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New Figure 2.4.3-228 (1 page)
New Figure 2.4.3-229 (1 page)
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New Figure 2.4.3-231 (1 page)
New Figure 2.4.3-232 (1 page)
New Figure 2.4.3-233 (1 page)
New Figure 2.4.3-234 (1 page)
New Figure 2.4.3-235 (1 page)
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New Figure 2.4.3-237 (1 page)

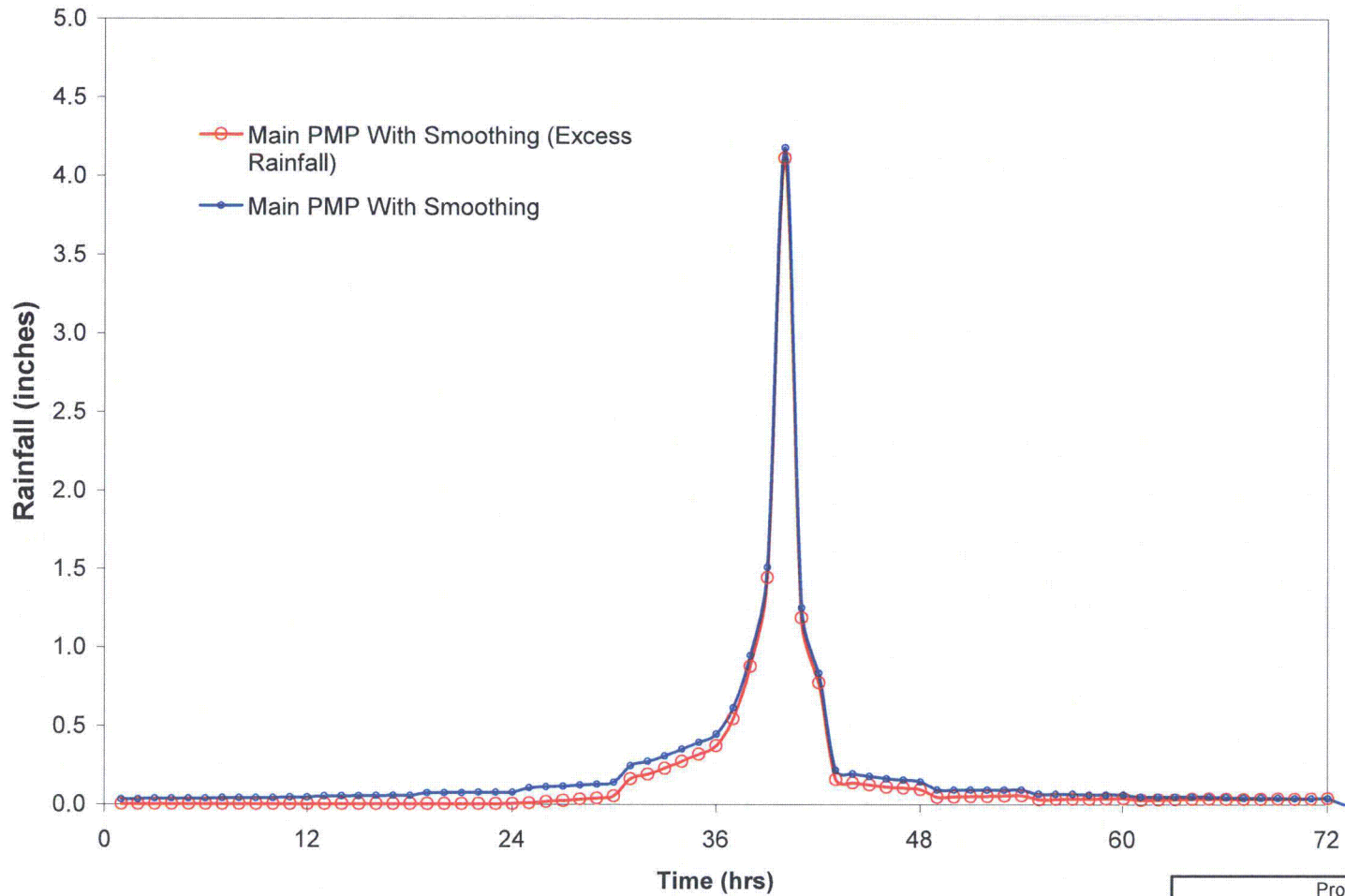


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Infiltration Rate as a Function of Time

FIGURE 2.4.3-209

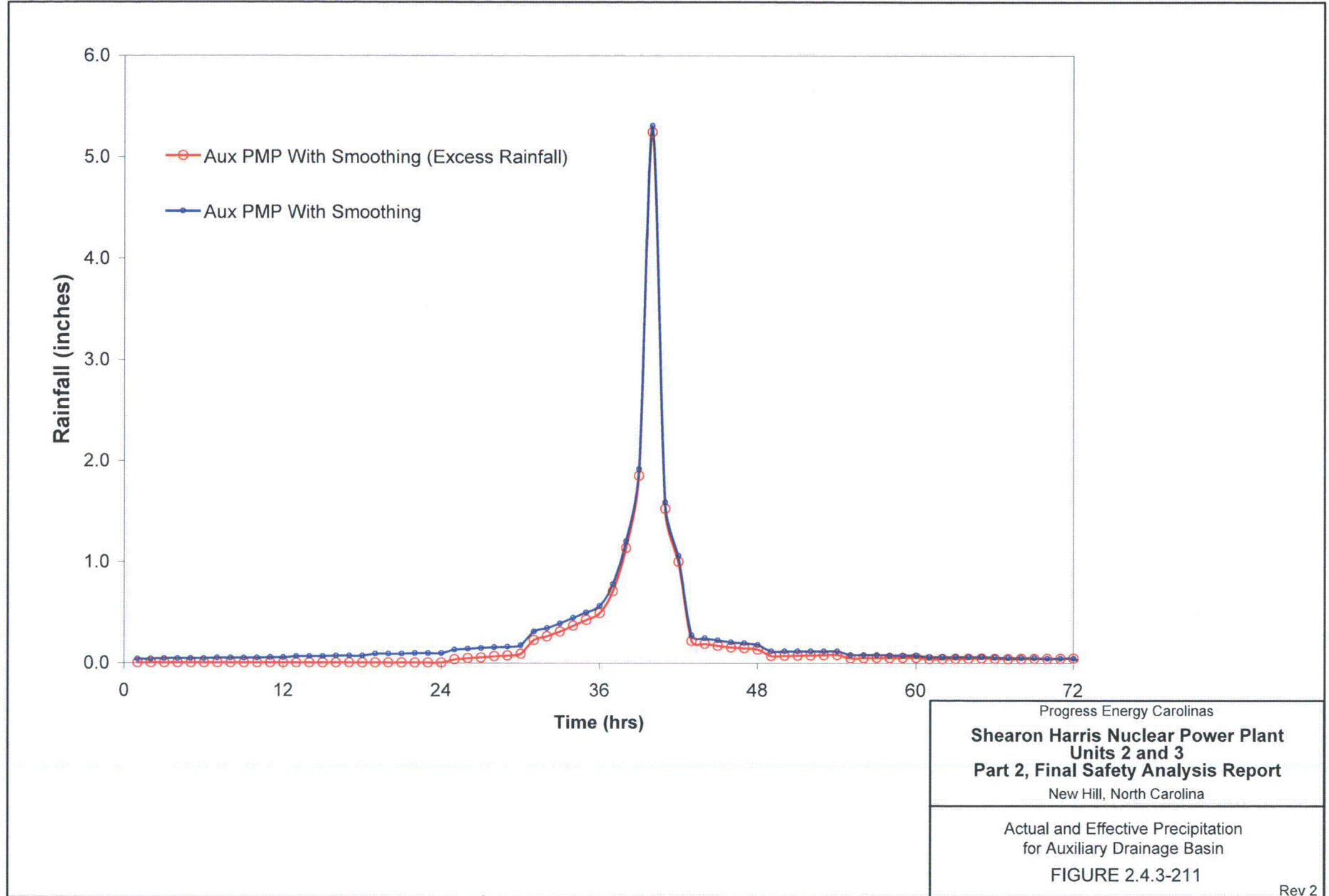
Rev 2

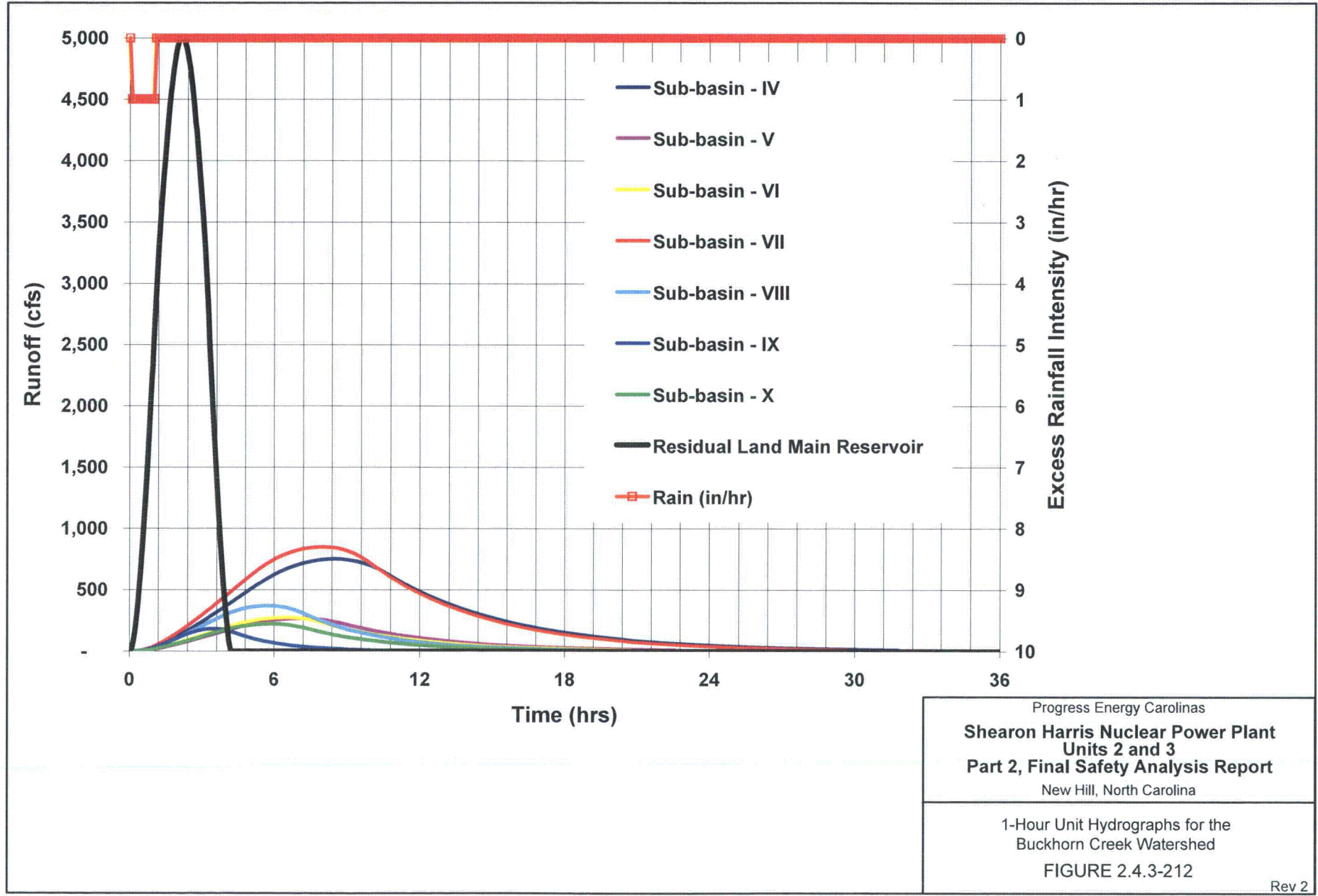


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Actual and Effective Precipitation
for Main Drainage Basin
FIGURE 2.4.3-210

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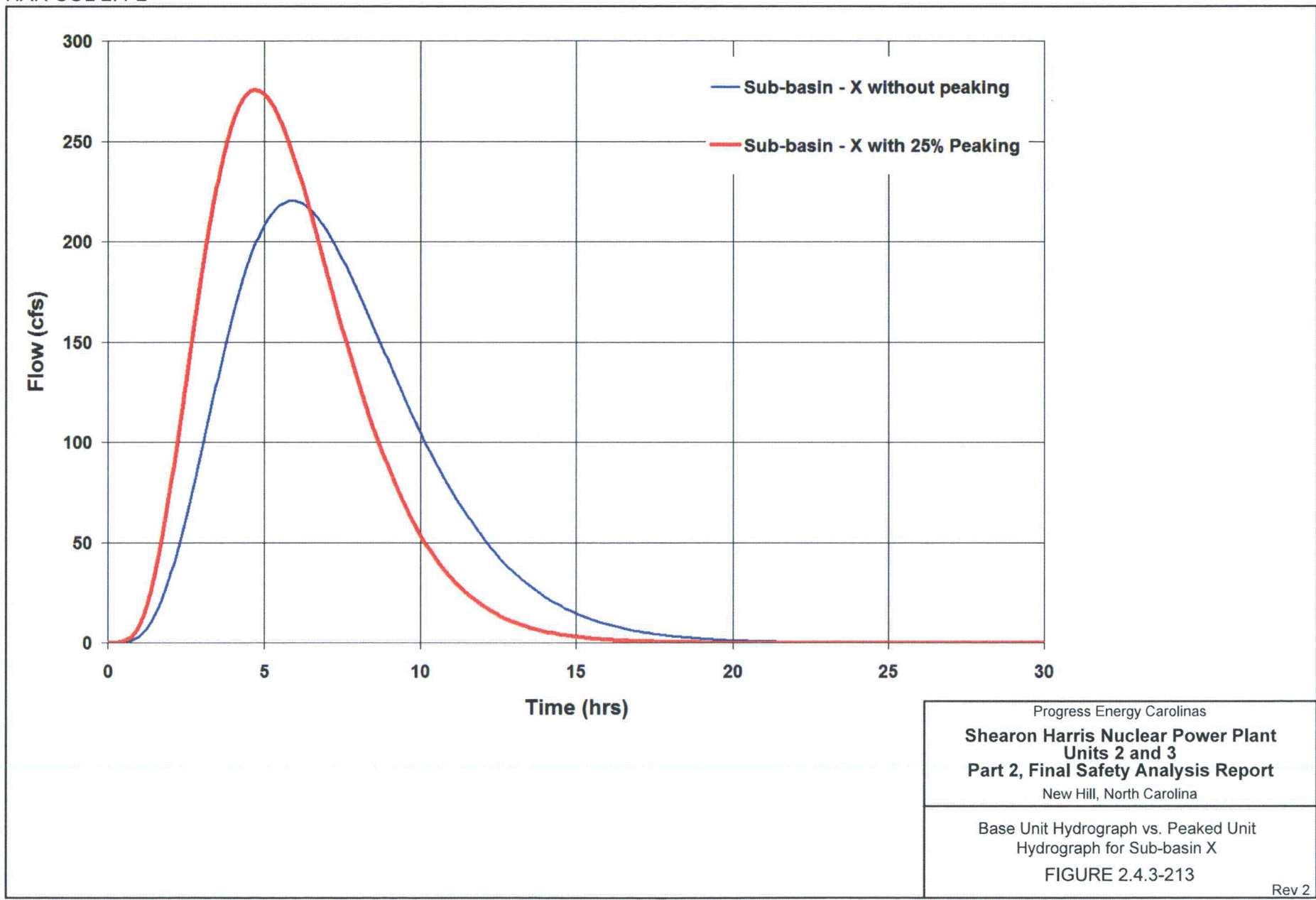


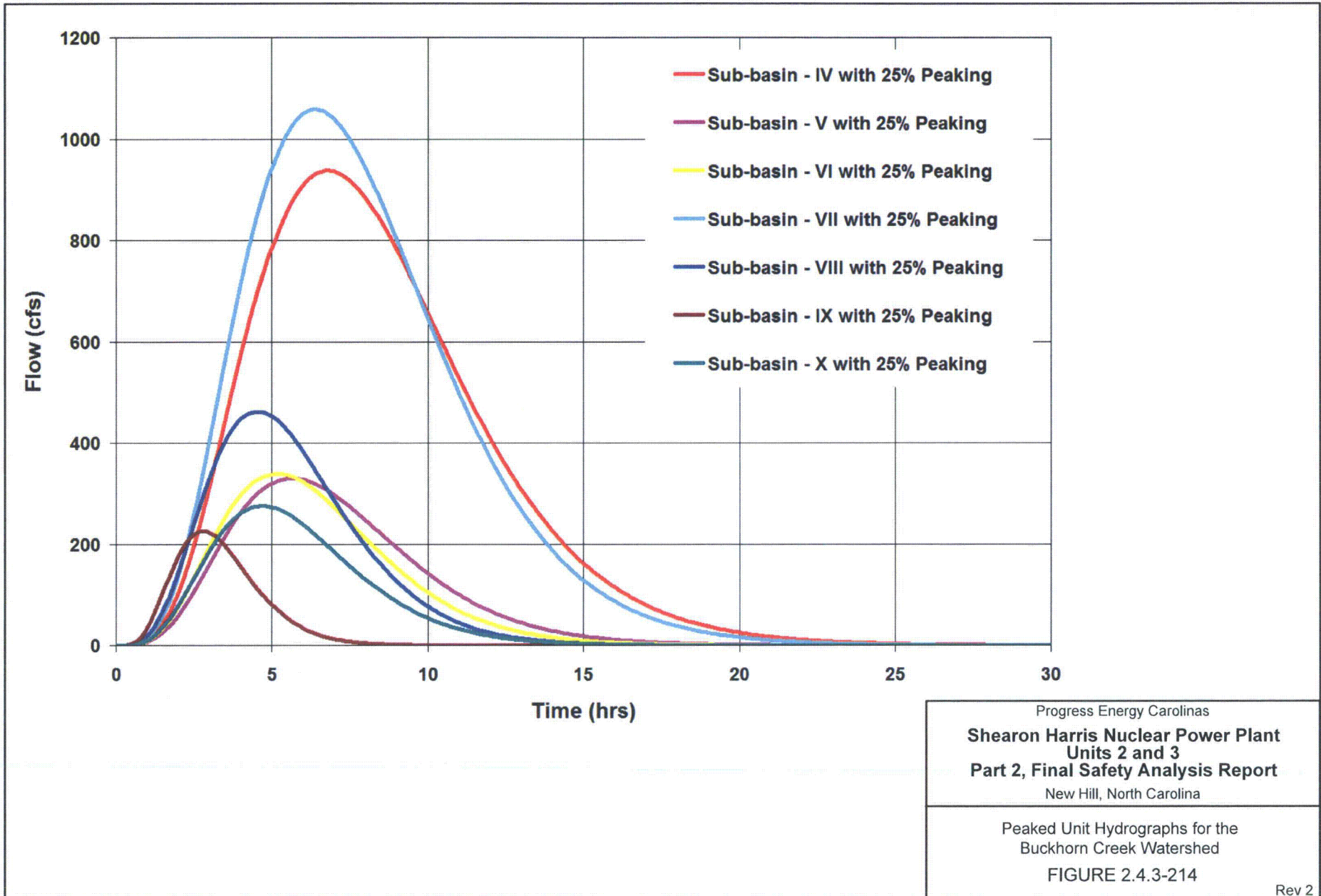


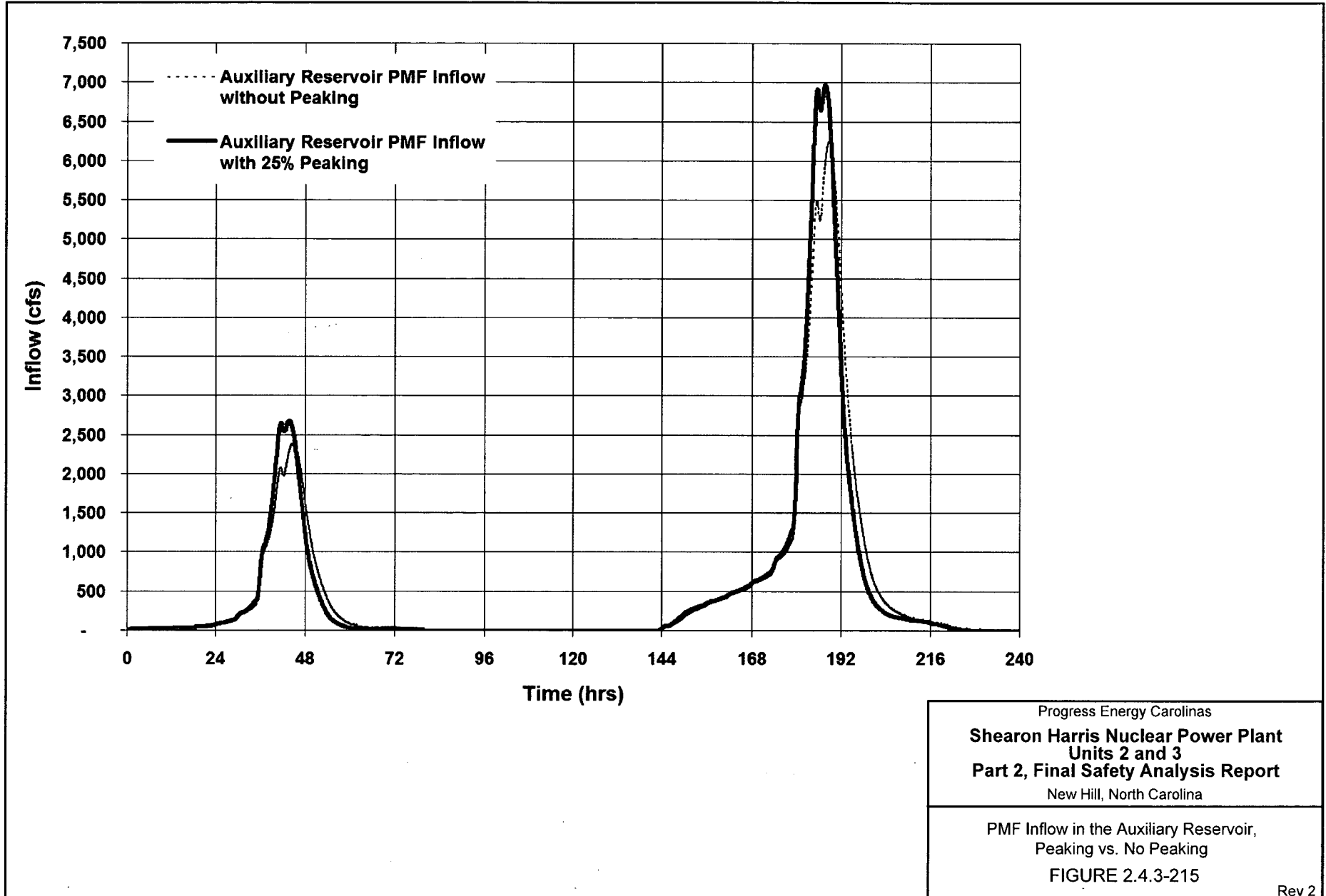
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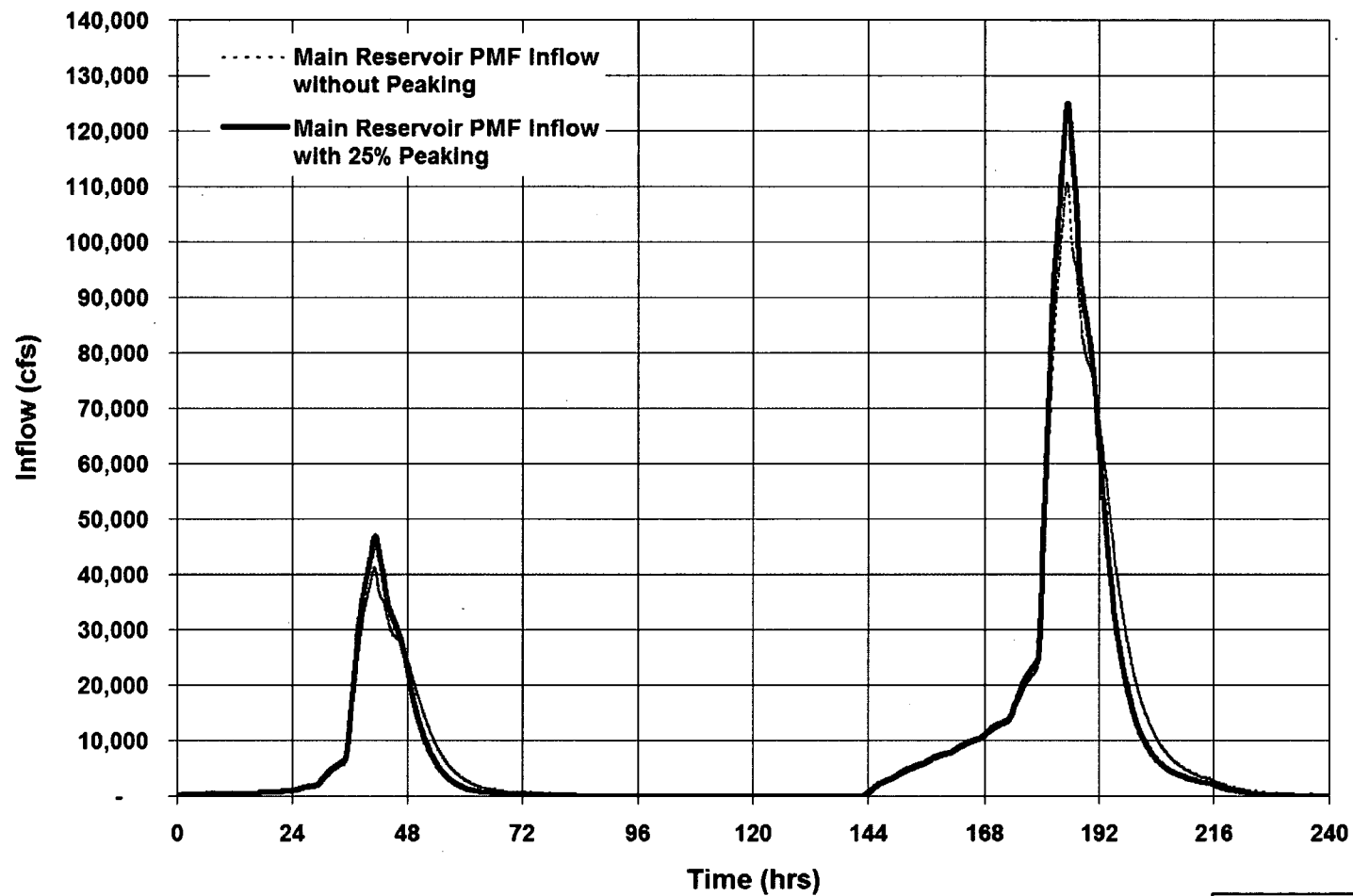
1-Hour Unit Hydrographs for the
 Buckhorn Creek Watershed
FIGURE 2.4.3-212

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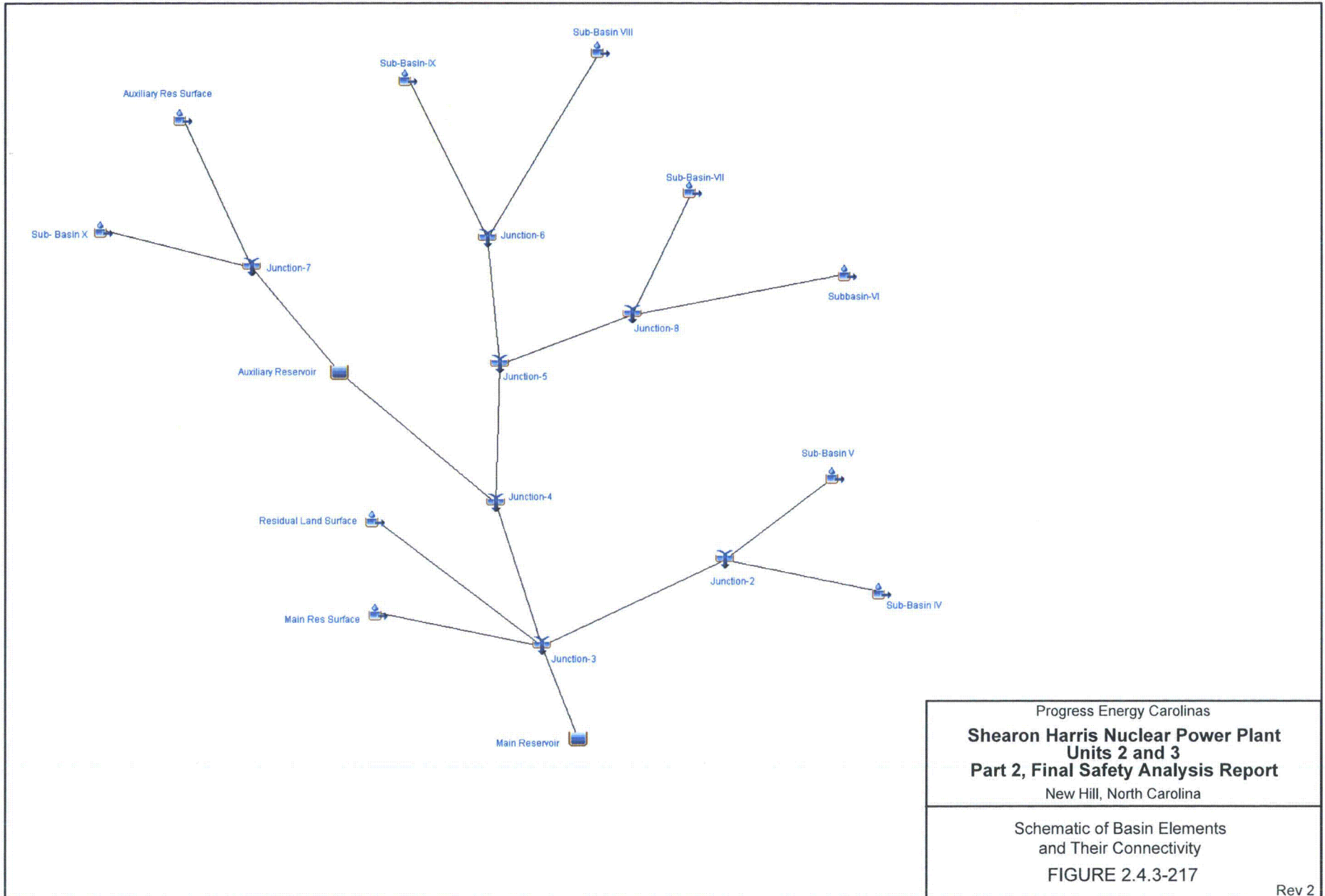






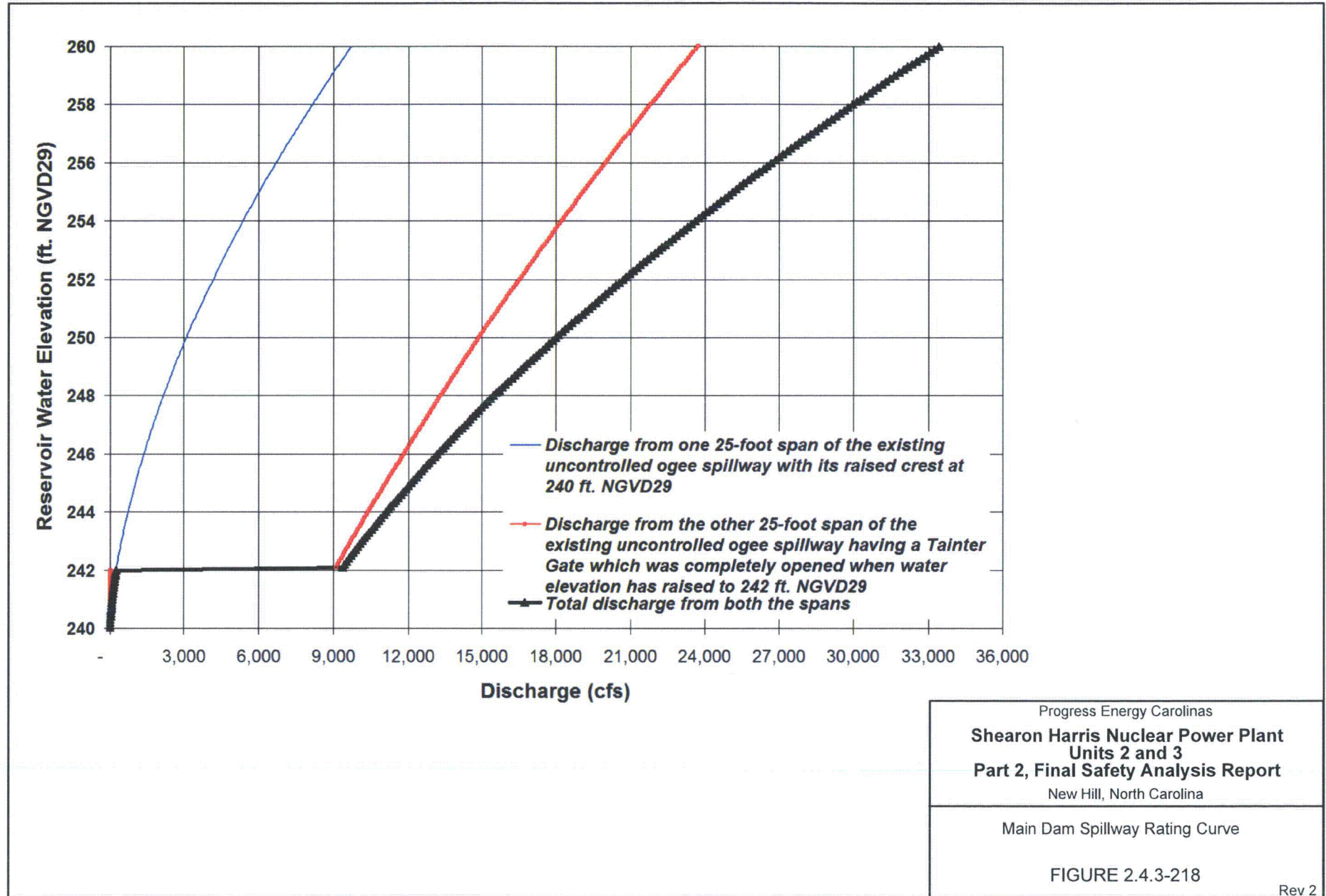
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PMF Inflow in the Main Reservoir,
Peaking vs. No Peaking
FIGURE 2.4.3-216



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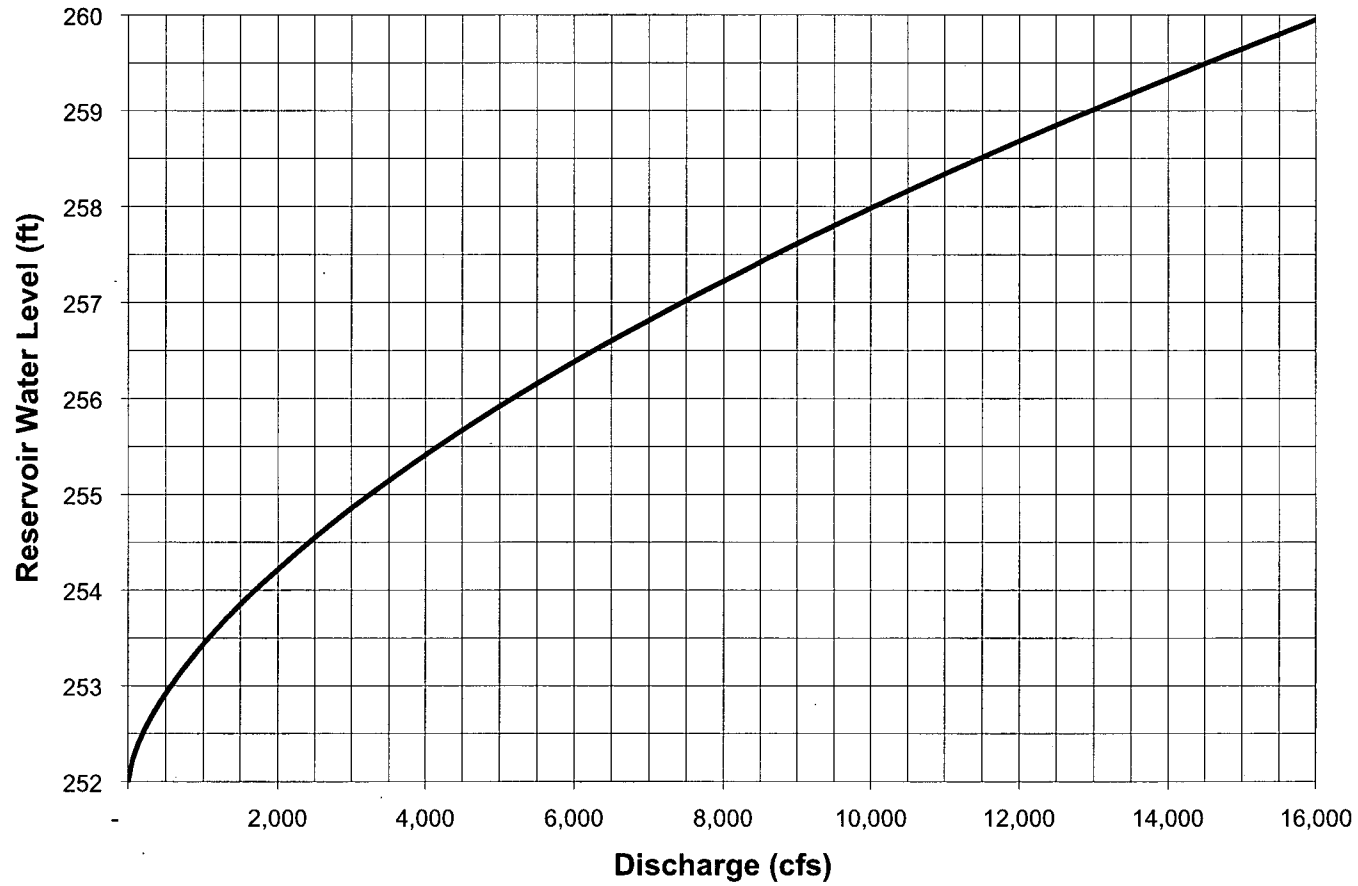
Schematic of Basin Elements
and Their Connectivity
FIGURE 2.4.3-217
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Main Dam Spillway Rating Curve

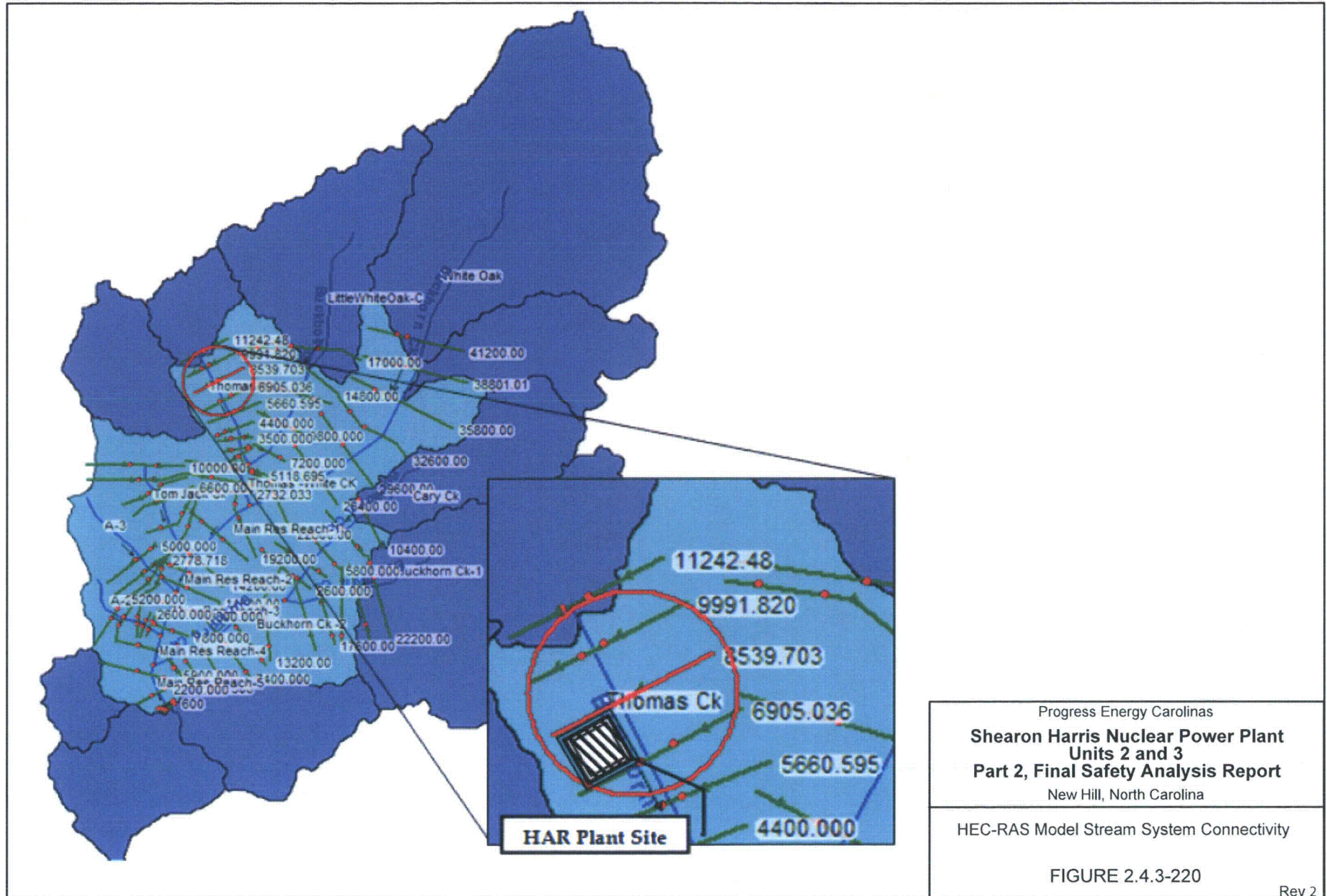
FIGURE 2.4.3-218



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Auxiliary Dam Spillway Rating Curve

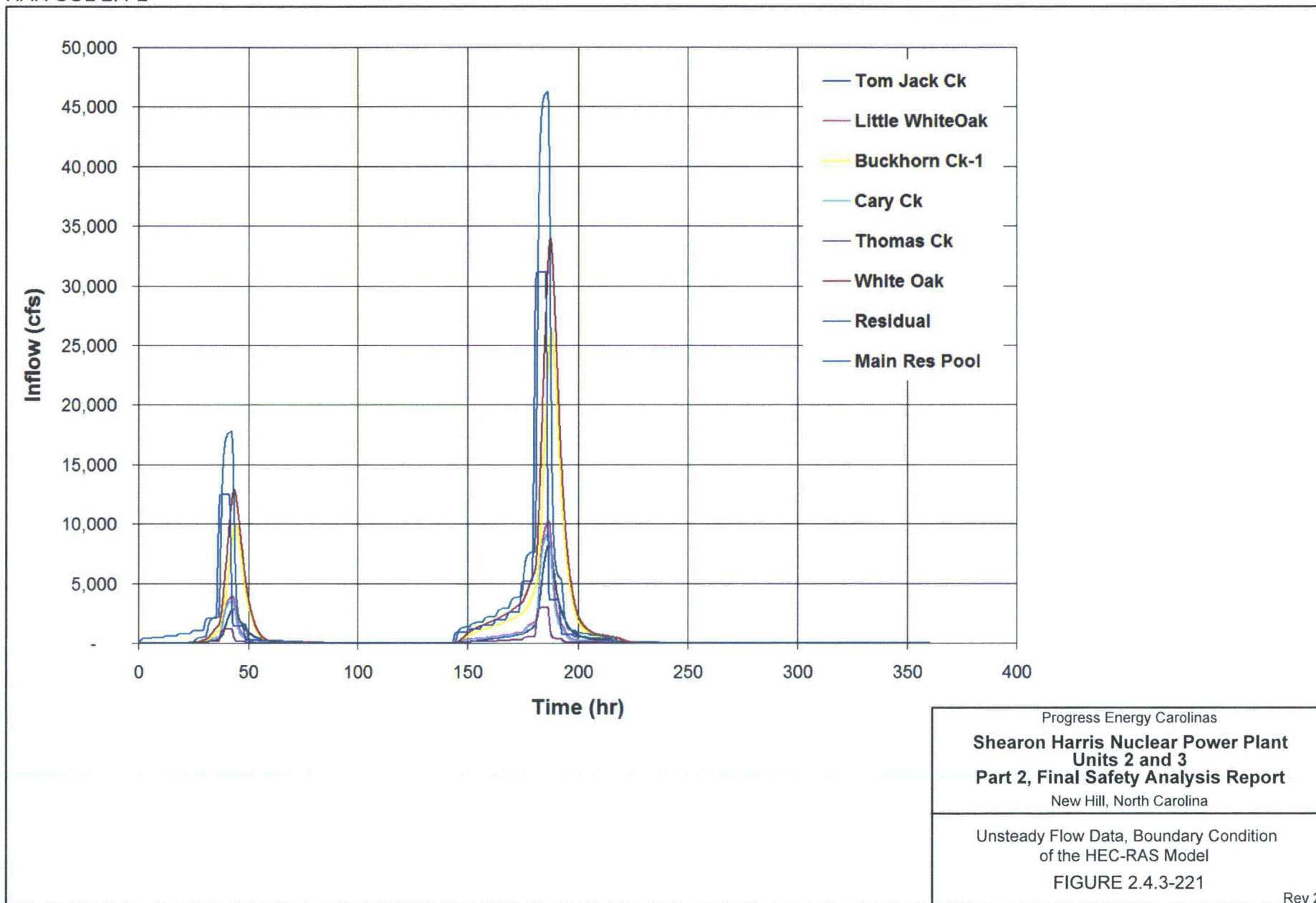
FIGURE 2.4.3-219

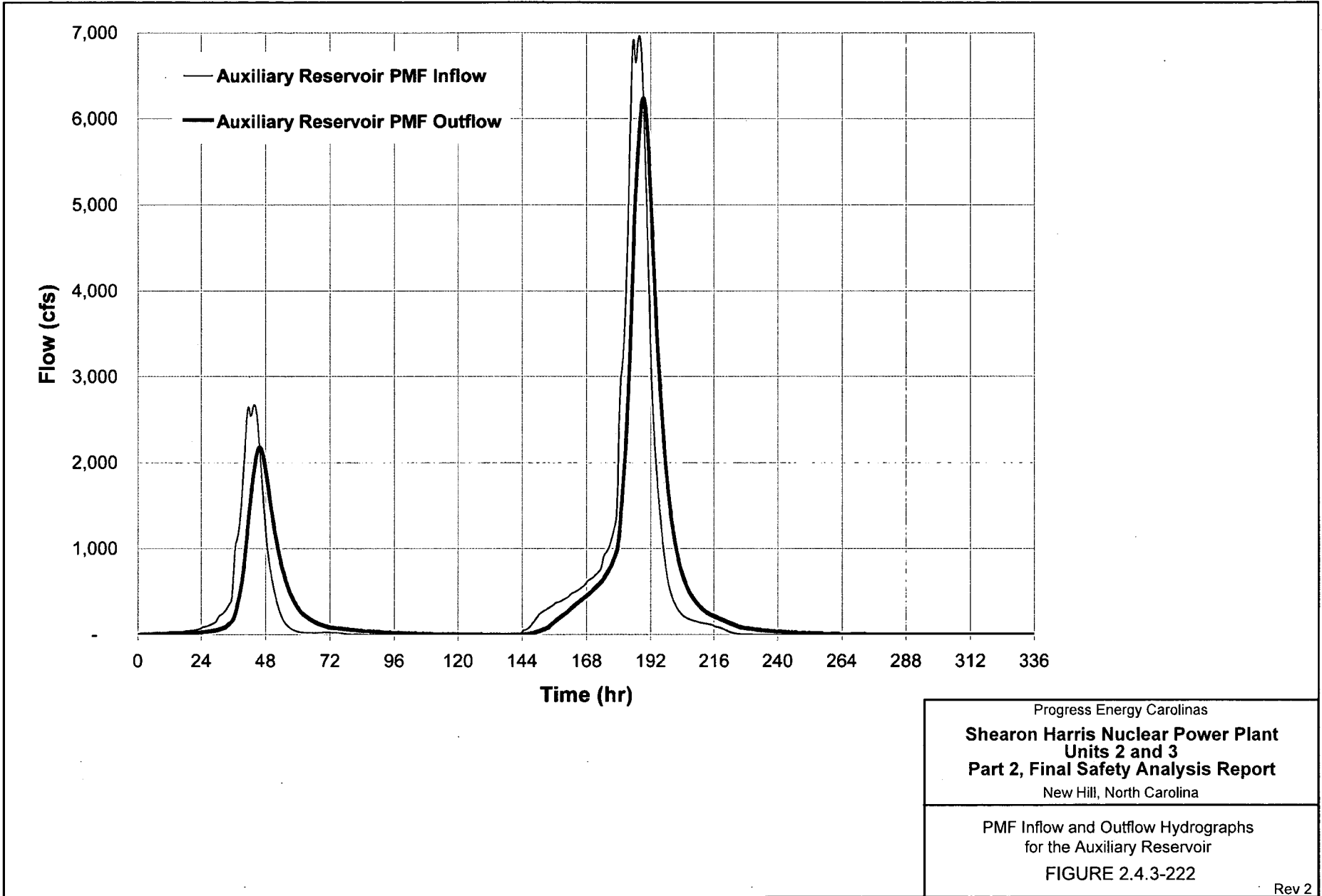


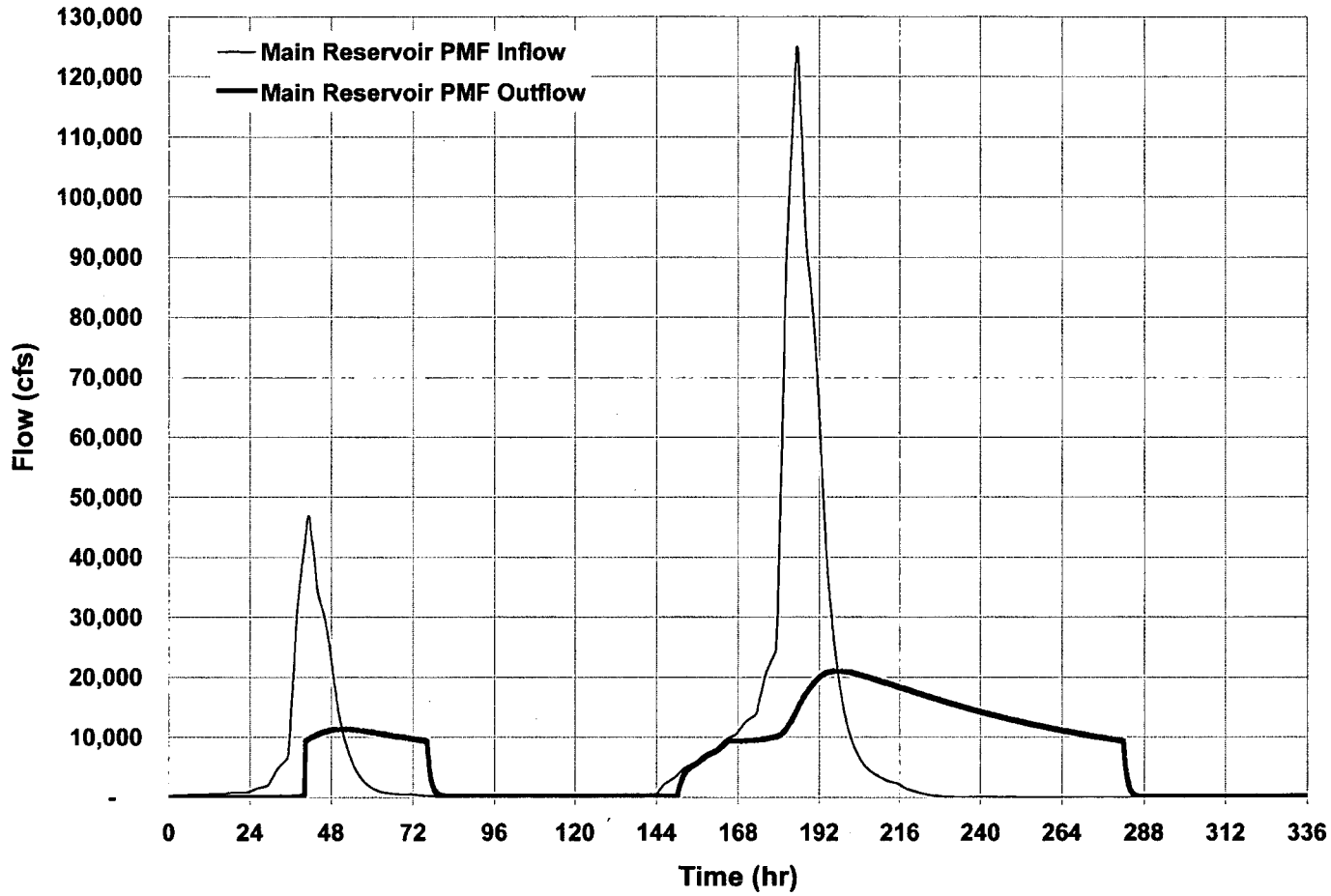
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HEC-RAS Model Stream System Connectivity

FIGURE 2.4.3-220

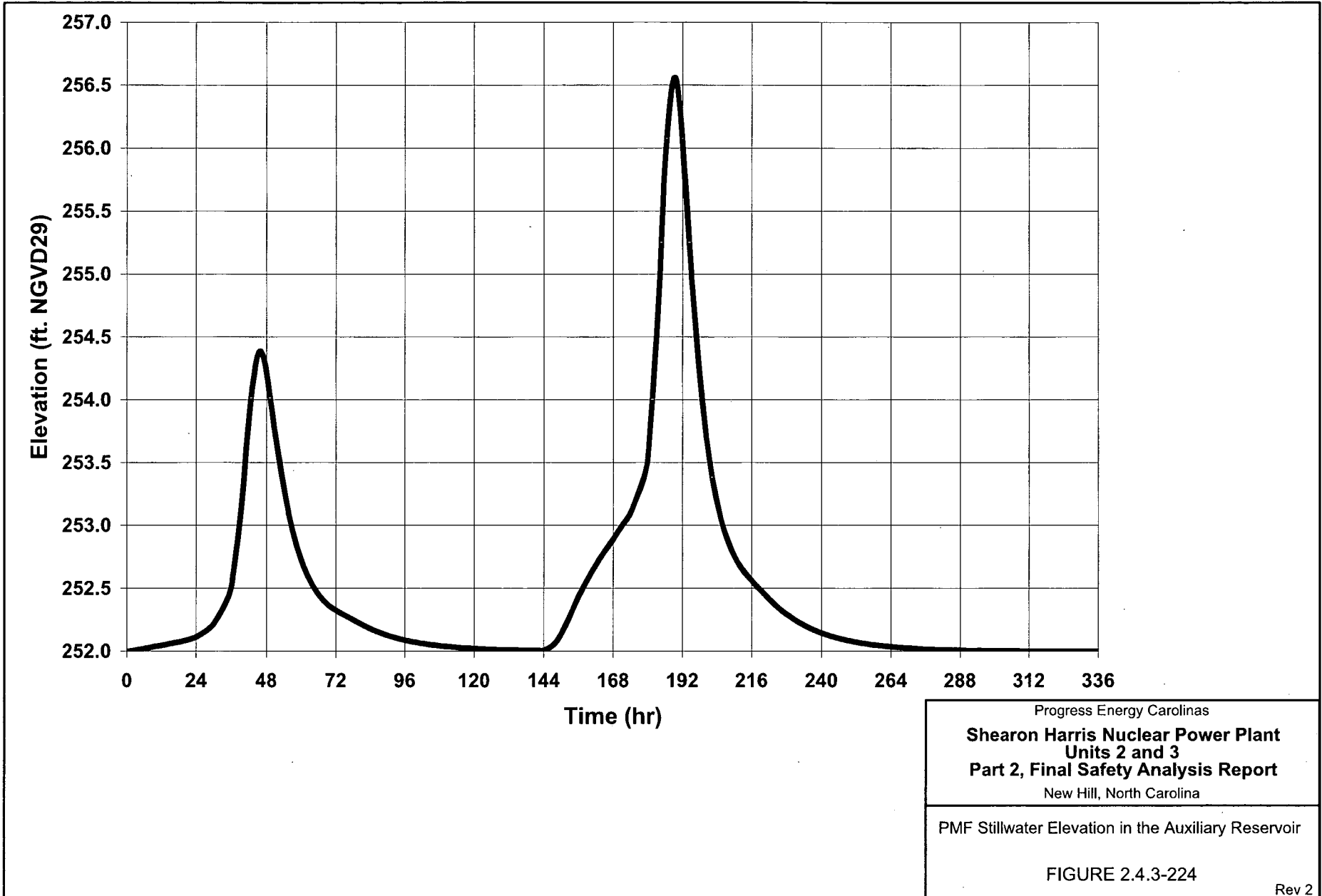


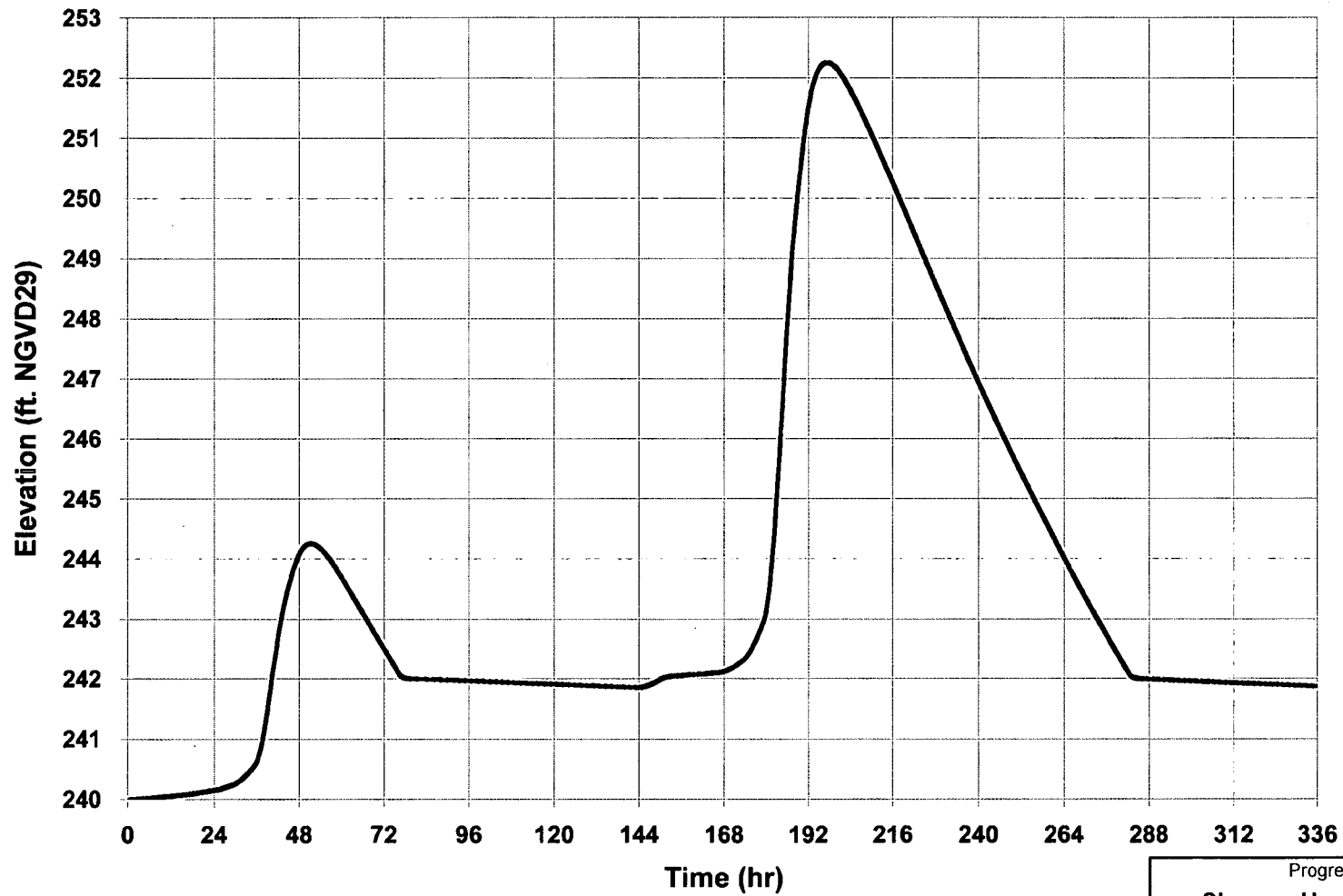




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PMF Inflow and Outflow Hydrographs
for the Main Reservoir
FIGURE 2.4.3-223



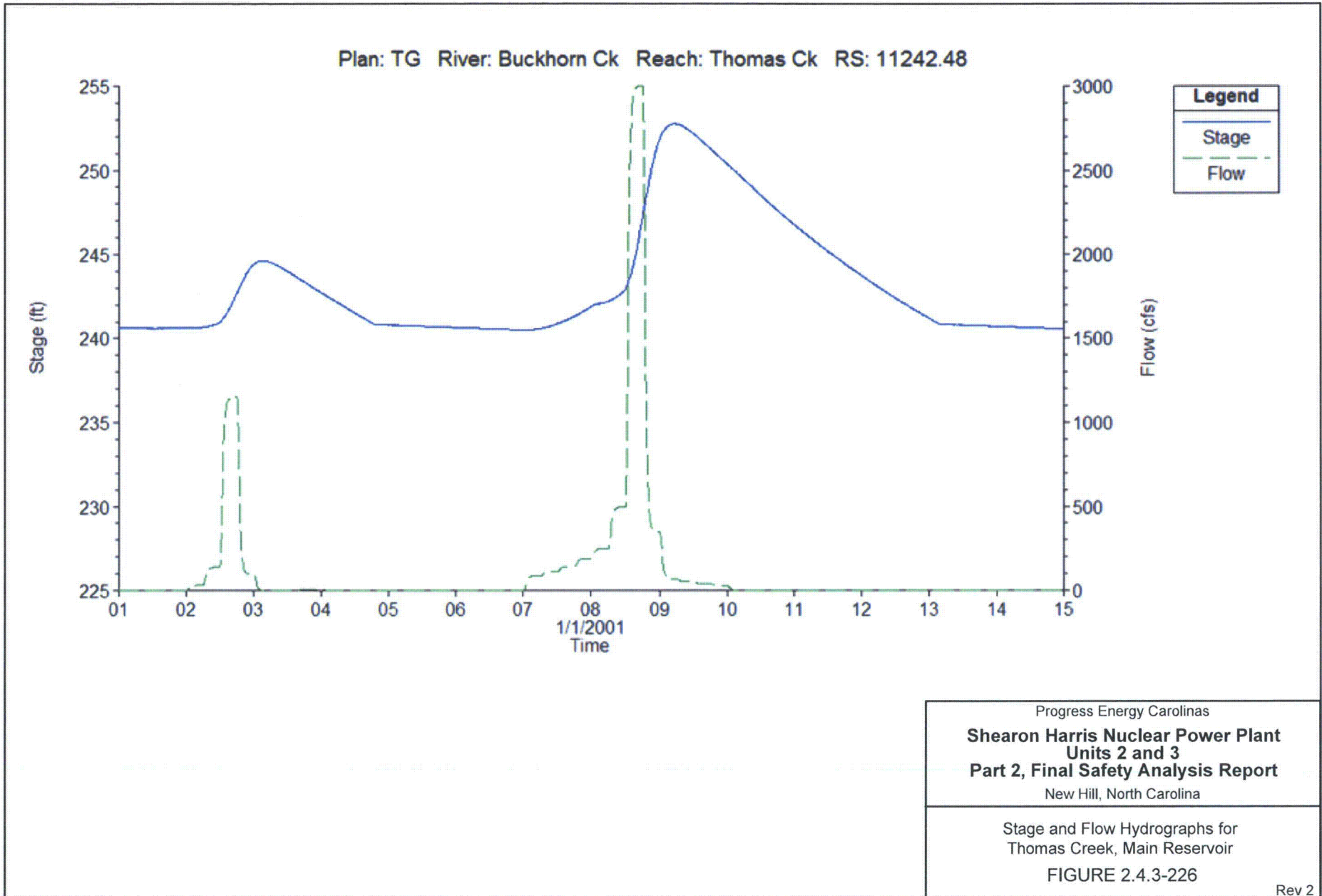


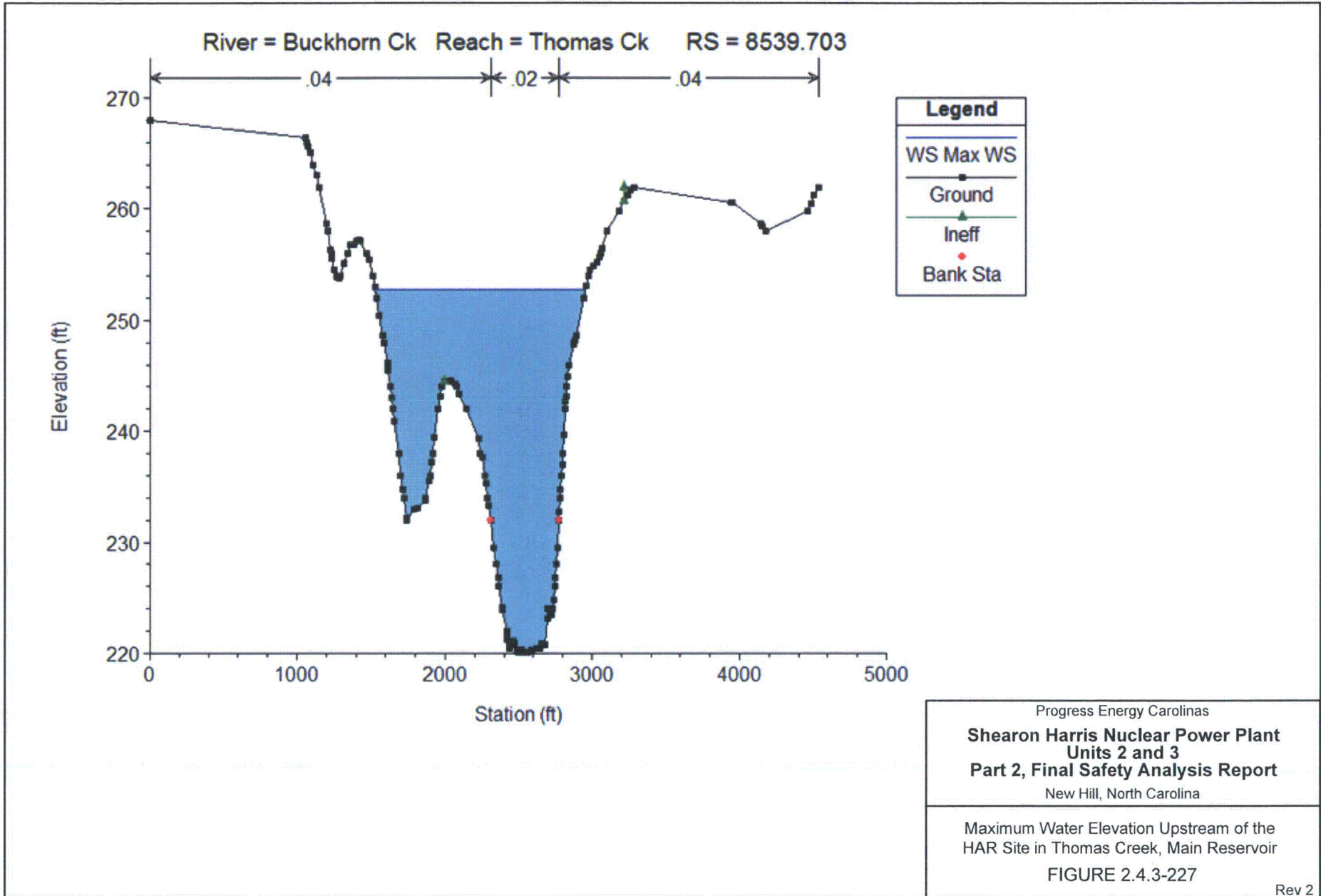
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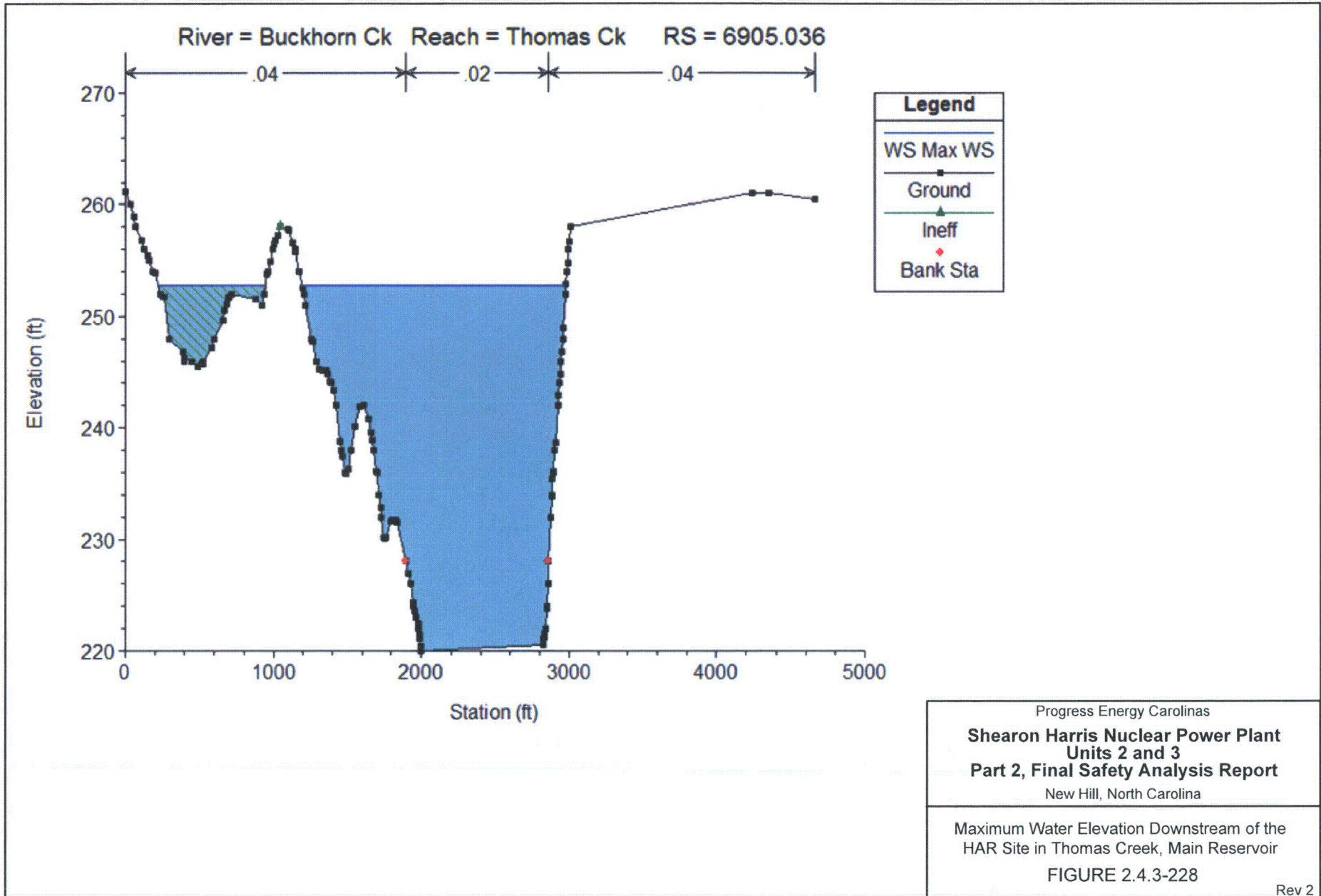
PMF Stillwater Elevation in the Main Reservoir

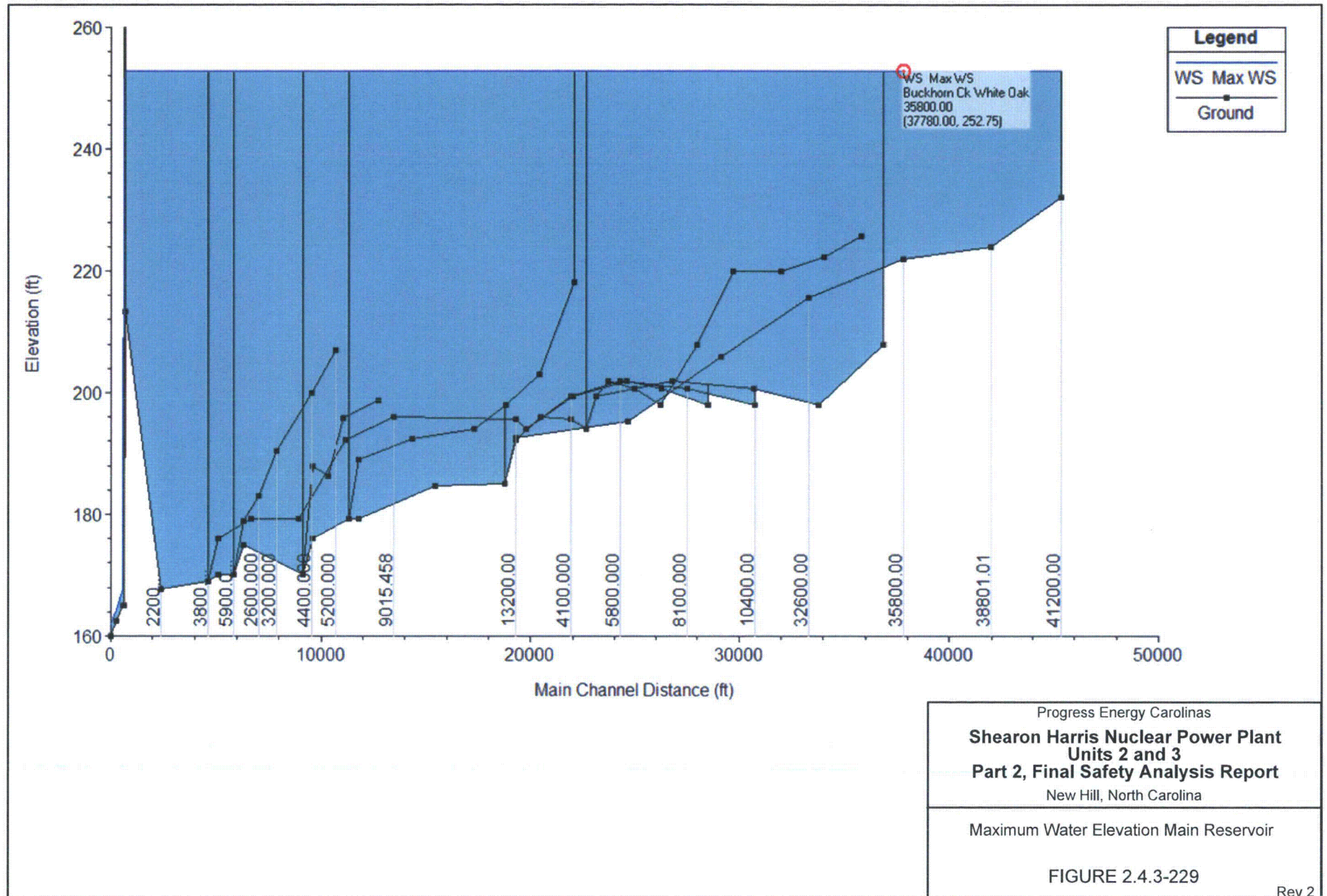
FIGURE 2.4.3-225

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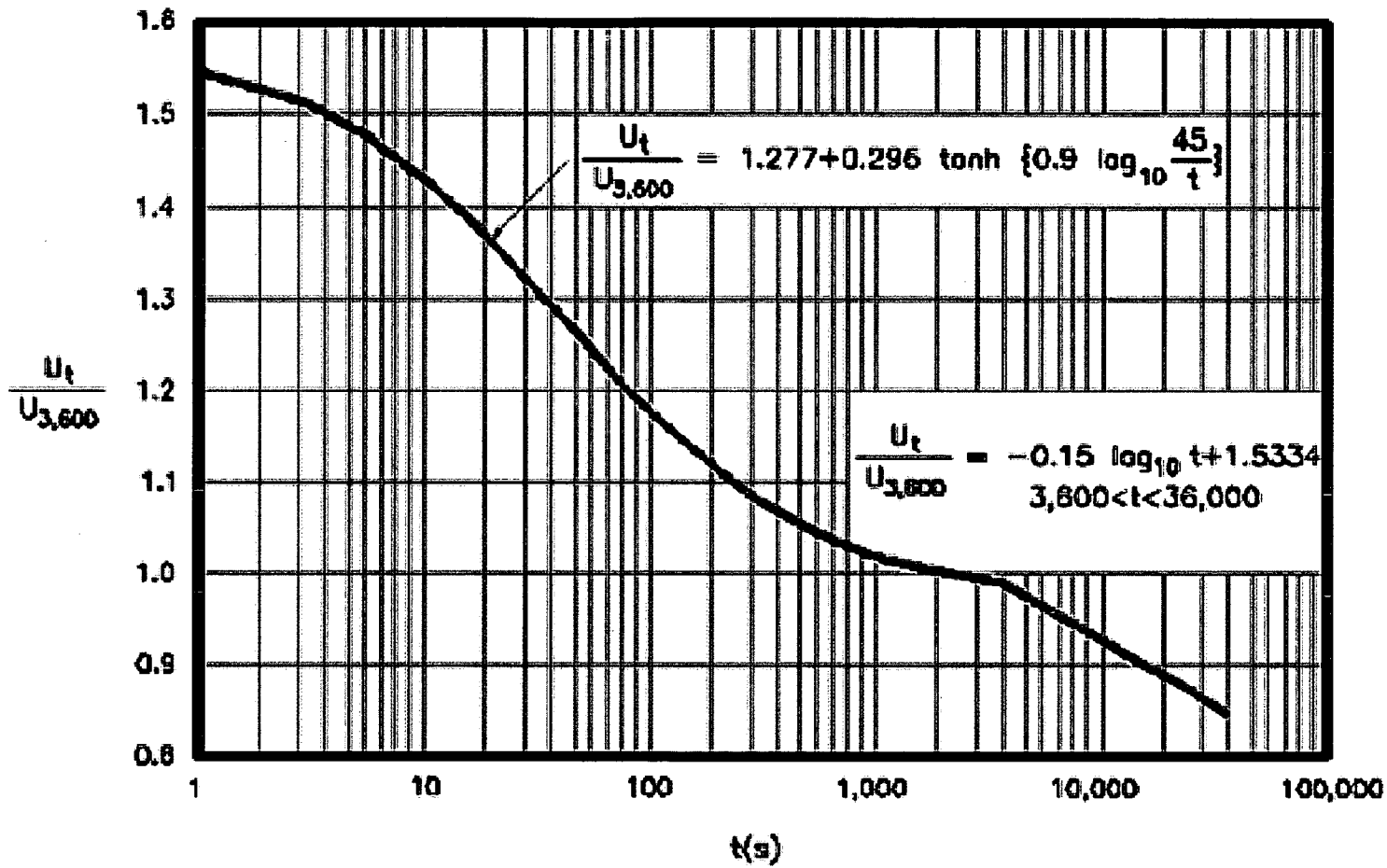






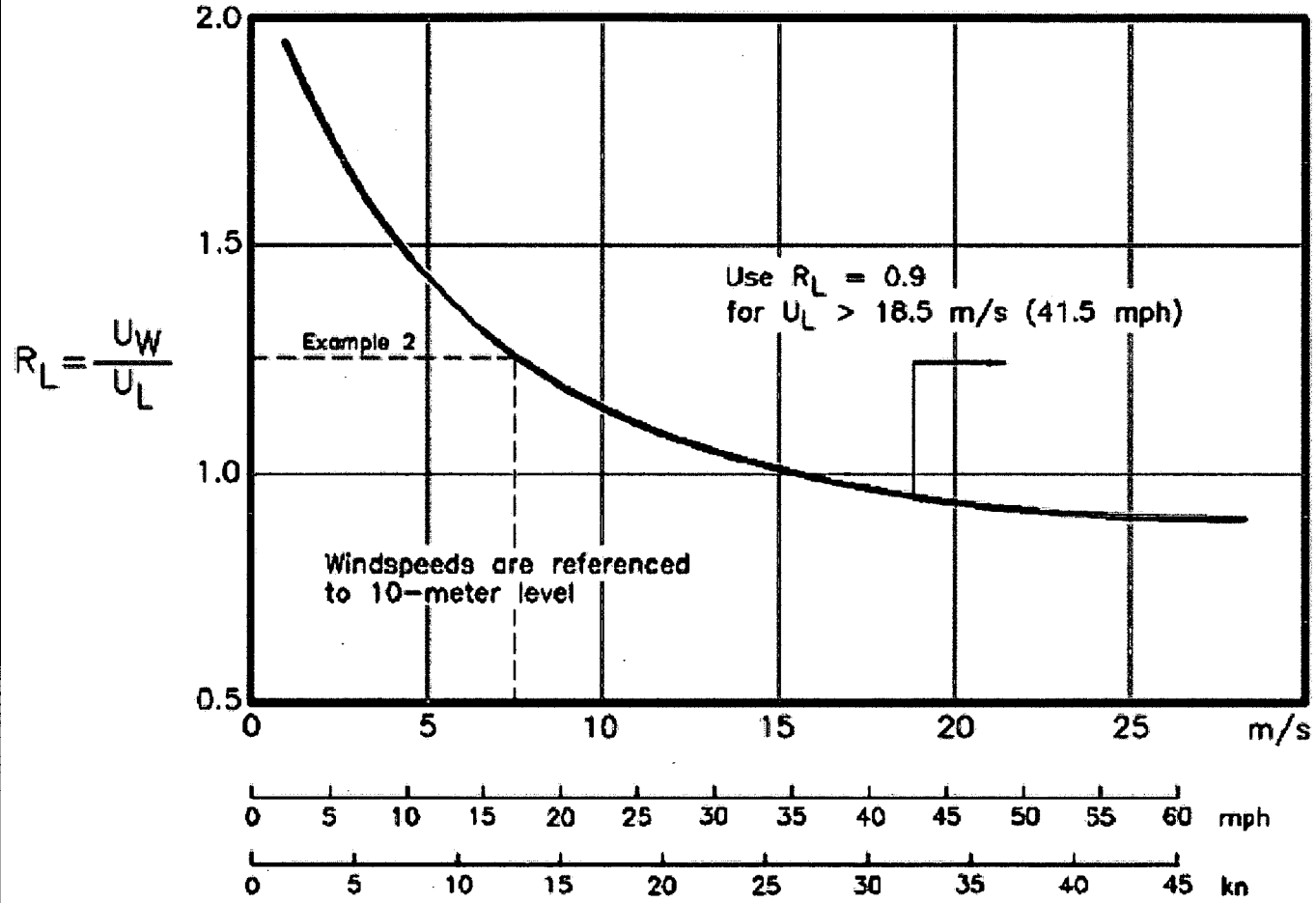






SOURCE: USACE, EM 1110-2-1100 PART II (2006)

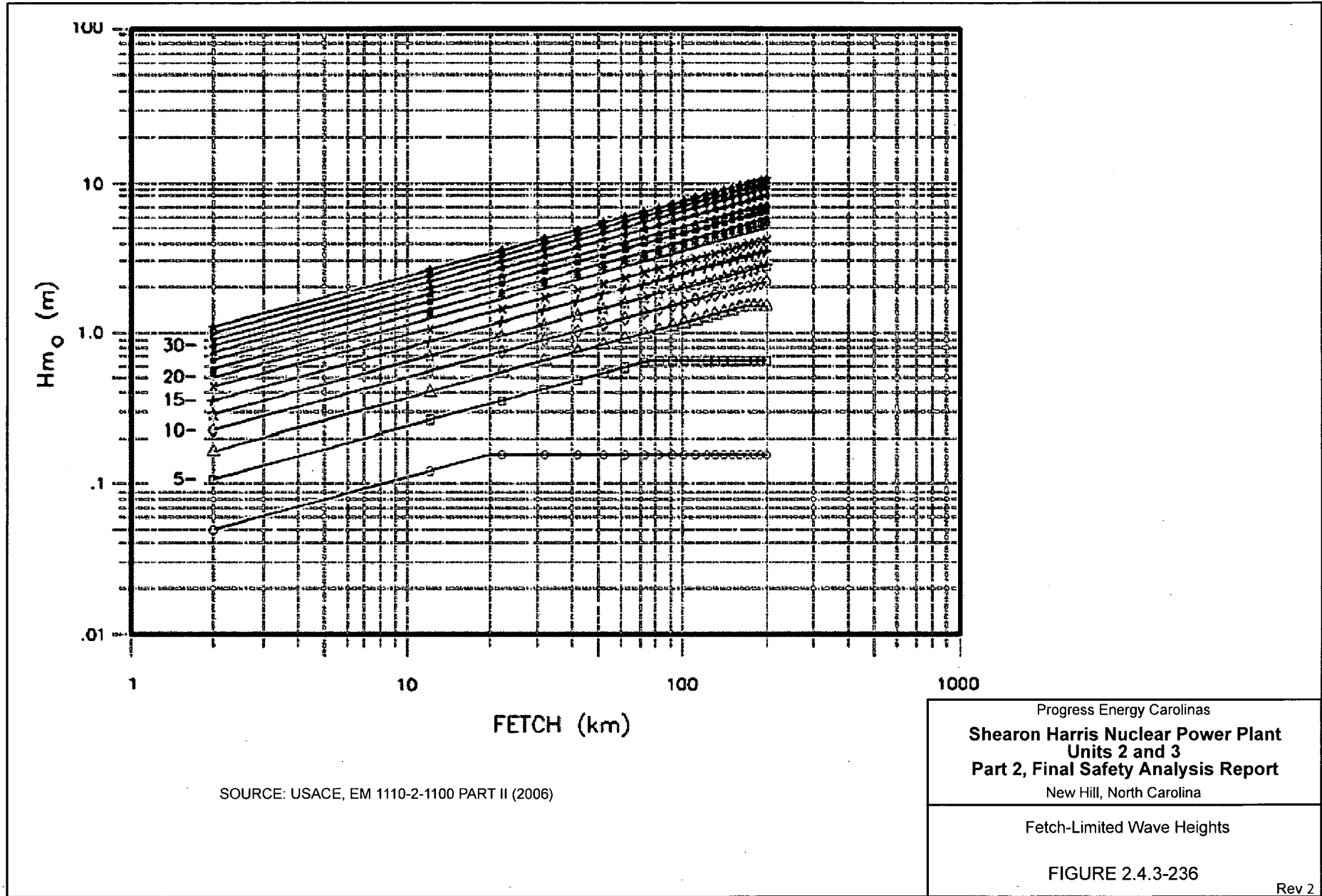
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Ratio of Wind Speed of t Duration, U _t to the 1-Hour Wind Speed U ₃₆₀₀ FIGURE 2.4.3-234

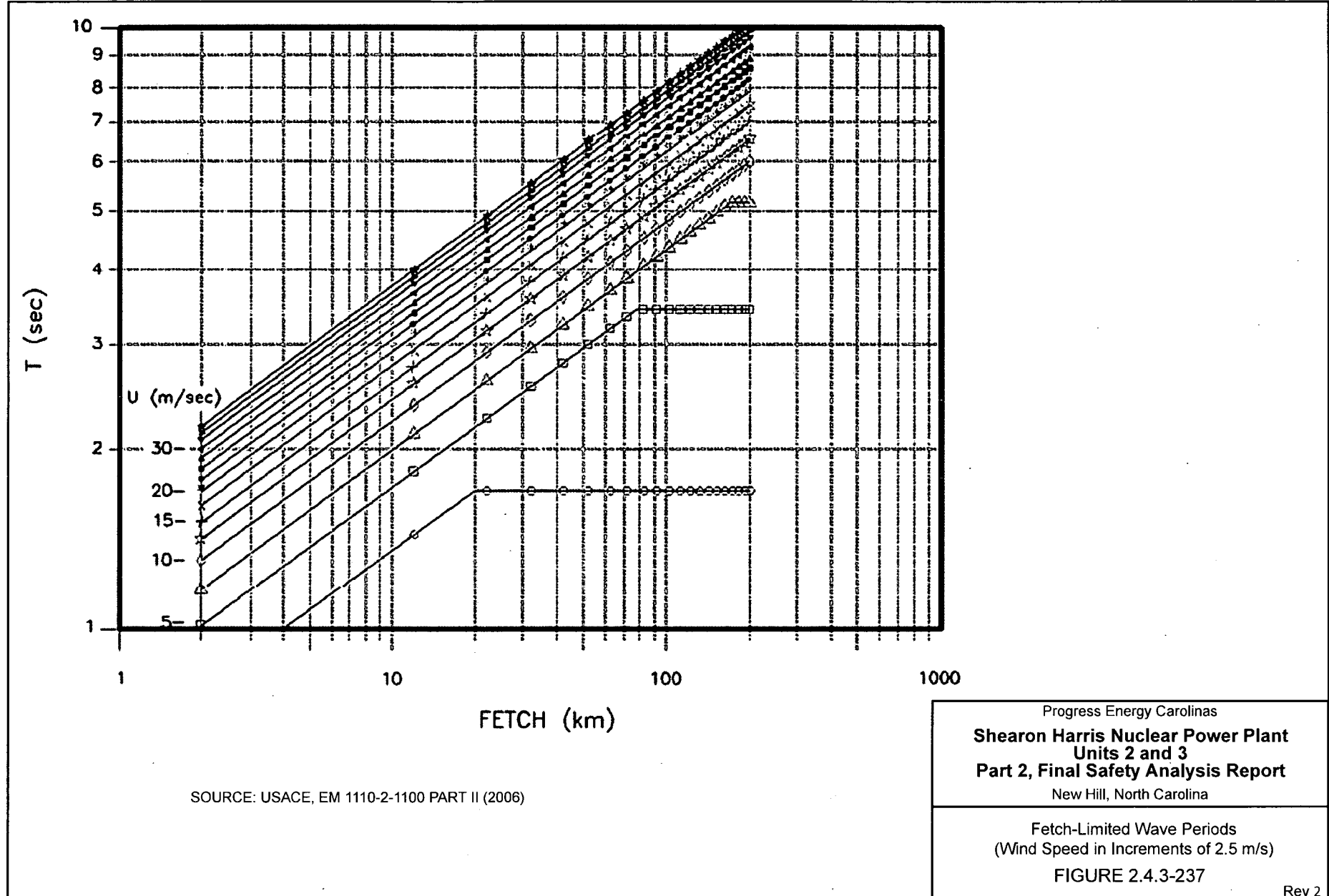


U_L

SOURCE: USACE, EM 1110-2-1100 PART II (2006)

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Ratio R_L of Wind Speed Overwater U_W to Wind Speed Overland U_L as a Function of Wind Speed Overland U_L FIGURE 2.4.3-235
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SOURCE: USACE, EM 1110-2-1100 PART II (2006)

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Fetch-Limited Wave Periods
 (Wind Speed in Increments of 2.5 m/s)
FIGURE 2.4.3-237
 Rev 2