Progress Energy

Serial: NPD-NRC-2009-229 10CFR52.79
November 19, 2009

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. 009 RELATED TO PROBABLE MAXIMUM FLOOD ON STREAMS AND RIVERS

References:

1. Letter from Manny Comar (NRC) to James Scarola (PEC), dated September 17, 2008, "Request for Additional Information Letter No. 009 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. 009 Related to Probable Maximum Flood on Streams and Rivers", Serial: NPD-NRC-2008-055
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. 009 Related to Probable Maximum Flood on Streams and Rivers", Serial: NPD-NRC-2009-050

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 Reference 1. A revised response to two of the NRC questions (02.04.03-1 and 02.04.03-3) is provided in the enclosure.

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,

John Elnitsky
Vice President
Nuclear Plant Development

Progress Energy Carolinas, Inc.
P.O. Box 1551
Raleigh, NC 27602

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NRO

Enclosure

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. 009
Related to SRP Section 02.04.03 for the Combined License Application,
Dated September 17, 2008**

NRC RAI #	Progress Energy RAI #	Progress Energy Response
02.04.03-1	H-0488	Revised response enclosed – see following pages
02.04.03-2	H-0433	April 1, 2009; NPD-NRC-2009-050
02.04.03-3	H-0489	Revised response enclosed – see following pages

NRC Letter No.: HAR-RAI-LTR-009

NRC Letter Date: September 17, 2008

NRC Review of Final Safety Analysis Report

NRC RAI NUMBER: 02.04.03-1

Text of NRC RAI:

The applicant derived longest flow path for the residual subbasin extends beyond the subbasin and terminates at the spillway of the Main dam, producing a longer lag time than if the flow path terminated at the edge of the residual subbasin. Staff request that the applicant describe how the increased lag time from this approach results in a conservative PMF estimate.

PGN RAI ID #: H-0488

PGN Response to NRC RAI:

After Progress Energy Carolina, Inc.'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 submitted revised responses to this RAI and RAI 02.04.03-3 and responded to new RAI 02.04.03-4 on April 1, 2009, in order to accomplish 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' *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 RAI 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 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. The two PMF strategies are as follows:

- 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 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.

This RAI includes the following sections:

- Part 1: Revised Runoff Modeling Approach – The revised modeling approach represents a kinematic routing of surface water runoff that includes both pool level routing and runoff from the reservoir pool surface area. The conservatism used for the lag time parameter is addressed in detail.
- Part 2: Estimate of the Backwater Effect at the Plant Site – An estimate of the impacts associated with PMF-induced backwater effects within the Main Reservoir has been incorporated to add additional conservatism in the PMF analysis for the HAR site.

The information in this RAI supersedes the previous RAI responses, which were submitted to the NRC by letters dated October 31, 2008 (Serial NPD-NRC-2008-055) and April 1, 2009 (Serial NPD-NRC-2009-050).

Part 1: Revised Runoff Modeling Approach

A runoff model is used to transform excess precipitation into surface runoff. As an alternative approach, a distributed inflow procedure was used, in which runoff was modeled using two different methods, one that developed PMF inflow hydrographs for all sub-basins, and one that considered direct rainfall on the 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 – Rainfall directly over reservoir pool areas was converted into runoff without any loss and lag time.
- No reach routing was used, such that the traveling time of runoff from land areas into the reservoir is neglected.
- Level pool routing was used for both the Auxiliary and Main Reservoirs.

Overland Runoff

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 (1 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 particular basin. Thus, unit hydrographs are developed for each sub-basin using their specific parameters.

Several different methods can be used to develop a unit hydrograph for a given sub-basin. Selecting an appropriate method depends on knowledge of the hydrologic response characteristics of the sub-basin. In this study, the Snyder's synthetic hydrograph method was selected to develop unit hydrographs for each sub-basin. The required generalized values of the Snyder Unit hydrograph parameters defining its shape were obtained from the existing Shearon Harris Nuclear Plant Unit 1 (HNP) Final Safety Analysis Report (FSAR), Amendment

53. 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 (QP) and the lag time (t_L) that are defined as (FSAR Reference 2.4-226):

$$t_L = CC_t\left(LL_C\right)^{0.3} \qquad (1)$$

$$Q_P = \frac{640C_PA}{t_L} \qquad (2)$$

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

C_t = Snyder basin lag coefficient

C_P = Snyder peaking coefficient

C = unit conversion factor; it is equal to 1.0 when English units are used.

A = drainage area in square miles

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.

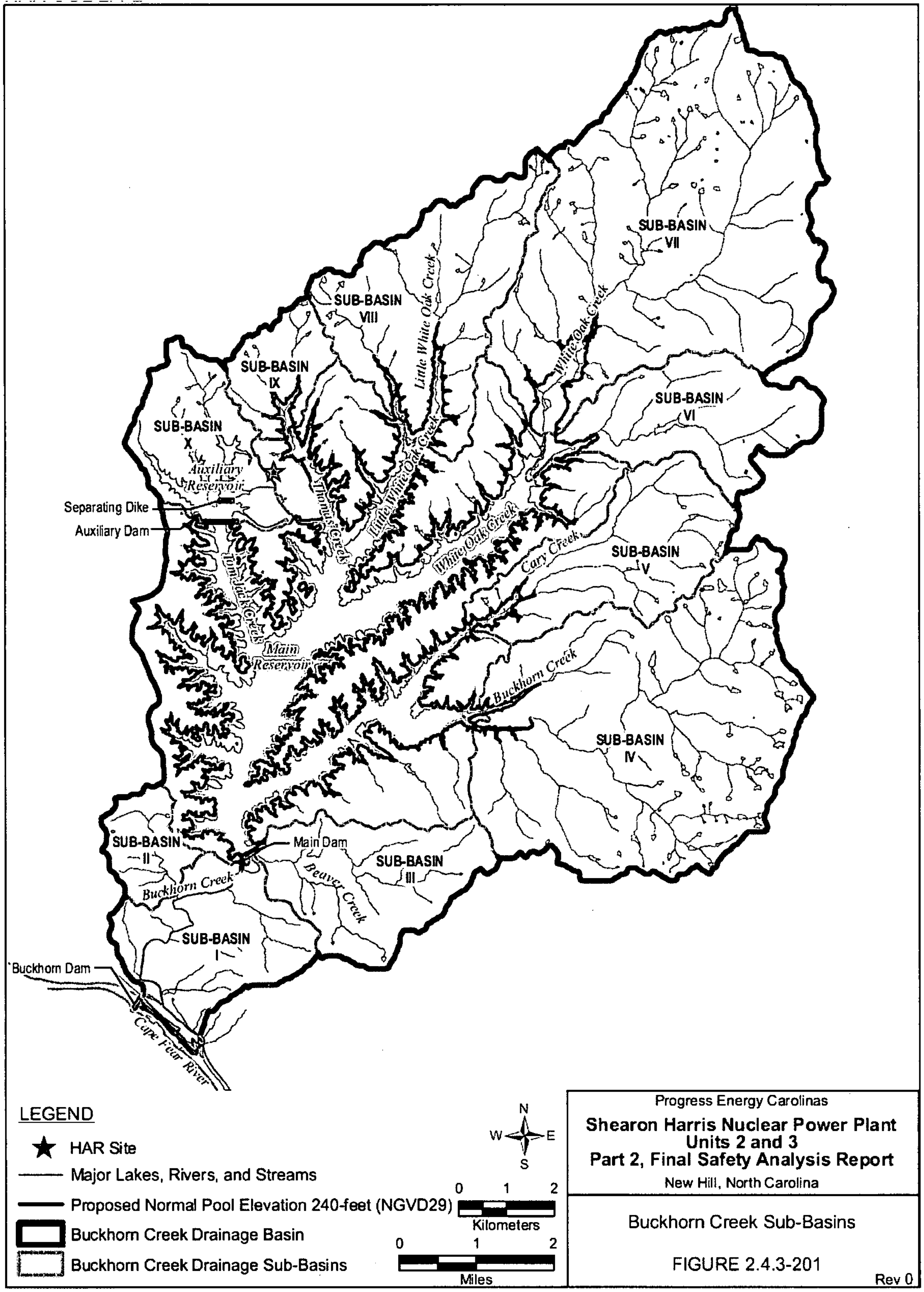
The Snyder hydrograph method was selected as acceptable. The HNP FSAR has generalized values for C_t and C_P. The values of C_t and C_P are 3.91 and 0.75, respectively.

To apply the unit hydrograph approach to the Buckhorn Creek Drainage Basin, unit hydrographs were developed for all the sub-basins above the Main Dam, excluding the reservoir pool areas. **Figure 1** (FSAR 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 square kilometers (km^2) (70.3 square miles [mi^2]). This area also includes the drainage area at the Auxiliary Reservoir. **Table 1** (FSAR Table 2.4.3-218) lists the areas of the Auxiliary Reservoir Surface, Main Reservoir Surface, Residual Land Surface, and Sub-basins IV, V, VI, VII, VIII, IX, and X.

Table 1 (FSAR Table 2.4.3-218): Sub-Basin Areas

Basin ID	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**	

Figure 1 (FSAR Figure 2.4.3-201): Study Area



Progress Energy Carolinas
**Shearon Harris Nuclear Power Plant
Units 2 and 3
Part 2, Final Safety Analysis Report**
New Hill, North Carolina

Buckhorn Creek Sub-Basins

FIGURE 2.4.3-201

Rev 0

A unit hydrograph has meaning only in connection with a specific duration of runoff, whereas a sub-basin may have many different unit hydrographs, each associated with a different duration of excess rainfall. For the Snyder's synthetic unit hydrograph, $D = T_L/5.5$, the catchment lag, T_L, 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 1**. **Table 2** (FSAR Table 2.4.3-219) lists various watershed parameters, along with the standard Snyder Hydrograph parameters used in the HEC-HMS model.

Table 2 (FSAR Table 2.4.3-219): 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 (0.5)
L_{ca} (mi.)	2.28	2.02	1.64	1.92	1.02	0.41	1.37	3.08 (0.2)
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 (hr)	8.4	6.9	6.3	7.8	5.5	3.1	5.6	10.6 (2.0)
Q_p (cfs)	712	252	259	808	353	175	211	796 (4311)

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
mi. = mile
hr = hour
cfs = cubic foot per second

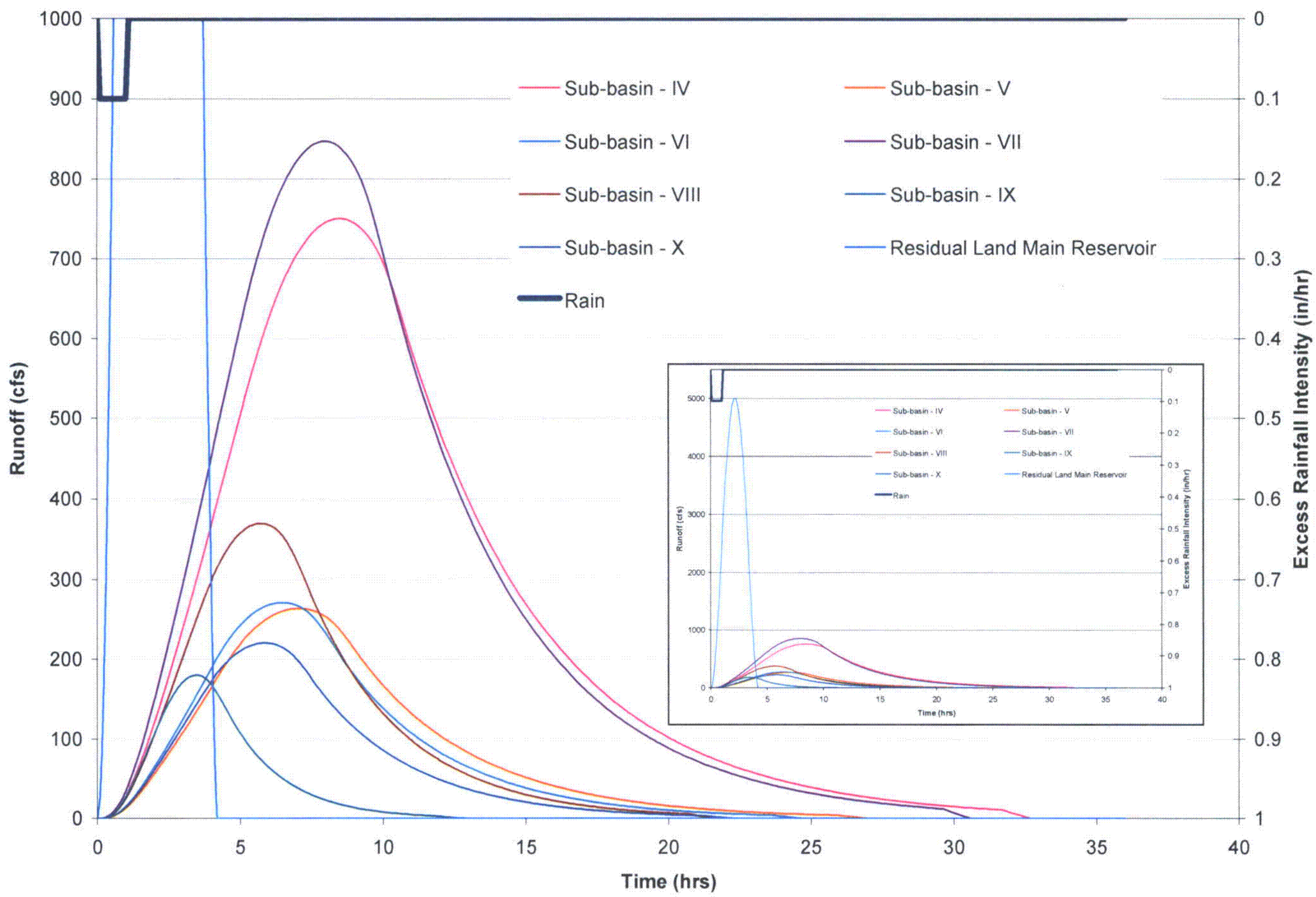
Alternate conservative parameters for the residual area were used in order to reduce the lag time from 10.6 to 2.0 hours. This change has increased the peak flow from the residual area from 796 cfs to 4311 cfs. These additional conservative steps were performed to address the NRC's concern with lag time based on Snyder's unit hydrograph equations and geometrical characteristics associated with the land area surrounding the Main Reservoir.

Using the Standard Snyder hydrograph parameters presented in **Table 2** as input in the HEC-HMS model, 1-hour unit hydrographs were developed, as shown in **Figure 2**. The parameters associated with the 1-hour unit hydrographs for each basin are provided in **Table 3**.

Table 3: 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)	749.9	263.8	271.2	846.9	369.4	180.5	220.3	4,991.9
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

Figure 2: 1-Hour Unit Hydrographs Developed for FSAR Study



Part 2: Estimate of the Backwater Effect at the Plant Site

To assess the impact of the PMF event, including backwater effects at Shearon Harris Nuclear Power Plant Unit 2 (HAR 2) and Shearon Harris Nuclear Power Plant Unit 3 (HAR 3), an unsteady state HEC-RAS model was developed. The geometric data necessary to develop such a model include the following:

- Stream system connectivity between the Auxiliary and Main Reservoirs and associated tributaries
- Cross-section data for various tributaries
- Hydraulic structure data, such as the Main Dam

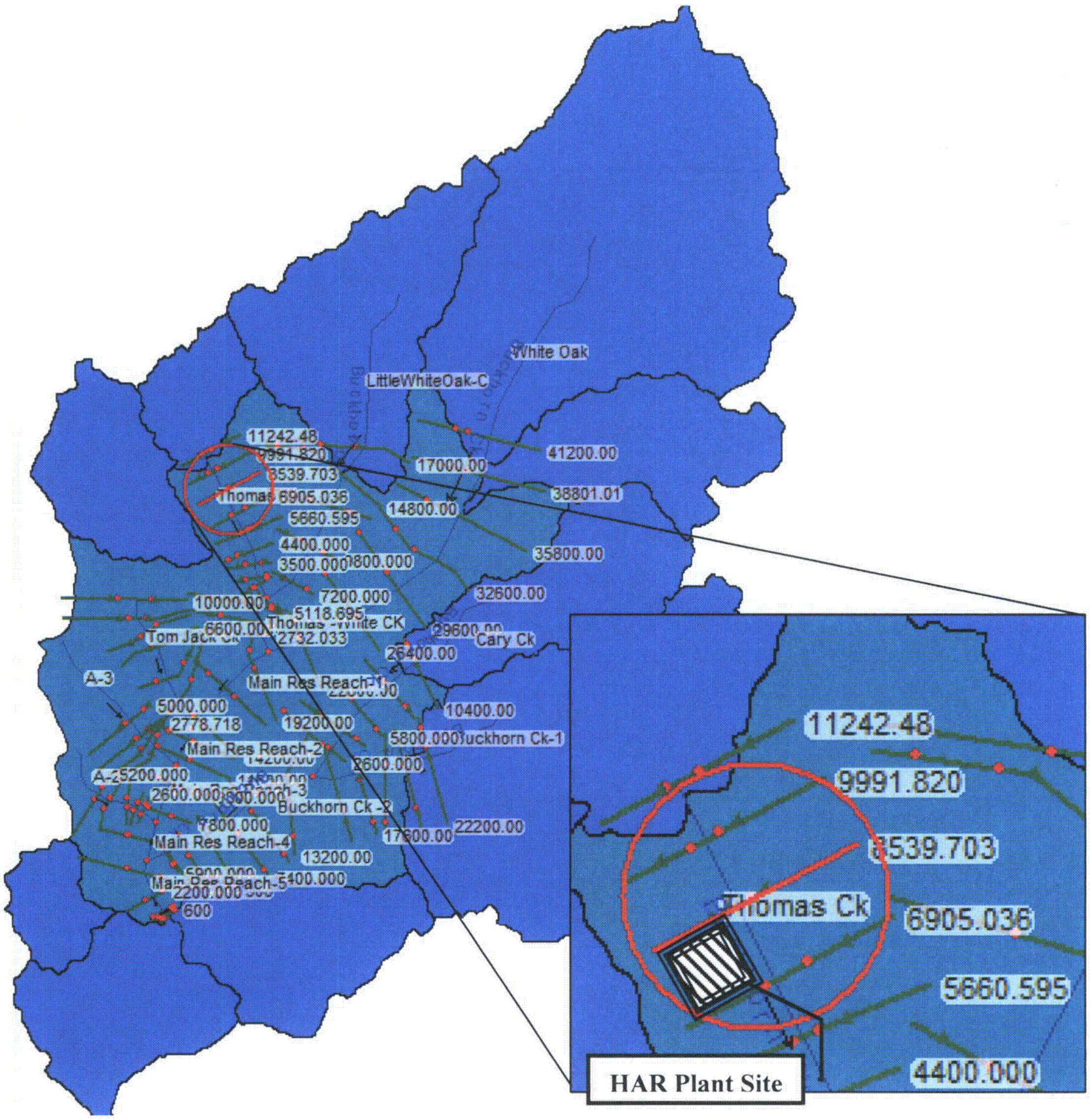
Figure 3 shows an overview of the study area and the various sub-basins and stream segments assessed in the model. The location of the HAR plant site is indicated by the shaded square in the inset.

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 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.

The 3-D surface was further processed using ArcGIS to generate 1-foot contour lines from the 220-foot NGVD29 level down to the bottom of the lake at approximately 154 feet NGVD29. Two-foot contours above the 220-foot NGVD29 level were extracted from the Chatham County and Wake County GIS databases. The Buckhorn Basin was obtained for a constraint, so as not to contour all three counties, which resulted in a contour range of 222 to 488 feet NGVD29. Using ArcGIS Append, the contours above 220 feet NGVD29 were combined with the existing contours below 220 feet NGVD29 to generate a comprehensive elevation surface for the basin.

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

Figure 3: Study Area and Stream System Connectivity



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 streams 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 main channel were defined up to 220 feet NGVD29 elevation)
- Contraction and Expansion Coefficients (contraction = 0.3 and expansion = 0.6)

In addition to geometric data, unsteady flow data that consist of the boundary and initial conditions are required input for the HEC-RAS model. For this requirement, PMF inflow hydrographs associated with each sub-basin and corresponding to the 25 percent peaking factor were applied. **Figure 4** presents the inflows as boundary conditions to the unsteady state model. Both the inflow hydrographs from the residual area and Main Reservoir pool area were combined together and introduced just downstream of the White Oak Creek junction with the Main Reservoir Reach-1. The model also requires initial flows at all the boundaries. **Figure 5** presents flow data that were input into the HEC-RAS model as the initial conditions.

Figure 4: Inflows as Boundary Conditions to the Unsteady State Model

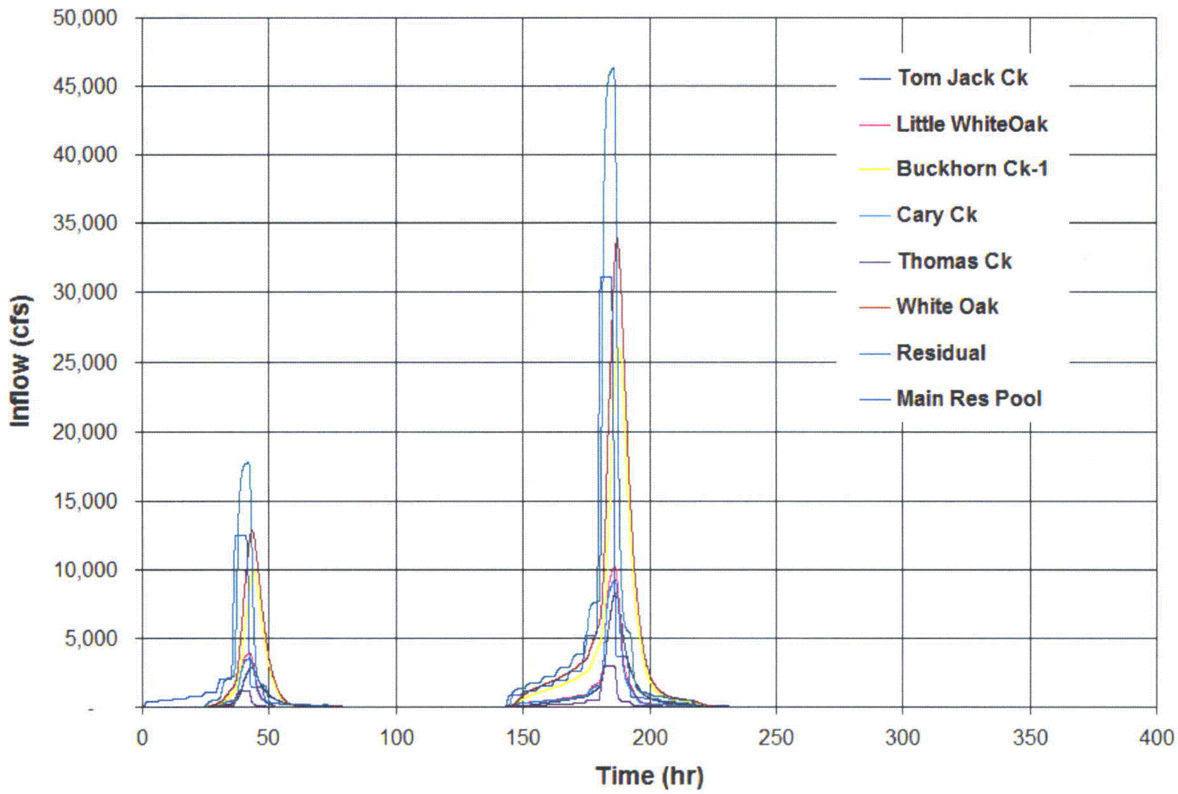
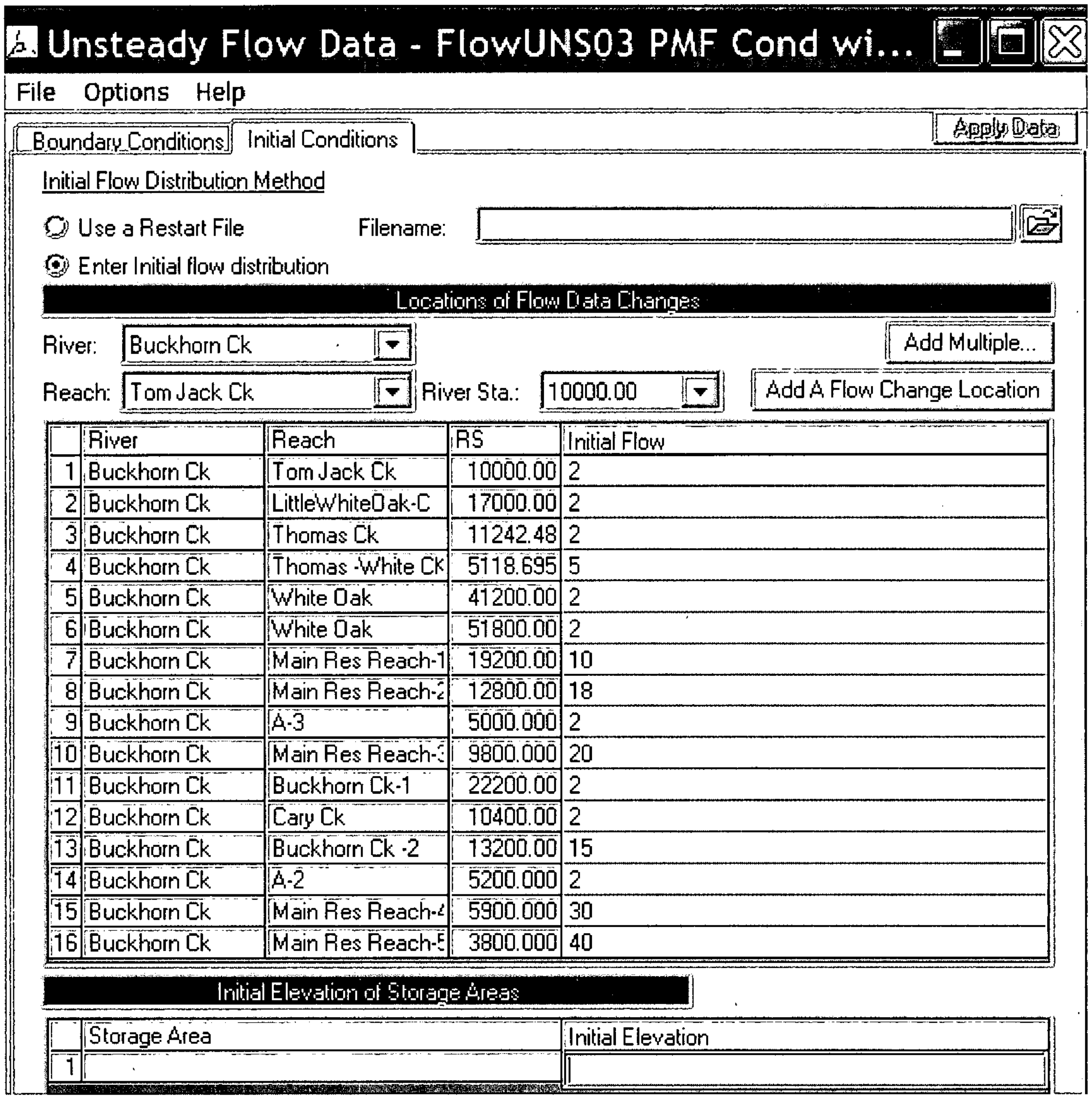


Figure 5: Initial Flows as Initial Conditions in the Unsteady State Model



Finally, 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 and most downstream end of the Main Reservoir. In order to ensure protection of safety-related structures against flooding and dynamic effects of wave action due to wind-generated activity, some modifications were considered in the spillway design of the Main Dam. These modifications are designated as Option 1 and Option 2:

- 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. In order to determine the length of emergency spillway, different scenarios using various emergency spillway lengths were evaluated.
- 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. In order to

determine the upstream water elevation at which the Tainter gate is completely opened, different scenarios were evaluated using various lake level target elevations to begin opening the Tainter gate.

PMF stillwater elevations were determined considering various scenarios for each of these two options. The results of these scenarios are summarized as follows:

- For Option 1, it was observed that the length of the emergency spillway should be at least 400 feet.
- For Option 2, it was observed that the Tainter gate needs to be opened before the lake water elevation exceeds 243 feet NGVD29.

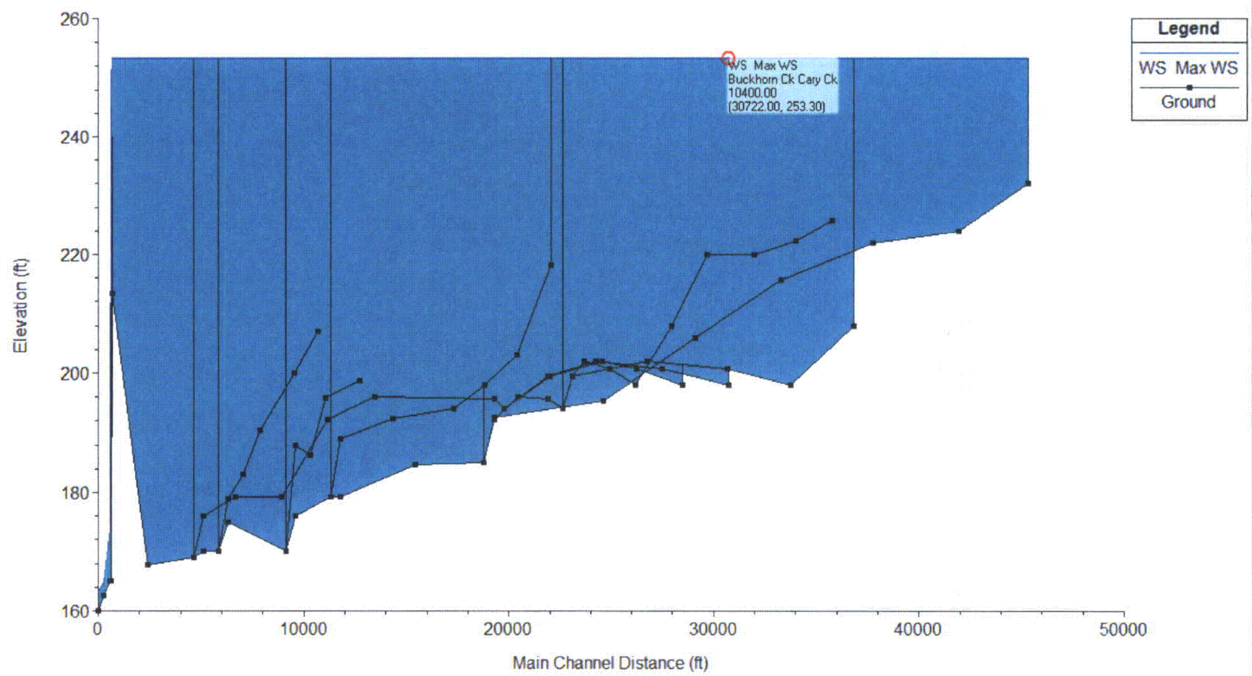
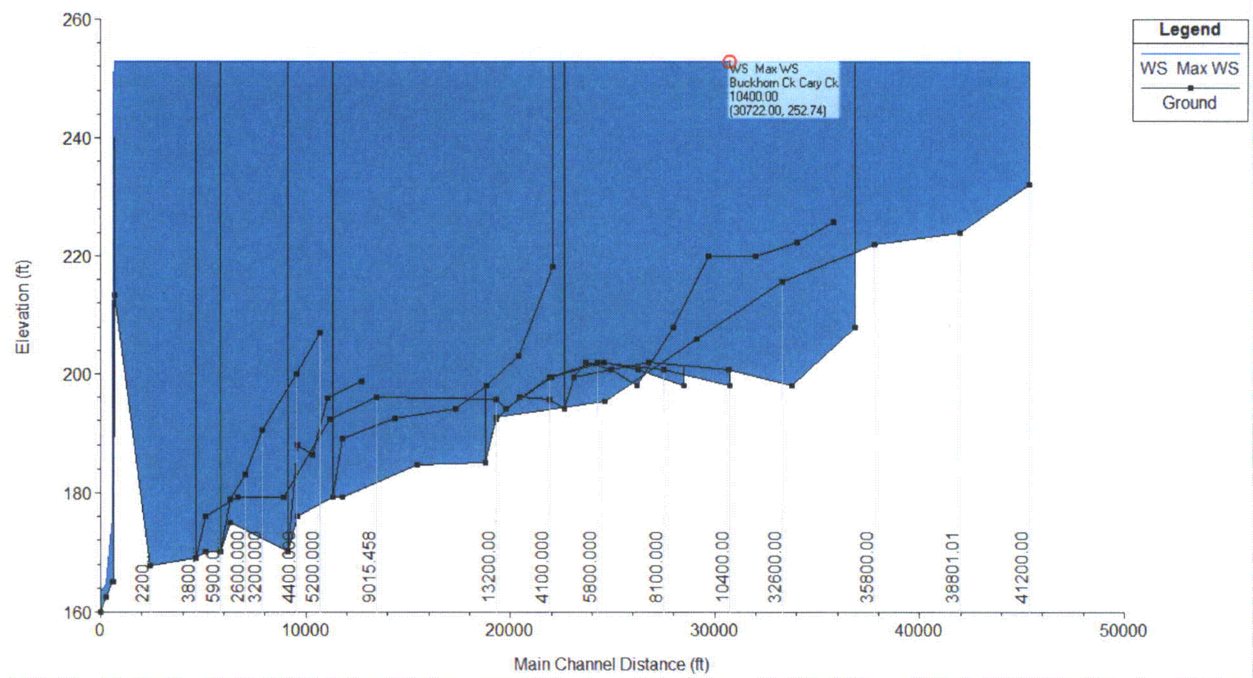
The HEC-RAS model for the Buckhorn Creek Drainage Basin was initiated with conservative conditions. Specifically, the initial stillwater elevation was given as 240.36 feet NGVD29; this elevation is 0.36 foot above the crest of the Main Dam spillway (240 feet NGVD29).

The results of the various scenarios corresponding to the two options described above are presented in the following sections.

Option 1 Results

Figure 6 and **Table 4** present the profile plots for various scenarios considered for Option 1. **Figure 7** compares these profiles along with the initial stillwater elevation for all locations from the downstream end of the Main Reservoir (Main Dam) to the upstream end of various creeks of the Main Reservoir, including Thomas Creek. As depicted on **Figure 3**, Thomas Creek originates just upstream of HAR 2 and HAR 3.

Figure 6: Profile Plots for Various Scenarios of Option 1

<table>
<tr><th>Scenario</th><th>Profile Plot</th></tr>
<tr><td>Length of Emergency Spillway = 400 ft</td><td>
</td></tr>
<tr><td>Length of Emergency Spillway = 500 ft</td><td>
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Length of Emergency Spillway = 600 ft

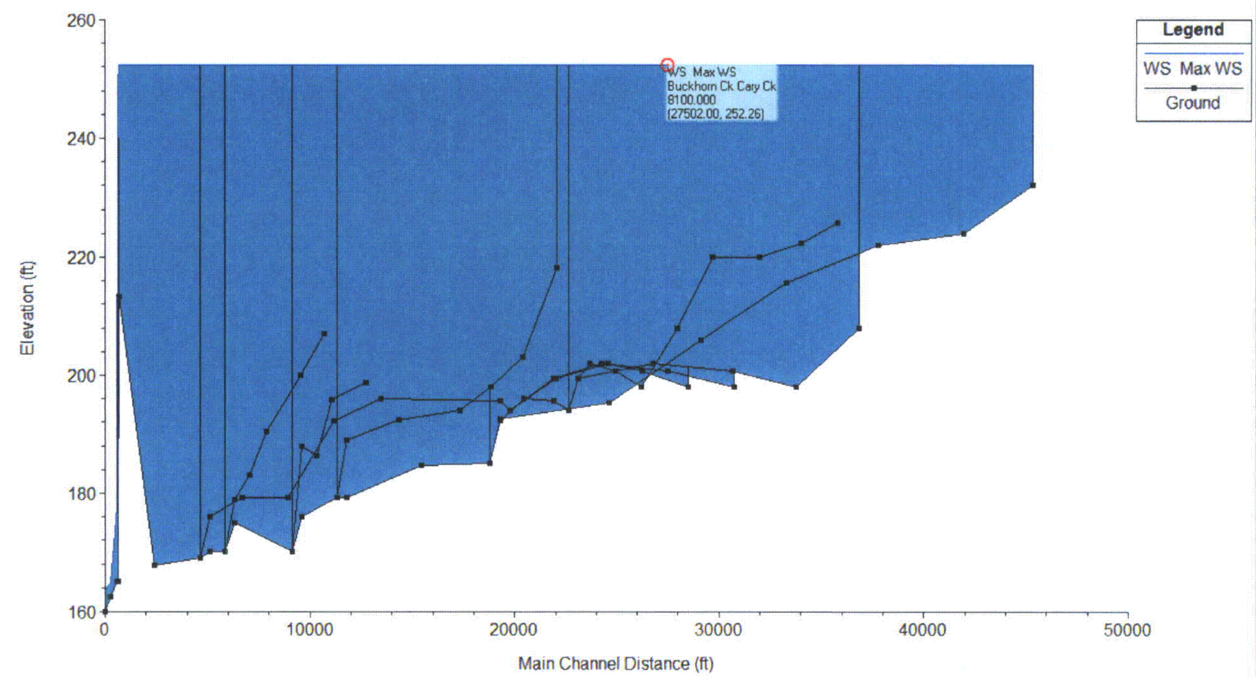


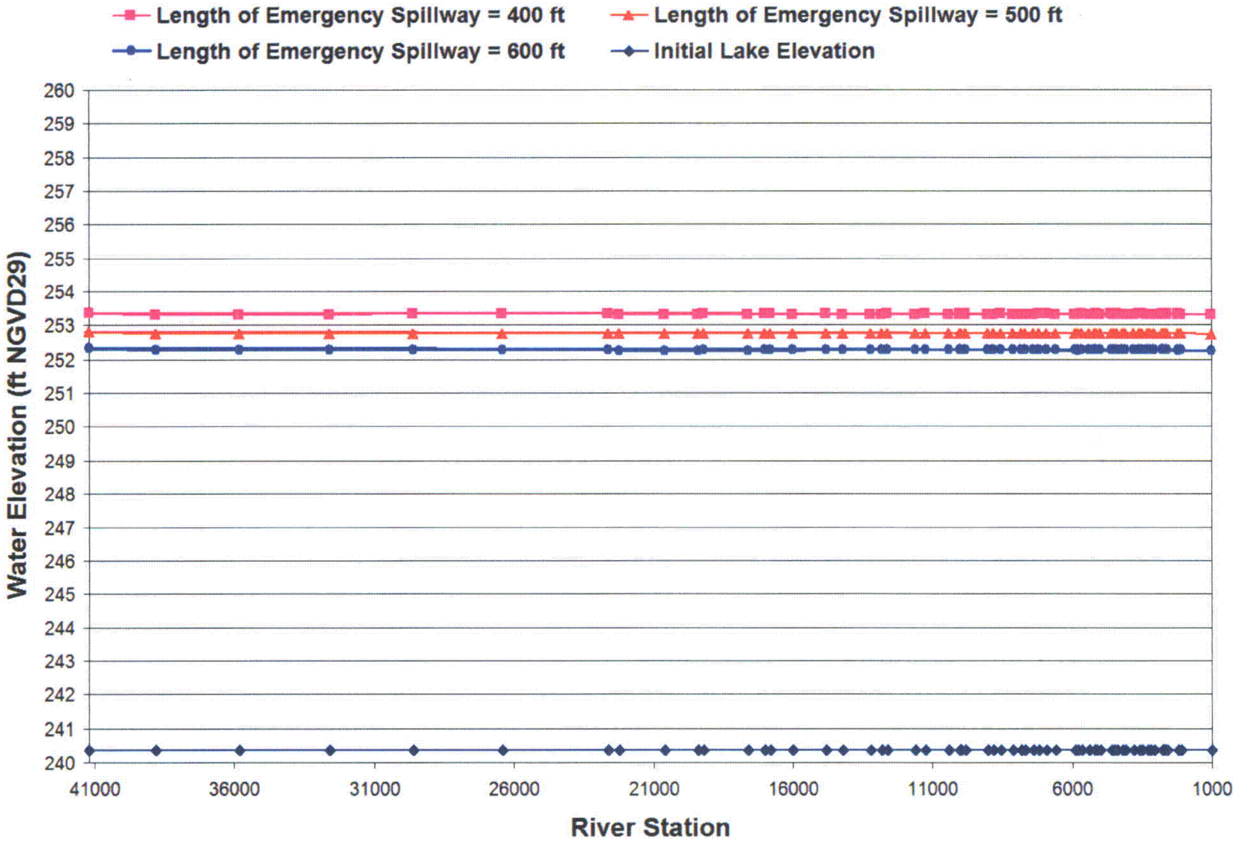
Table 4: PMF Profiles for Various Scenarios of Option 1

Reach	River Station	Q Total (cfs)	Initial Lake Elevation (ft NGVD29)	Length of Emergency Spillway = 400 ft	Length of Emergency Spillway = 500 ft	Length of Emergency Spillway = 600 ft
Tom Jack Ck	10000	2470.61	240.36	253.30	252.74	252.26
Tom Jack Ck	8800	2471.02	240.36	253.30	252.74	252.26
Tom Jack Ck	7700	2467.67	240.36	253.30	252.74	252.26
Tom Jack Ck	6600	2467.20	240.36	253.30	252.74	252.26
Tom Jack Ck	4500	2471.90	240.36	253.30	252.74	252.26
Tom Jack Ck	2661.272	2450.20	240.36	253.30	252.74	252.26
LittleWhiteOak-C	17000	1457.44	240.36	253.31	252.75	252.27
LittleWhiteOak-C	14800	1453.65	240.36	253.31	252.75	252.27
LittleWhiteOak-C	12600	1453.52	240.36	253.31	252.75	252.27
LittleWhiteOak-C	9800	1457.84	240.36	253.31	252.75	252.26
LittleWhiteOak-C	7200	1477.89	240.36	253.31	252.75	252.26
Thomas Ck	11242.48	216.92	240.36	253.31	252.75	252.27
Thomas Ck	9991.82	216.41	240.36	253.31	252.75	252.27
Thomas Ck	8539.703	210.90	240.36	253.31	252.75	252.27
Thomas Ck	6905.036	202.81	240.36	253.31	252.75	252.26
Thomas Ck	5660.595	192.20	240.36	253.31	252.75	252.26
Thomas Ck	4400	182.82	240.36	253.31	252.75	252.26
Thomas Ck	3500	175.69	240.36	253.31	252.75	252.26
Thomas Ck	2600	171.37	240.36	253.31	252.75	252.26
Thomas Ck	2205.884	168.13	240.36	253.31	252.75	252.26
Thomas -White CK	5118.695	1646.03	240.36	253.31	252.75	252.26
Thomas -White CK	4600	1631.91	240.36	253.31	252.75	252.26
Thomas -White CK	3578.48	1650.99	240.36	253.31	252.74	252.26
Thomas -White CK	2732.033	1639.85	240.36	253.31	252.74	252.26
White Oak	41200	20944.84	240.36	253.33	252.78	252.32
White Oak	38801.01	19429.99	240.36	253.30	252.74	252.26
White Oak	35800	19447.94	240.36	253.30	252.74	252.26
White Oak	32600	19536.05	240.36	253.30	252.74	252.26
White Oak	29600	19594.27	240.36	253.31	252.74	252.26
White Oak	26400	19648.73	240.36	253.31	252.74	252.26
White Oak	22600	19697.22	240.36	253.31	252.74	252.26
Main Res Reach-1	19200	21337.07	240.36	253.31	252.74	252.26
Main Res Reach-1	16800	21325.02	240.36	253.31	252.75	252.26
Main Res Reach-1	14200	42366.45	240.36	253.30	252.74	252.26
Main Res Reach-2	12800	44816.64	240.36	253.30	252.74	252.26
Main Res Reach-2	11600	44775.68	240.36	253.30	252.74	252.26
A-3	5000	2.00	240.36	253.30	252.74	252.26
A-3	3800	-0.07	240.36	253.30	252.74	252.26
A-3	3289.593	-0.60	240.36	253.30	252.74	252.26
A-3	2778.718	3.93	240.36	253.30	252.74	252.26

Table 4: PMF Profiles for Various Scenarios of Option 1

Reach	River Station	Q Total (cfs)	Initial Lake Elevation (ft NGVD29)	Length of Emergency Spillway = 400 ft	Length of Emergency Spillway = 500 ft	Length of Emergency Spillway = 600 ft
Main Res Reach-3	9800	44779.61	240.36	253.30	252.74	252.26
Main Res Reach-3	7800	44794.35	240.36	253.30	252.74	252.26
Buckhorn Ck-1	22200	11136.05	240.36	253.30	252.74	252.25
Buckhorn Ck-1	20600	11154.48	240.36	253.30	252.74	252.25
Buckhorn Ck-1	19400	11167.05	240.36	253.30	252.74	252.25
Buckhorn Ck-1	17600	11196.67	240.36	253.30	252.74	252.25
Buckhorn Ck-1	16000	11171.41	240.36	253.30	252.74	252.26
Cary Ck	10400	1211.55	240.36	253.30	252.74	252.26
Cary Ck	8100	1207.69	240.36	253.30	252.74	252.26
Cary Ck	5800	1202.94	240.36	253.30	252.74	252.26
Cary Ck	4100	1202.60	240.36	253.30	252.74	252.26
Cary Ck	2600	1196.97	240.36	253.30	252.74	252.26
Buckhorn Ck -2	13200	12368.37	240.36	253.30	252.74	252.26
Buckhorn Ck -2	9015.458	12360.96	240.36	253.30	252.74	252.26
Buckhorn Ck -2	7400	12358.13	240.36	253.30	252.74	252.26
Buckhorn Ck -2	5800	12353.34	240.36	253.30	252.74	252.25
Buckhorn Ck -2	4200	12348.79	240.36	253.30	252.74	252.26
Buckhorn Ck -2	3070.306	12345.45	240.36	253.30	252.74	252.26
A-2	5200	2.00	240.36	253.30	252.74	252.26
A-2	4400	-0.08	240.36	253.30	252.74	252.26
A-2	3200	-3.98	240.36	253.30	252.74	252.26
A-2	2600	-0.26	240.36	253.30	252.74	252.26
A-2	2089.779	-2.39	240.36	253.30	252.74	252.26
Main Res Reach-4	5900	44791.96	240.36	253.30	252.74	252.26
Main Res Reach-4	5400	44789.37	240.36	253.30	252.74	252.26
Main Res Reach-5	3800	57134.82	240.36	253.30	252.74	252.26
Main Res Reach-5	2200	57131.14	240.36	253.30	252.73	252.25
Main Res Reach-5	1000	57130.71	240.36	253.29	252.72	252.23

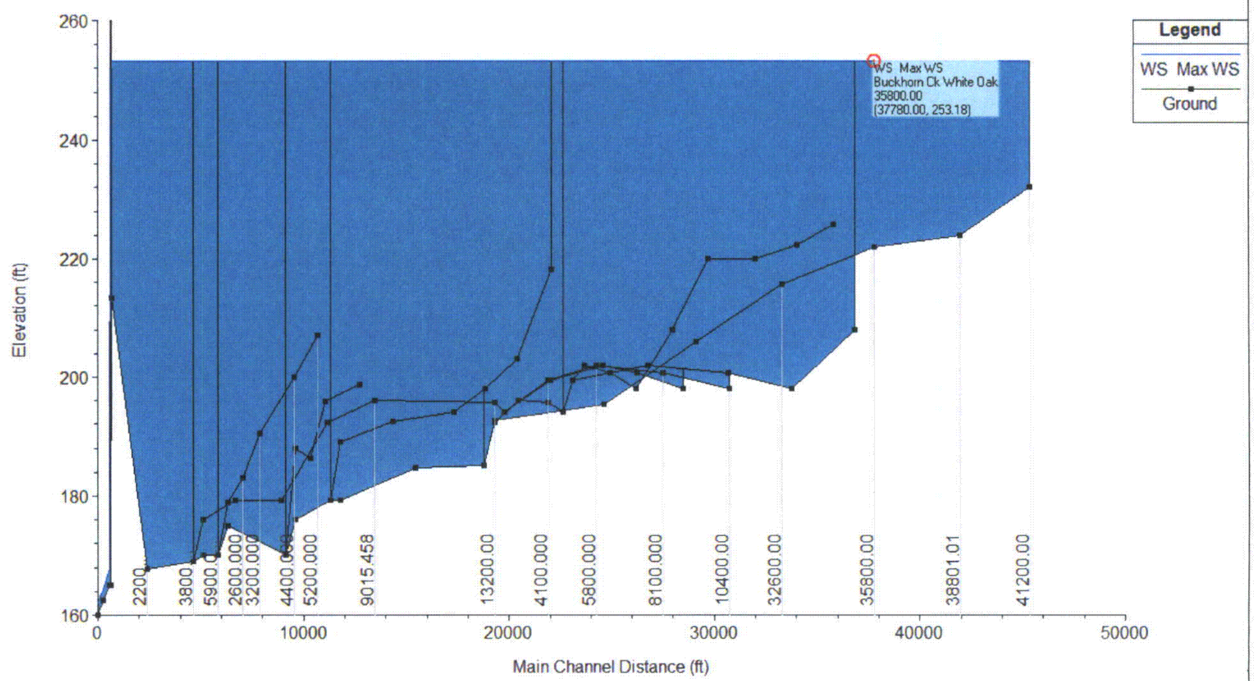
Figure 7: Comparison of PMF Stillwater Elevation for Various Scenarios of Option 1



Option 2 Results

Figure 8 and **Table 5** present the profile plots for various scenarios considered for Option 2. **Figure 9** compares these profiles along with the initial stillwater elevation for all locations from the downstream end of the Main Reservoir (Main Dam) to the upstream end of various creeks of the Main Reservoir, including Thomas Creek.

Figure 8: Profile Plots for Various Scenarios of Option 2

Scenario	**Profile Plot**
Lake Water Elevation at which Tainter Gate is Opened = 243 ft NGVD29.	

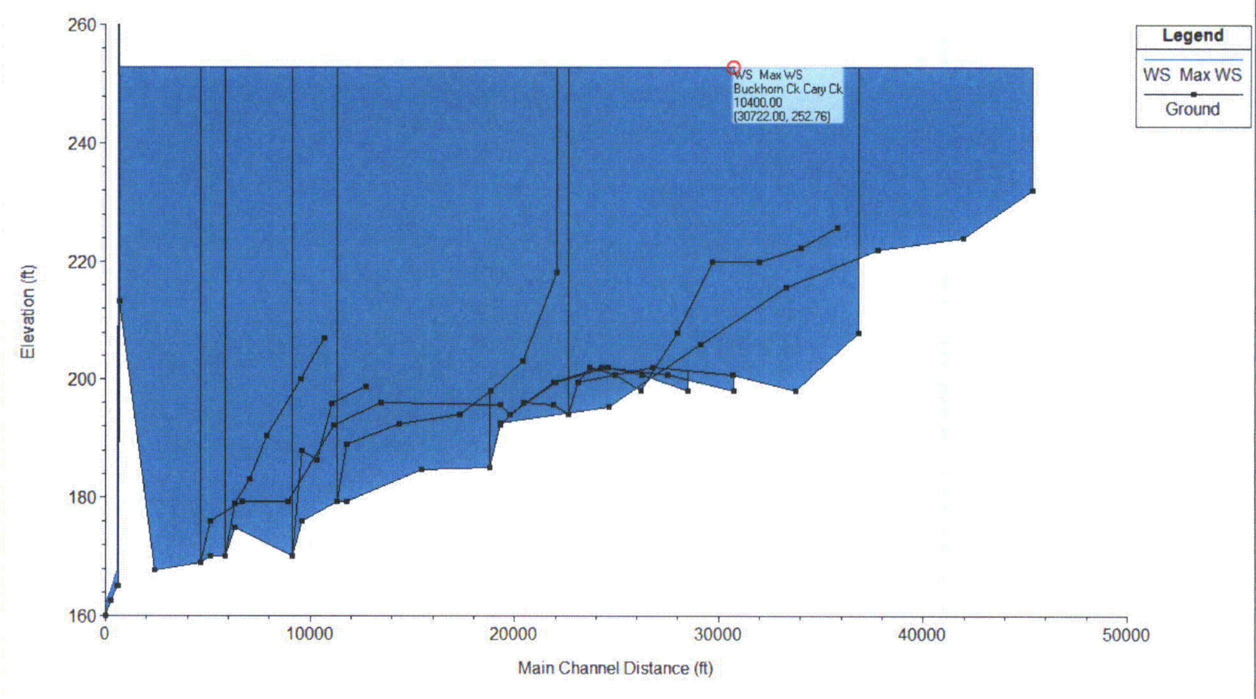
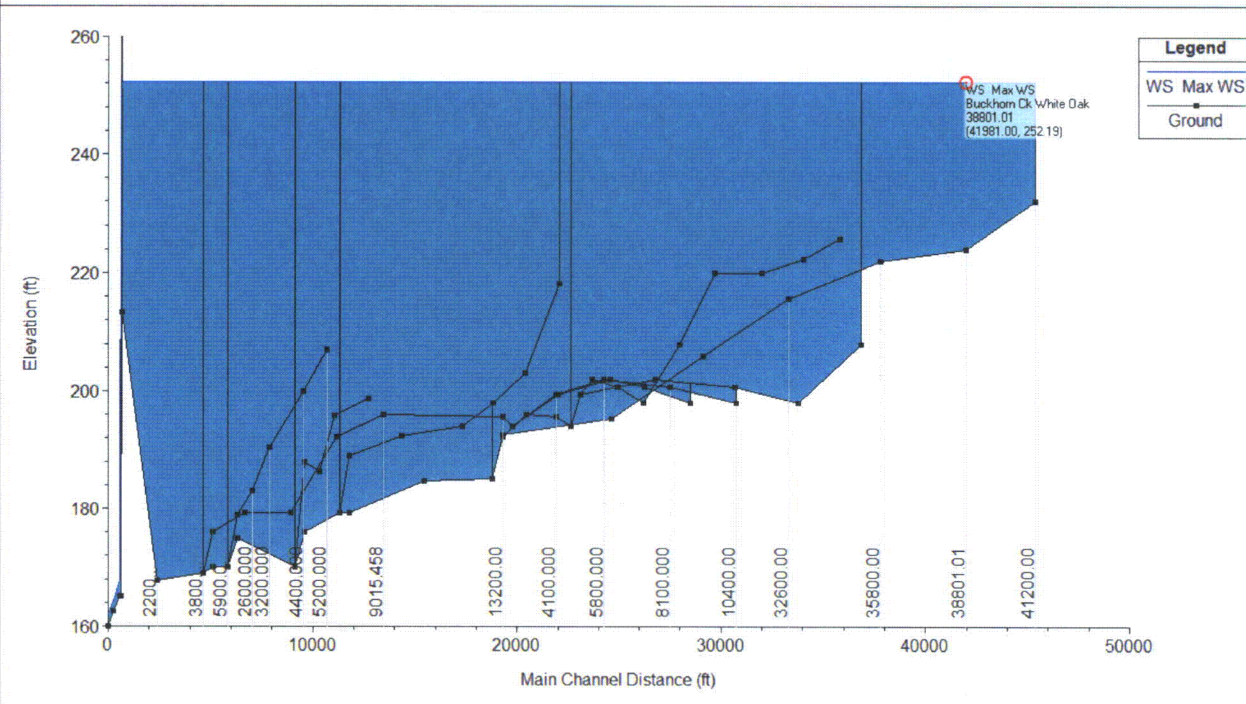
Length Lake Water Elevation at which Tainter Gate is Opened = 242 ft NGVD29.	
Lake Water Elevation at which Tainter Gate is Opened = 241 ft NGVD29.	

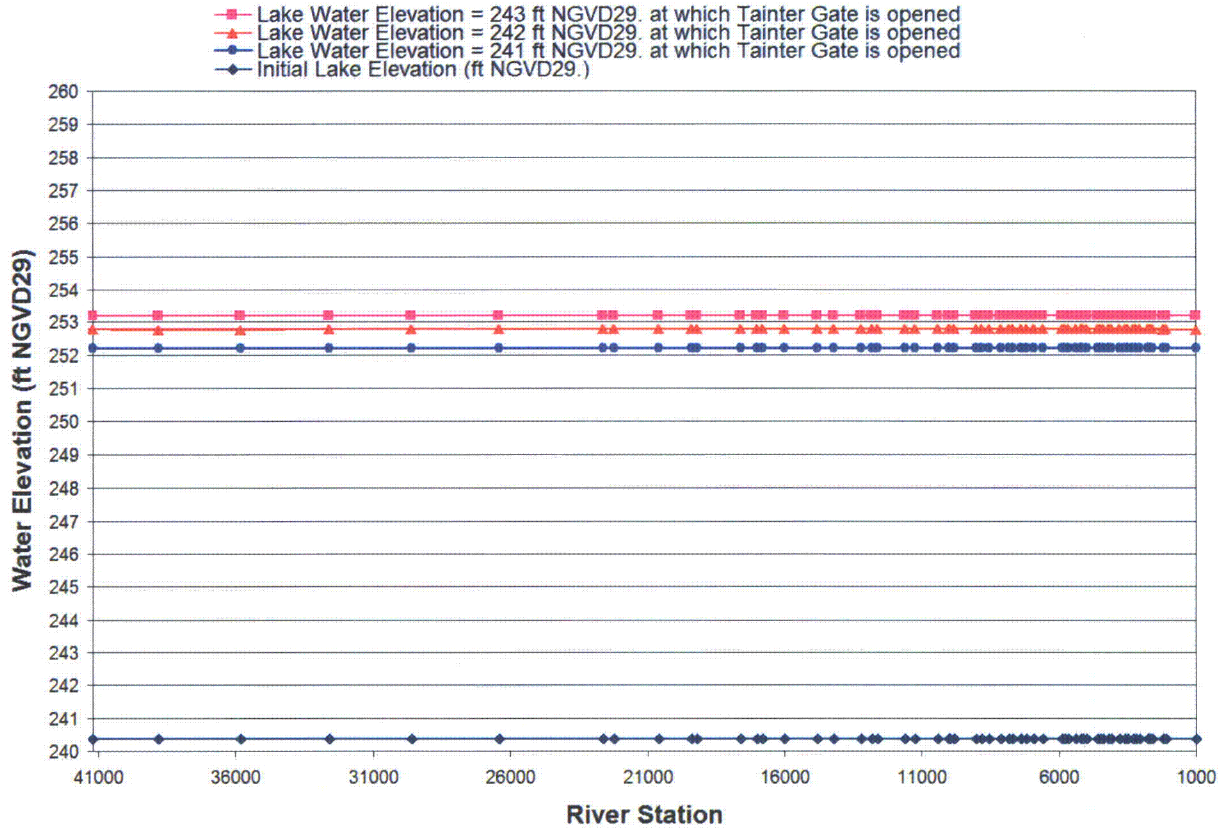
Table 5: PMF Profiles for Various Scenarios of Option 2

Reach	River Station	Flow (cfs)	Initial Lake Elevation (ft NGVD29)	Lake Water Elevation = 243 ft NGVD29 at which Tainter Gate is Opened	Lake Water Elevation = 242 ft NGVD29 at which Tainter Gate is Opened	Lake Water Elevation = 241 ft NGVD29 at which Tainter Gate is Opened
Tom Jack Ck	10000	2470.61	240.36	253.18	252.76	252.20
Tom Jack Ck	8800	2471.02	240.36	253.18	252.76	252.20
Tom Jack Ck	7700	2467.67	240.36	253.18	252.76	252.20
Tom Jack Ck	6600	2467.20	240.36	253.18	252.76	252.20
Tom Jack Ck	4500	2471.90	240.36	253.18	252.76	252.20
Tom Jack Ck	2661.272	2450.20	240.36	253.18	252.76	252.20
LittleWhiteOak-C	17000	1457.44	240.36	253.18	252.76	252.20
LittleWhiteOak-C	14800	1453.65	240.36	253.18	252.76	252.20
LittleWhiteOak-C	12600	1453.52	240.36	253.18	252.76	252.20
LittleWhiteOak-C	9800	1457.84	240.36	253.18	252.76	252.20
LittleWhiteOak-C	7200	1477.89	240.36	253.18	252.76	252.20
Thomas Ck	11242.48	216.92	240.36	253.18	252.76	252.20
Thomas Ck	9991.82	216.41	240.36	253.18	252.76	252.20
Thomas Ck	8539.703	210.90	240.36	253.18	252.76	252.20
Thomas Ck	6905.036	202.81	240.36	253.18	252.76	252.20
Thomas Ck	5660.595	192.20	240.36	253.18	252.76	252.20
Thomas Ck	4400	182.82	240.36	253.18	252.76	252.20
Thomas Ck	3500	175.69	240.36	253.18	252.76	252.20
Thomas Ck	2600	171.37	240.36	253.18	252.76	252.20
Thomas Ck	2205.884	168.13	240.36	253.18	252.76	252.20
Thomas -White CK	5118.695	1646.03	240.36	253.18	252.76	252.20
Thomas -White CK	4600	1631.91	240.36	253.18	252.76	252.20
Thomas -White CK	3578.48	1650.99	240.36	253.18	252.76	252.20
Thomas -White CK	2732.033	1639.85	240.36	253.18	252.76	252.20
White Oak	41200	20944.84	240.36	253.18	252.76	252.20
White Oak	38801.01	19429.99	240.36	253.17	252.75	252.19
White Oak	35800	19447.94	240.36	253.18	252.75	252.19
White Oak	32600	19536.05	240.36	253.18	252.76	252.19
White Oak	29600	19594.27	240.36	253.18	252.76	252.20
White Oak	26400	19648.73	240.36	253.18	252.76	252.20
White Oak	22600	19697.22	240.36	253.18	252.76	252.20
Main Res Reach-1	19200	21337.07	240.36	253.18	252.76	252.20
Main Res Reach-1	16800	21325.02	240.36	253.18	252.76	252.20
Main Res Reach-1	14200	42366.45	240.36	253.18	252.76	252.20
Main Res Reach-2	12800	44816.64	240.36	253.18	252.76	252.20
Main Res Reach-2	11600	44775.68	240.36	253.18	252.76	252.20
A-3	5000	2.00	240.36	253.18	252.76	252.20
A-3	3800	-0.07	240.36	253.18	252.76	252.20

Table 5: PMF Profiles for Various Scenarios of Option 2

Reach	River Station	Flow (cfs)	Initial Lake Elevation (ft NGVD29)	Lake Water Elevation = 243 ft NGVD29 at which Tainter Gate is Opened	Lake Water Elevation = 242 ft NGVD29 at which Tainter Gate is Opened	Lake Water Elevation = 241 ft NGVD29 at which Tainter Gate is Opened
A-3	3289.593	-0.60	240.36	253.18	252.76	252.20
A-3	2778.718	3.93	240.36	253.18	252.76	252.20
Main Res Reach-3	9800	44779.61	240.36	253.18	252.76	252.20
Main Res Reach-3	7800	44794.35	240.36	253.18	252.76	252.20
Buckhorn Ck-1	22200	11136.05	240.36	253.17	252.76	252.20
Buckhorn Ck-1	20600	11154.48	240.36	253.17	252.76	252.20
Buckhorn Ck-1	19400	11167.05	240.36	253.18	252.76	252.20
Buckhorn Ck-1	17600	11196.67	240.36	253.18	252.76	252.20
Buckhorn Ck-1	16000	11171.41	240.36	253.18	252.76	252.20
Cary Ck	10400	1211.55	240.36	253.18	252.76	252.20
Cary Ck	8100	1207.69	240.36	253.18	252.76	252.20
Cary Ck	5800	1202.94	240.36	253.18	252.76	252.20
Cary Ck	4100	1202.60	240.36	253.18	252.76	252.20
Cary Ck	2600	1196.97	240.36	253.18	252.76	252.20
Buckhorn Ck -2	13200	12368.37	240.36	253.18	252.76	252.20
Buckhorn Ck -2	9015.458	12360.96	240.36	253.18	252.76	252.20
Buckhorn Ck -2	7400	12358.13	240.36	253.18	252.76	252.20
Buckhorn Ck -2	5800	12353.34	240.36	253.18	252.76	252.20
Buckhorn Ck -2	4200	12348.79	240.36	253.18	252.76	252.20
Buckhorn Ck -2	3070.306	12345.45	240.36	253.18	252.76	252.20
A-2	5200	2.00	240.36	253.18	252.76	252.20
A-2	4400	-0.08	240.36	253.18	252.76	252.20
A-2	3200	-3.98	240.36	253.18	252.76	252.20
A-2	2600	-0.26	240.36	253.18	252.76	252.20
A-2	2089.779	-2.39	240.36	253.18	252.76	252.20
Main Res Reach-4	5900	44791.96	240.36	253.18	252.76	252.20
Main Res Reach-4	5400	44789.37	240.36	253.18	252.76	252.20
Main Res Reach-5	3800	57134.82	240.36	253.18	252.76	252.20
Main Res Reach-5	2200	57131.14	240.36	253.18	252.75	252.20
Main Res Reach-5	1000	57130.71	240.36	253.17	252.75	252.19

Figure 9: Comparison of PMF Stillwater Elevation for Various Scenarios of Option 2



Summary

Table 6 summarizes the Stillwater PMF elevations in the Main Reservoir for various scenarios considered in Options 1 and 2 for potential spillway design modification in order to protect the safety-related facilities at HNP, HAR 2, and HAR 3.

Table 6: Summary of PMF Stillwater Elevations for Options 1 and 2

OPTION 1		OPTION 2	
Length of Emergency Spillway (ft)	**Main Reservoir PMF Elevation (ft NGVD29)**	**Lake Water Elevation (ft NGVD29) at which Tainter Gate is Opened**	**Main Reservoir PMF Elevation (ft NGVD29)**
Initial Condition	240.36	Initial Condition	240.36
400	253.33	243	253.18
500	252.74	242	252.76
600	252.26	241	252.20

Associated HAR COL Application Revisions:

Associated HAR COL Application revisions are detailed in HAR FSAR RAI 02.04.03-4 (supplemental response to HAR-RAI-LTR-023; NPD-NRC-2009-230).

Attachments / Enclosures:

None.

NRC Letter No.: HAR-RAI-LTR-009

NRC Letter Date: September 17, 2008

NRC Review of Final Safety Analysis Report

NRC RAI NUMBER: 02.04.03-3

Text of NRC RAI:

Please describe how a peaking coefficient of 0.75 results in a conservative PMF estimate.

PGN RAI ID #: H-0489

PGN Response to NRC RAI:

After Progress Energy Carolina, Inc.'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 revised response to this RAI and RAI 02.04.03-1 and submitted a response to new RAI 02.04.03-4 on April 1, 2009, in order to accomplish 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 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. The two PMF strategies are as follows:

- 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 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 was also incorporated into this response. The entire response is intended to supersede and replace the previous responses, which were submitted to the NRC by letters dated October 31, 2008 (Serial NPD-NRC-2008-055) and April 1, 2009 (NPD-NRC-2009-050):

- Increase the Unit Hydrograph used in the HEC-HMS Model by 25 percent – The current unit hydrograph will be manually increased by 25 percent while changing the hydrograph lag time accordingly to maintain unit volume. The new unit hydrograph will be used in the HEC-HMS model to account for a conservative peaking coefficient in the PMF analysis.

In addition, peak flows generated by probable maximum precipitation (PMP) storms and associated with a unit hydrograph not peaked by 25 percent were also evaluated using peak flow equations developed by the U.S. Geological Survey (USGS) for rural basins in North Carolina (USGS, *Estimating the Magnitude and Frequency of Floods in Rural Basins of North Carolina—Revised,* Water-Resources Investigations Report 01-4207, Raleigh, North Carolina, 2001). 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 Final Safety Analysis Report (FSAR) HEC-HMS model for various storm events (**Table 1**) without making any change in the HEC-HMS parameters used for the PMP storm event.

Table 1: Comparison of Peak Flows determined using the USGS Equations and the FSAR 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%

Notes:
cfs = cubic foot per second

As shown in **Table 1**, the estimated magnitude of peak flow events generated by the FSAR HEC-HMS model exceeds the corrected peak flows (Col-4) 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 peak flows presented in the FSAR analysis are conservative. In other words, these can be considered as the implicit peaking factors.

In an effort to further address the concerns expressed by RAI 02.04.03-3, the following additional assessment was performed to determine the effect of applying peaking factors on PMF elevation. This assessment is an extension of the base method described in the FSAR, as explained below.

In order to represent the PMF conditions according to the recommendation of *ER 1110-8-2(FR)* (USACE, *Engineering and Design, Inflow Design Floods for Dams and Reservoirs, Engineer Regulation 1110-8-2(FR)*, USACE, Washington, D.C., 1991), the 1-hour base unit hydrographs developed for the FSAR analysis using the Snyder method were peaked by 25 percent. That is, the unit hydrographs (**Figure 1**) 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 provided in **Table 2**. (Refer to **FSAR Table 2.4.3-219** for the 1-hour base unit hydrograph parameters.)

Figure 1: 1-Hour Base Unit Hydrographs from FSAR

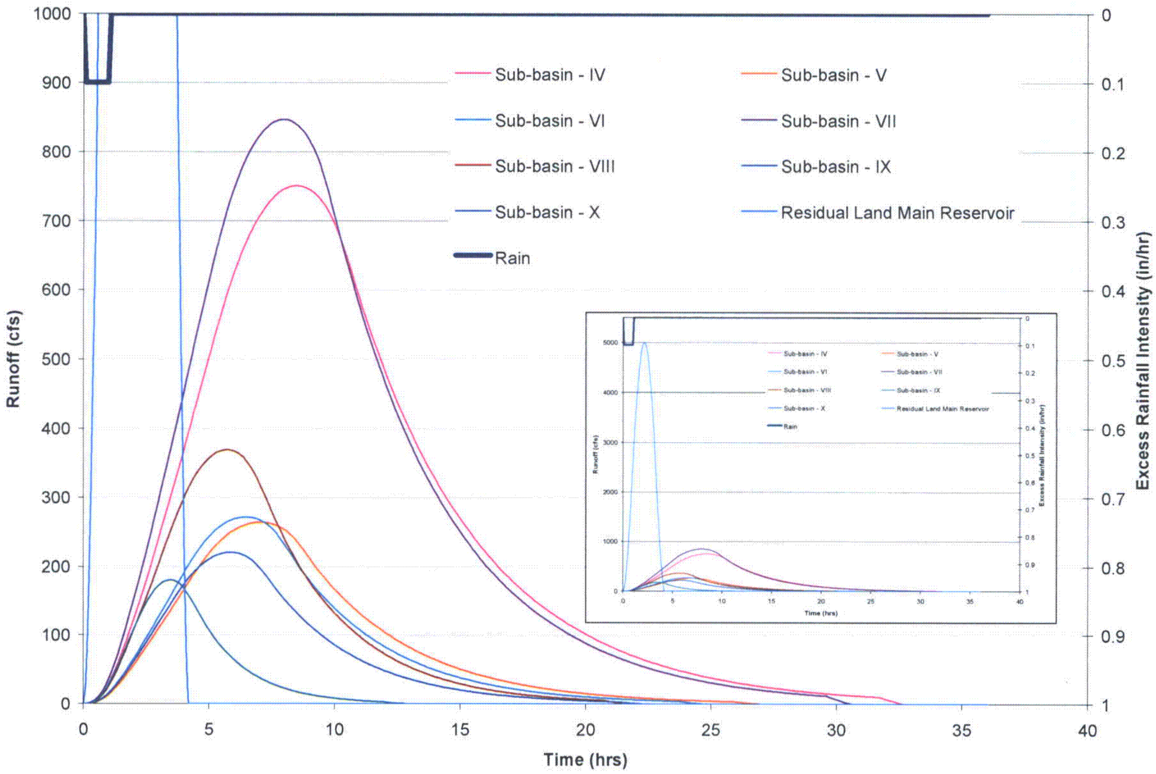


Table 2: Unit Hydrograph Parameters with Peaking and without Peaking

Item	Sub-Basin IV	Sub-Basin V	Sub-Basin VI	Sub-Basin VII	Sub-Basin VIII	Sub-Basin IX	Sub-Basin X	Residual Area
Unit Hydrograph Parameters without any Peaking								
Time to Peak, t_p (hr)	8.50	7.10	6.50	8.00	5.70	3.50	5.90	2.20
Peak Flow, Q_p (cfs)	750	264	271	847	369	181	220	4992
Lag time, t_L (hr)	8.00	6.60	6.00	7.50	5.20	3.00	5.40	1.70

Item	Sub-Basin IV	Sub-Basin V	Sub-Basin VI	Sub-Basin VII	Sub-Basin VIII	Sub-Basin IX	Sub-Basin X	Residual Area
Unit Hydrograph Parameters with Peaking								
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

A comparison of the base unit hydrograph with the peaked unit hydrograph for Sub-basin X is shown on **Figure 2**, while the peaked unit hydrographs for all sub-basins are shown on **Figure 3**.

Figure 2: Base Unit Hydrograph vs. Peaked Unit Hydrograph for Subbasin X

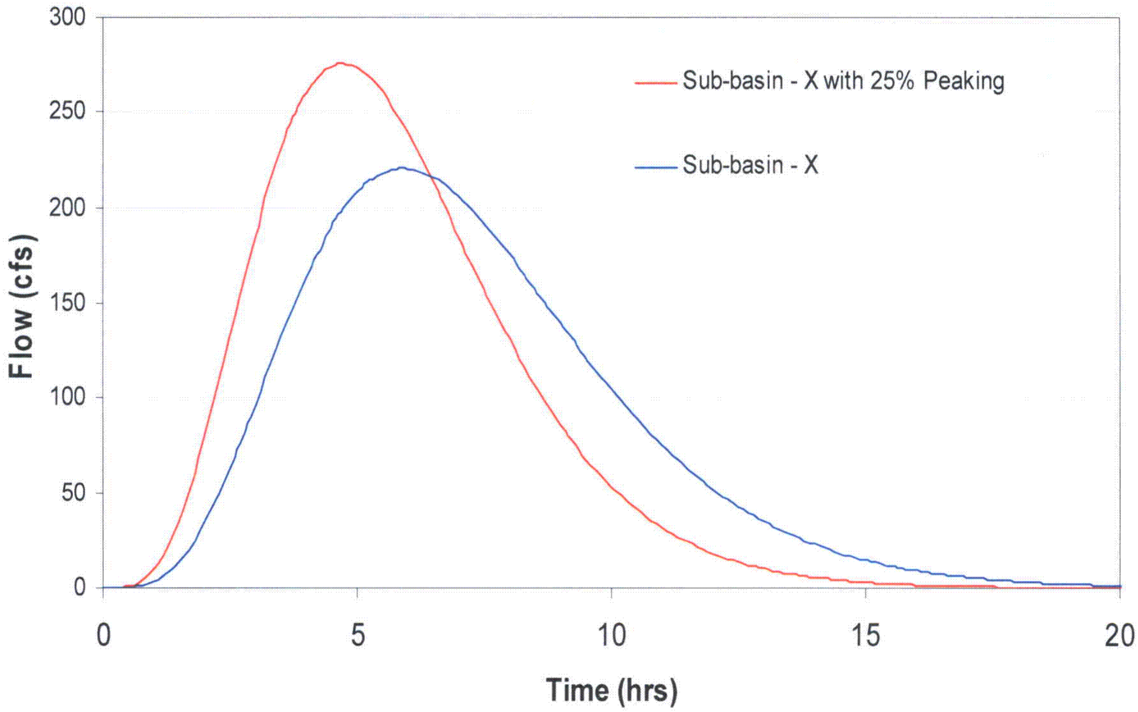
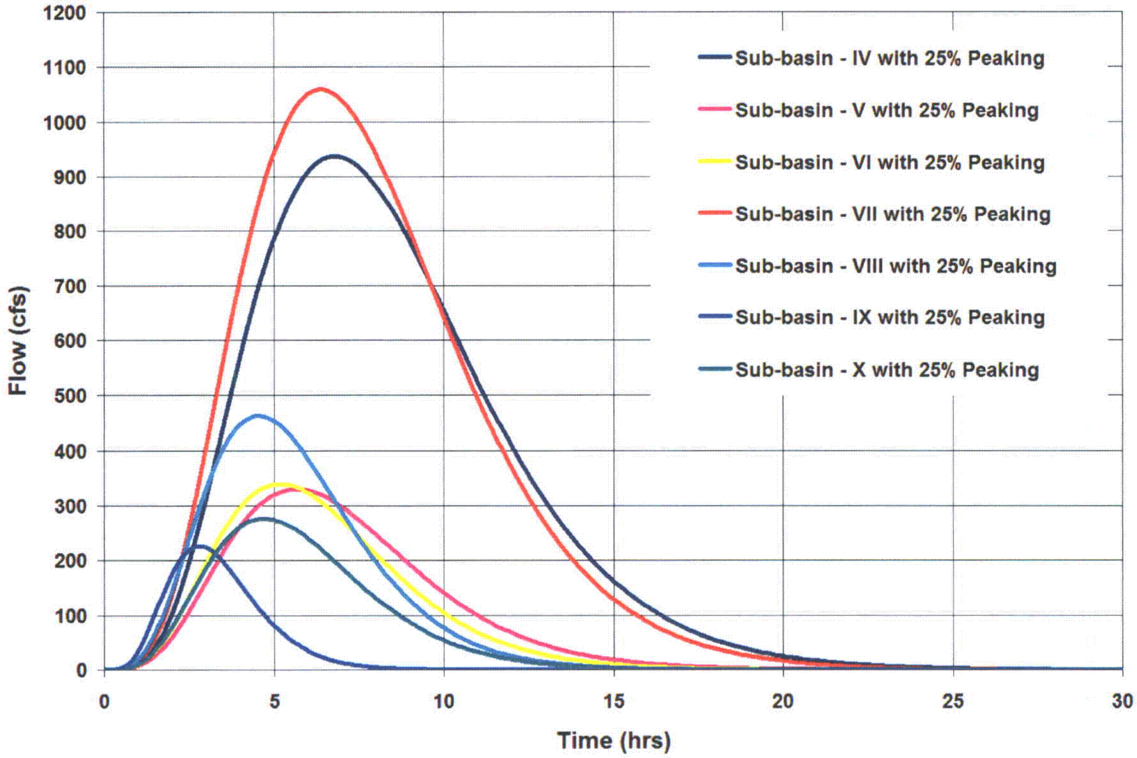


Figure 3: Peaked Unit Hydrographs



These revised unit hydrographs were incorporated in the HEC-HMS model in order to determine the peaking impact.

Option 1 Results

As previously discussed, in order to ensure protection of safety-related structures against flooding and dynamic effects of wave action due to wind-generated activity, some modifications were considered in the spillway design of the Main Dam. These modifications have been considered as Option 1 and Option 2. Option 1 consists of raising the existing open spillway to 240 feet NGVD29 in both spans and adding an emergency spillway with its crest at 243 feet NGVD29. In order to determine the length of emergency spillway, different scenarios using various emergency spillway lengths were evaluated. For Option 1, it was observed that the length of the emergency spillway should be at least 400 feet.

Tables 3 and **4** summarize the HEC-HMS model results for both the Auxiliary and Main Reservoirs considering 25 percent peaking and no peaking for the PMP event. These tables compare the PMF resulting from the base unit hydrographs from the FSAR model to the PMF resulting from the peaked unit hydrographs described above. **Figures 4** and **5** compare the inflows and outflows with 25 percent peaking versus no peaking for the Auxiliary and Main Reservoirs, respectively. Similarly, **Figures 6** and **7** compare the stillwater elevations during the PMF events in the Auxiliary and Main Reservoirs while considering 25 percent peaking versus no peaking. Based on these results, consideration of 25 percent peaking in the PMP event increases the PMF stillwater elevations by 0.3 and 0.4 foot in the Auxiliary and Main Reservoirs, respectively. The impact of peaking is less than 0.2 percent and thus is negligible. Therefore, the peaking coefficient of 0.75 gives a conservative estimate of the PMF elevations in the Auxiliary and Main Reservoirs.

Table 3: Summary of HEC-HMS PMF Output, With Peaking vs. Without Peaking for the Auxiliary Reservoir for Option 1

Metric	Scenario 1		Scenario 2		Scenario 3	
	Without Peaking	With Peaking	Without Peaking	With Peaking	Without Peaking	With Peaking
Peak Inflow (cfs)	6,242	6,961	6,242	6,961	6,242	6,961
Peak Outflow (cfs)	5,581	6,236	5,581	6,235	5,580	6,235
Total Inflow (in)	66.06	66.06	66.06	66.06	66.06	66.06
Total Outflow (in)	66.06	66.06	66.06	66.06	66.06	66.06
Peak Storage (ac-ft)	6,663	6,795	6,673	6,805	6,684	6,816
Peak Elevation (feet NGVD29)	256.23	256.53	256.25	256.55	256.28	256.58
Notes: Scenario 1: Length of Emergency Spillway at the Main Dam = 400 ft Scenario 2: Length of Emergency Spillway at the Main Dam = 500 ft Scenario 3: Length of Emergency Spillway at the Main Dam = 600 ft						

Table 4: Summary of HEC-HMS PMF Output, With Peaking vs. Without Peaking for the Main Reservoir for Option 1

Metric	Scenario 1		Scenario 2		Scenario 3	
	Without Peaking	With Peaking	Without Peaking	With Peaking	Without Peaking	With Peaking
Peak Inflow (cfs)	110,600	125,022	110,597	125,020	110,597	125,021
Peak Outflow (cfs)	40,262	42,479	44,956	47,740	48,971	52,283
Total Inflow (in)	52.16	52.16	52.16	52.16	52.16	52.16
Total Outflow (in)	45.14	45.17	45.55	45.58	45.81	45.83
Peak Storage (ac-ft)	285,755	289,550	279,765	283,705	274,764	278,772
Peak Elevation (feet NGVD29)	252.61	252.96	252.04	252.42	251.56	251.94
Notes: Scenario 1: Length of Emergency Spillway at the Main Dam = 400 ft Scenario 2: Length of Emergency Spillway at the Main Dam = 500 ft Scenario 3: Length of Emergency Spillway at the Main Dam = 600 ft						

Figure 4a: Comparison of Auxiliary Reservoir PMF Inflows and Outflows, Peaking vs. No Peaking, Option 1, Scenario 1 (400 ft Emergency Spillway at the Main Dam)

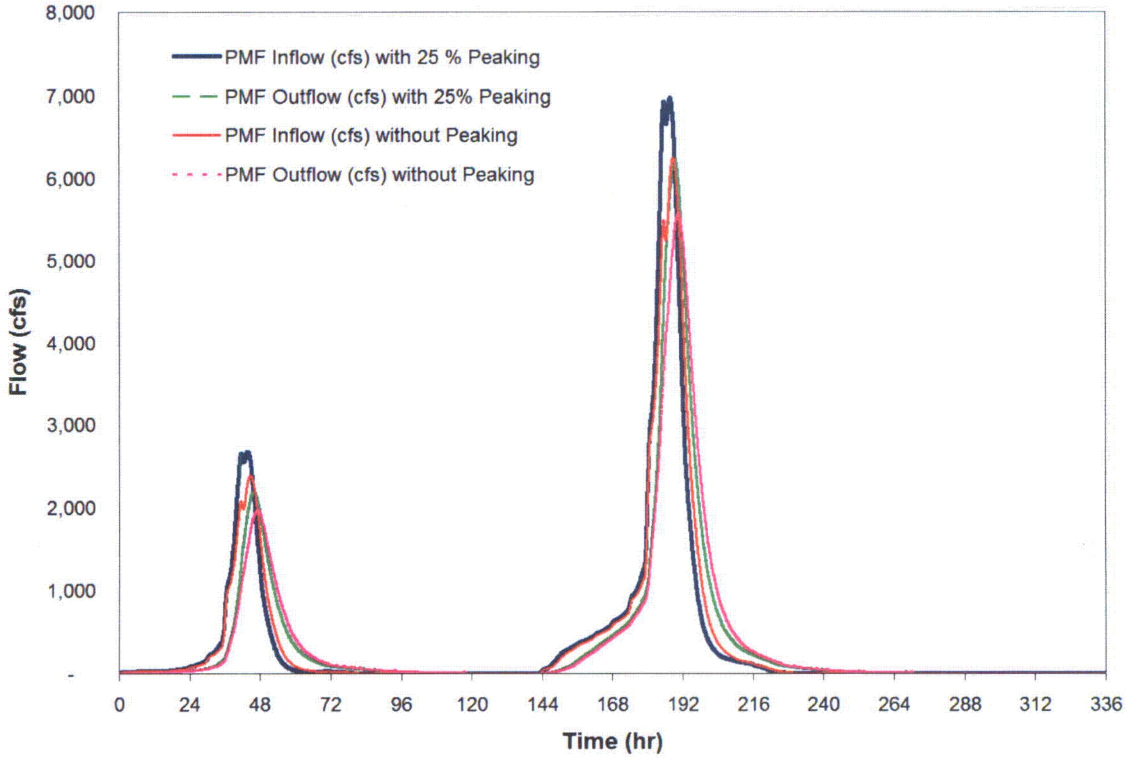


Figure 4b: Comparison of Auxiliary Reservoir PMF Inflows and Outflows, Peaking vs. No Peaking, Option 1, Scenario 2 (500 ft Emergency Spillway at the Main Dam)

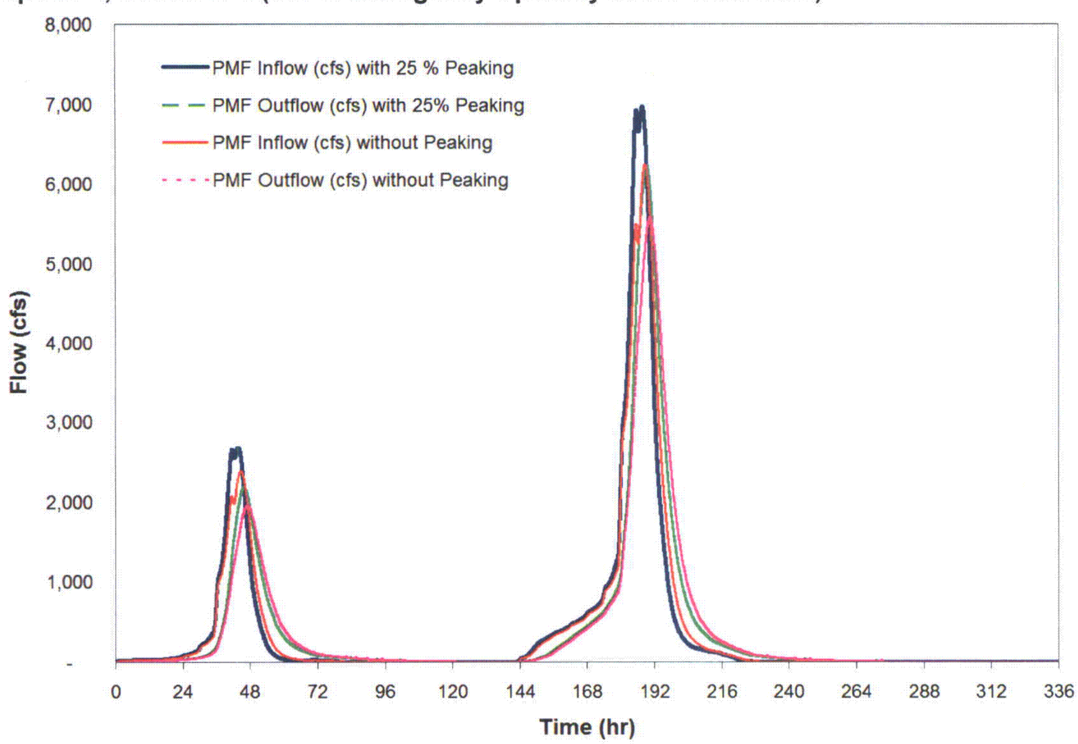


Figure 4c: Comparison of Auxiliary Reservoir PMF Inflows and Outflows, Peaking vs. No Peaking, Option 1, Scenario 3 (600 ft Emergency Spillway at the Main Dam)

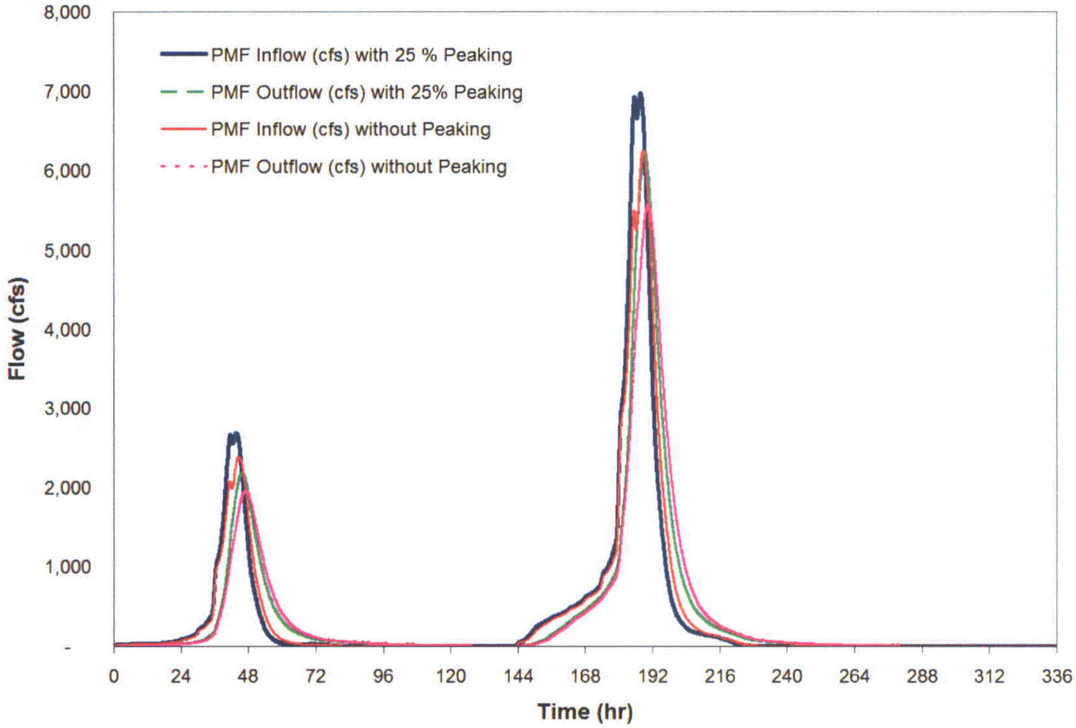


Figure 5a: Comparison of Main Reservoir PMF Inflows and Outflows, Peaking vs. No Peaking, Option 1, Scenario 1 (400 ft Emergency Spillway at the Main Dam)

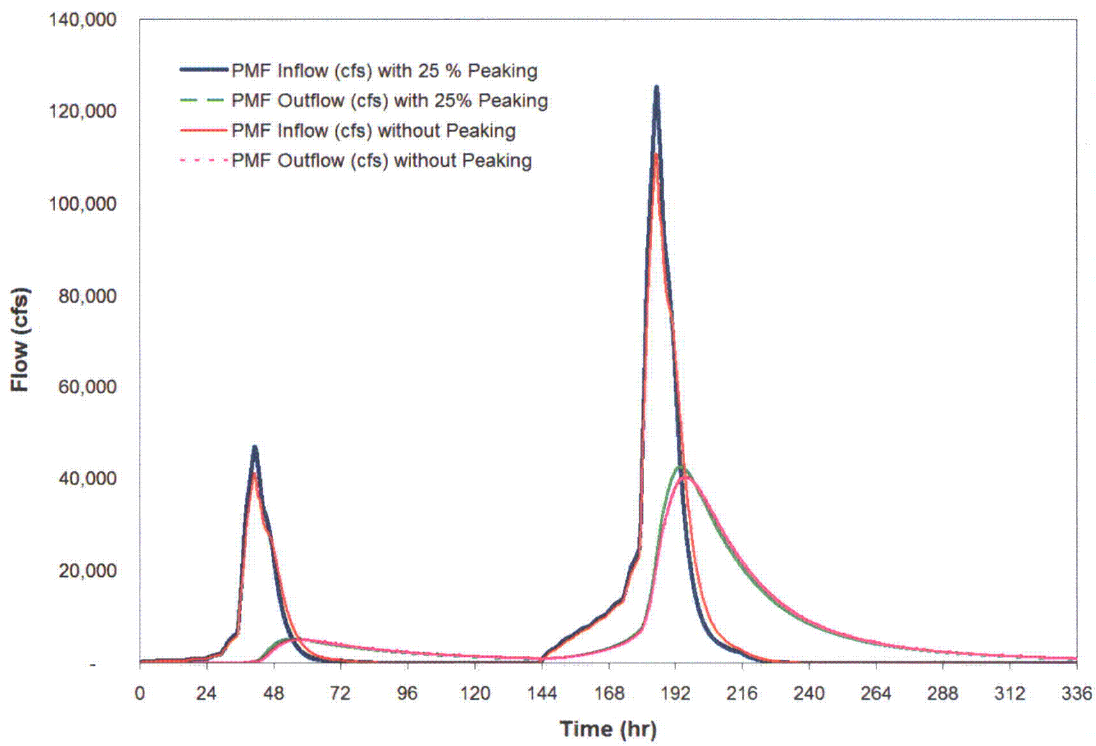


Figure 5b: Comparison of Main Reservoir PMF Inflows and Outflows, Peaking vs. No Peaking, Option 1, Scenario 2 (500 ft Emergency Spillway at the Main Dam)

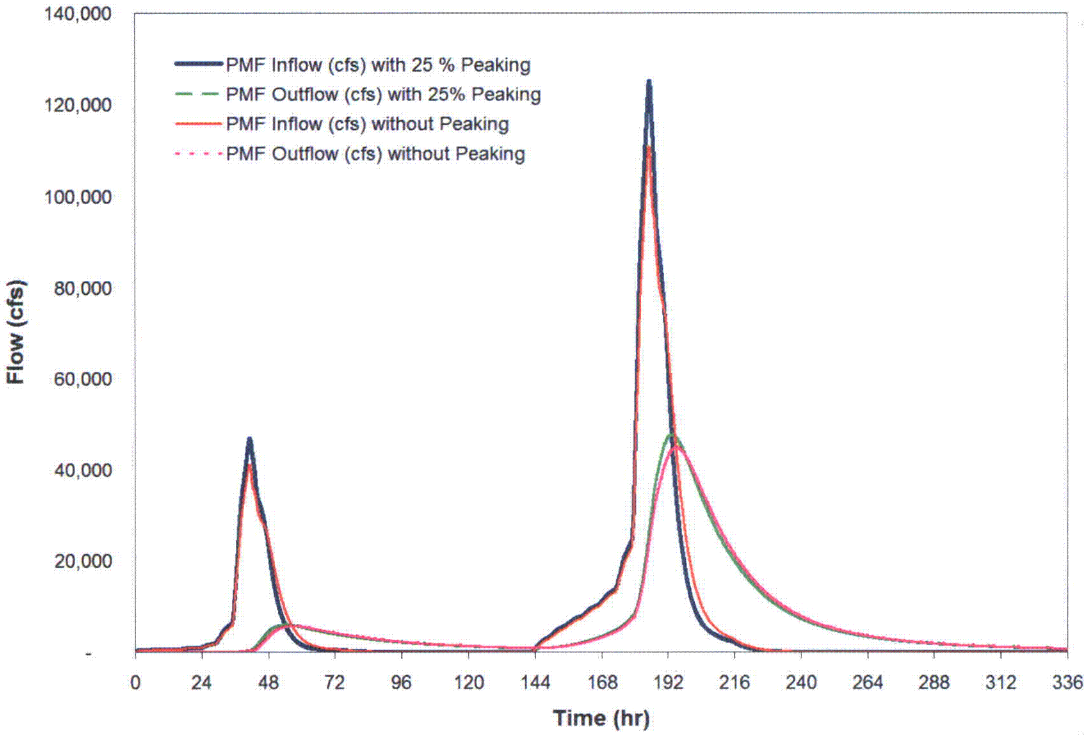


Figure 5c: Comparison of Main Reservoir PMF Inflows and Outflows, Peaking vs. No Peaking, Option 1, Scenario 3 (600 ft Emergency Spillway at the Main Dam)

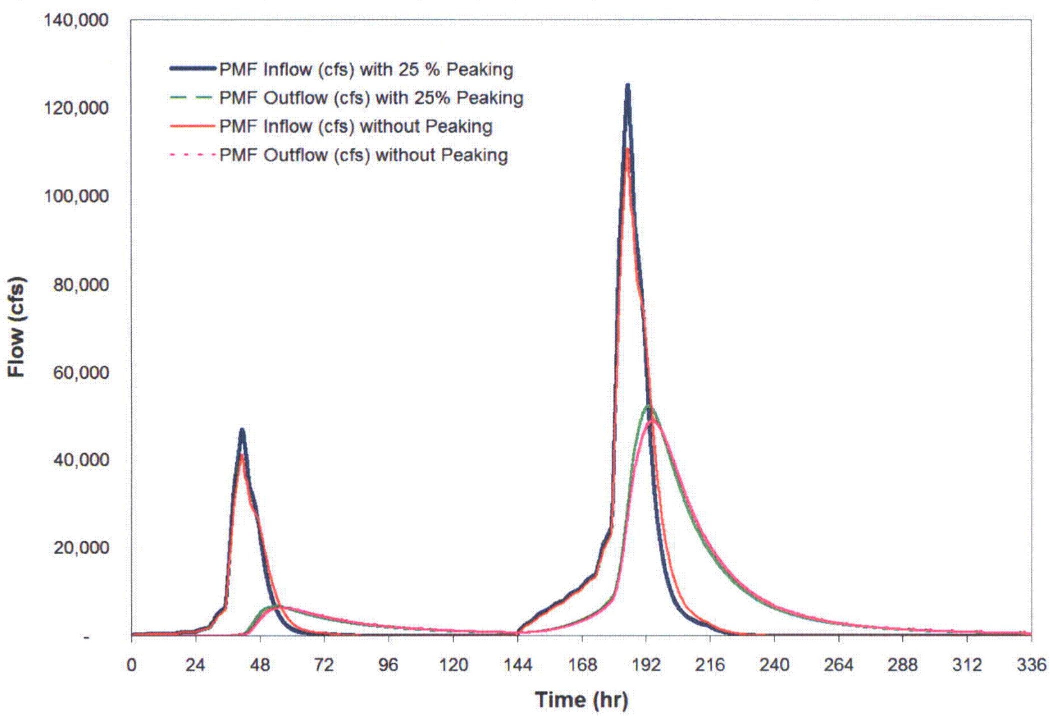


Figure 6a: Comparison of PMF Stillwater Elevation in Auxiliary Reservoir, Peaking vs. No Peaking, Option 1, Scenario 1 (400 ft Emergency Spillway at the Main Dam)

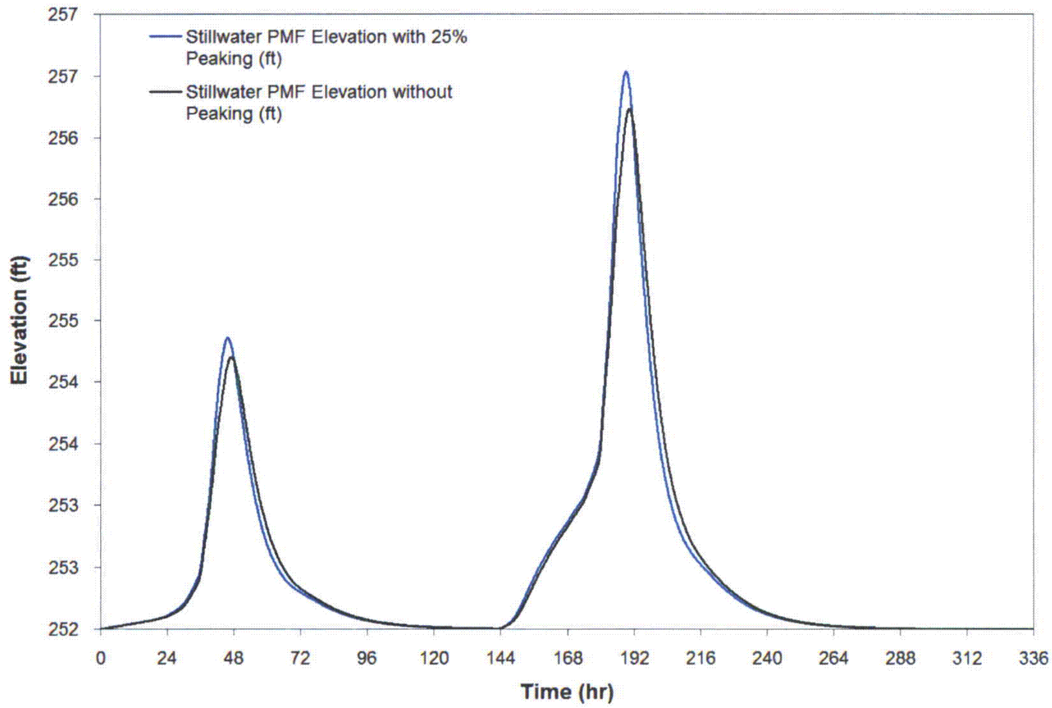


Figure 6b: Comparison of PMF Stillwater Elevation in Auxiliary Reservoir, Peaking vs. No Peaking, Option 1, Scenario 2 (500 ft Emergency Spillway at the Main Dam)

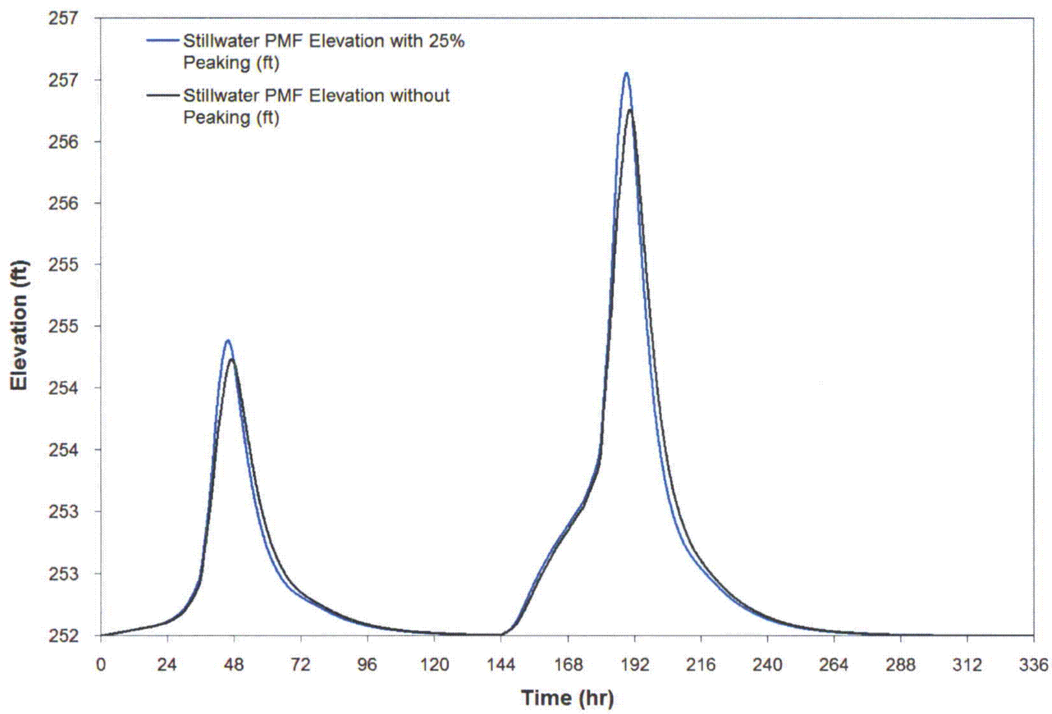


Figure 6c: Comparison of PMF Stillwater Elevation in Auxiliary Reservoir, Peaking vs. No Peaking, Option 1, Scenario 3 (600 ft Emergency Spillway at the Main Dam)

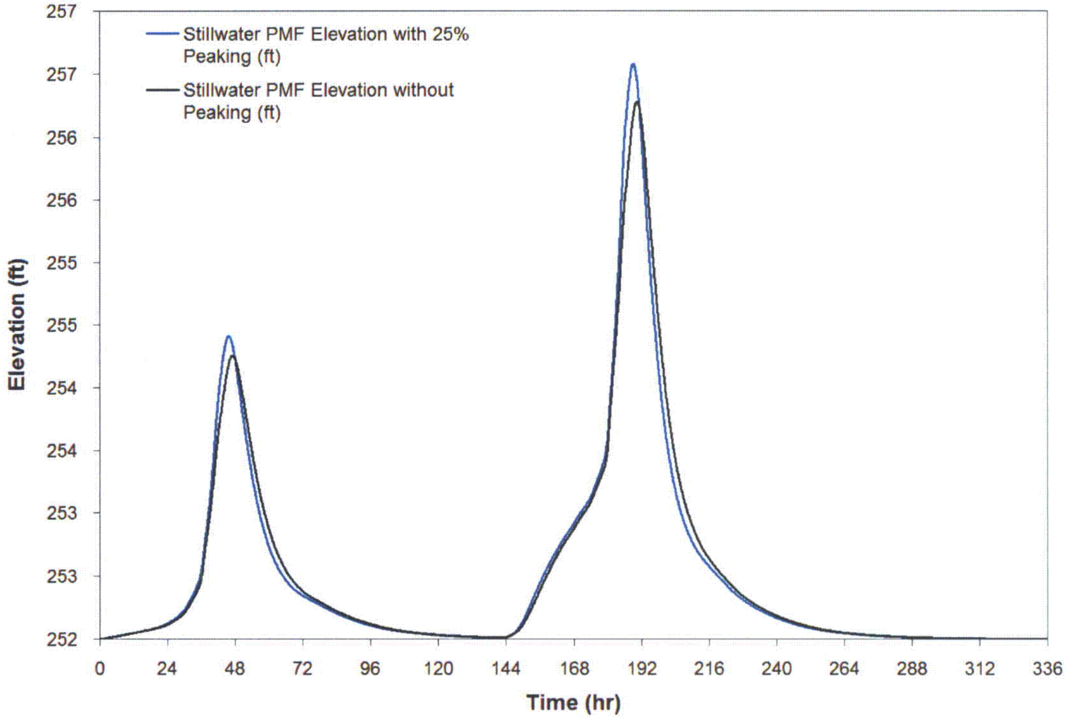


Figure 7a: Comparison of PMF Stillwater Elevation in Main Reservoir, Peaking vs. No Peaking, Option 1, Scenario 1 (400 ft Emergency Spillway at the Main Dam)

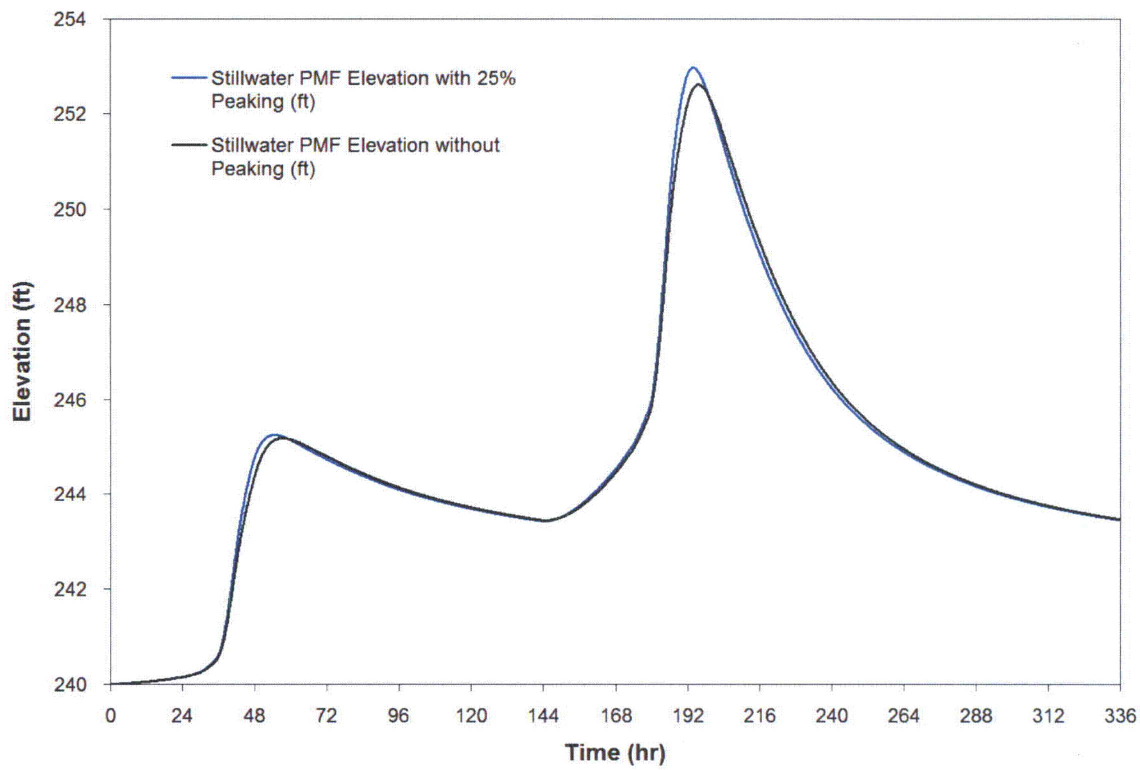


Figure 7b: Comparison of PMF Stillwater Elevation in Main Reservoir, Peaking vs. No Peaking, Option 1, Scenario 2 (500 ft Emergency Spillway at the Main Dam)

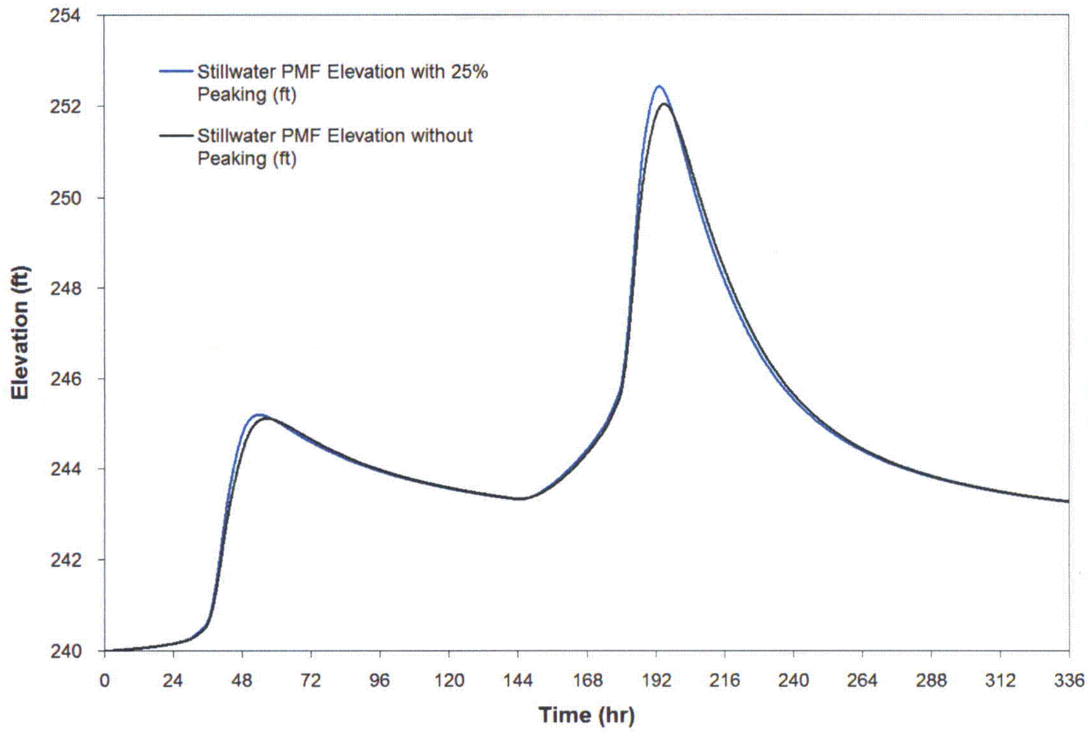
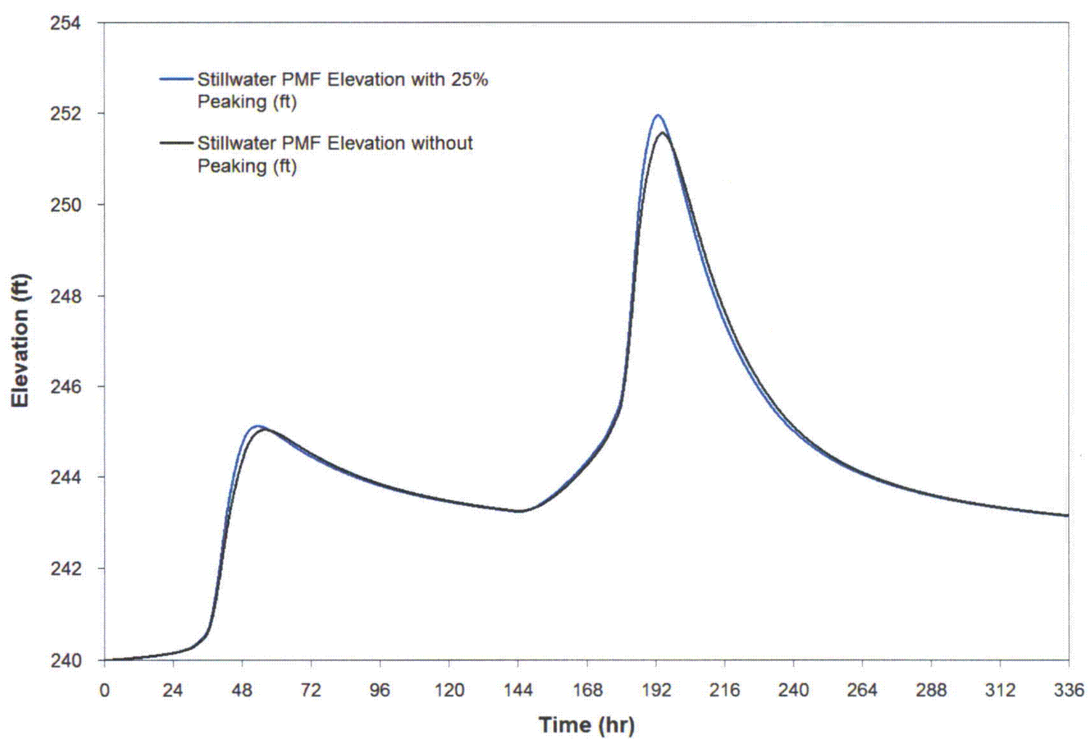


Figure 7c: Comparison of PMF Stillwater Elevation in Main Reservoir, Peaking vs. No Peaking, Option 1, Scenario 3 (600 ft Emergency Spillway at the Main Dam)



Option 2 Results

As previously discussed, in order to ensure protection of safety-related structures against flooding and dynamic effects of wave action due to wind-generated activity, some modifications were considered in the spillway design of the Main Dam. These modifications have been considered as Option 1 and Option 2. Option 2 consists of raising the existing open spillway to 240 feet NGVD29 in one span and installing a Tainter gate in the second span with a spillway crest at 220 feet NGVD29. In order to determine the upstream water elevation at which the Tainter gate is completely opened, different scenarios were evaluated using various lake level target elevations to begin opening the Tainter gate. For Option 2, it was observed that the Tainter gate needs to be opened before the lake water elevation exceeds 243 feet NGVD29.

Tables 5 and **6** provide a summary of the HEC-HMS model results for both the Auxiliary and Main Reservoirs, considering 25 percent peaking and no peaking for the PMP event. These tables compare the PMF resulting from the base unit hydrographs from the FSAR model to the PMF resulting from the peaked unit hydrographs described above. **Figures 8** and **9** compare the inflows and outflows with 25 percent peaking versus no peaking for the Auxiliary and Main Reservoirs, respectively. Similarly, **Figures 10** and **11** compare the stillwater elevations during the PMF events in the Auxiliary and Main Reservoirs while considering 25 percent peaking versus no peaking. Based on these results, consideration of 25 percent peaking in the PMP event increases the PMF stillwater elevations by approximately 0.3 foot in both the Auxiliary and Main Reservoirs. The impact of peaking is approximately 0.1 percent and is negligible. Therefore, the peaking coefficient of 0.75 gives a conservative estimate of the PMF elevations in the Auxiliary and Main Reservoirs.

Table 5: Summary of HEC-HMS PMF Output, With Peaking vs. Without Peaking for the Auxiliary Reservoir for Option 2

Metric	Scenario 1		Scenario 2		Scenario 3	
	Without Peaking	**With Peaking**	**Without Peaking**	**With Peaking**	**Without Peaking**	**With Peaking**
Peak Inflow (cfs)	6,242	6,961	6,242	6,961	6,242	6,961
Peak Outflow (cfs)	5,581	6,235	5,581	6,235	5,581	6,235
Total Inflow (in)	66.06	66.06	66.06	66.06	66.06	66.06
Total Outflow (in)	66.06	66.06	66.06	66.06	66.06	66.06
Peak Storage (ac-ft)	6,666	6,798	6,677	6,809	6689	6821
Peak Elevation (feet NGVD29)	256.24	256.54	256.26	256.56	256.29	256.59

Notes:
Scenario 1: Lake Water Elevation at which the Tainter Gate is opened = 243 feet NGVD29
Scenario 2: Lake Water Elevation at which the Tainter Gate is opened = 242 feet NGVD29
Scenario 3: Lake Water Elevation at which the Tainter Gate is opened = 241 feet NGVD29

Table 6: Summary of HEC-HMS PMF Output, With Peaking vs. Without Peaking for the Main Reservoir for Option 2

Metric	Scenario 1		Scenario 2		Scenario 3	
	Without Peaking	With Peaking	Without Peaking	With Peaking	Without Peaking	With Peaking
Peak Inflow (cfs)	110,598	125,020	110,597	125,020	110,597	125,021
Peak Outflow (cfs)	21,429	21,811	20,677	21,050	19962	20324
Total Inflow (in)	52.16	52.16	52.16	52.16	52.16	52.16
Total Outflow (in)	46.65	46.66	48.45	48.53	50.28	50.29
Peak Storage (ac-ft)	284,803	287,627	279,228	281,990	273908	276599
Peak Elevation (feet NGVD29)	252.52	252.78	251.99	252.25	251.47	251.73

Notes:
Scenario 1: Lake Water Elevation at which the Tainter Gate is opened = 243 feet NGVD29
Scenario 2: Lake Water Elevation at which the Tainter Gate is opened = 242 feet NGVD29
Scenario 3: Lake Water Elevation at which the Tainter Gate is opened = 241 feet NGVD29

Figure 8a: Comparison of Auxiliary Reservoir PMF Inflows and Outflows, Peaking vs. No Peaking, Option 2, Scenario 1 (Tainter Gate Opened at 243 feet NGVD29)

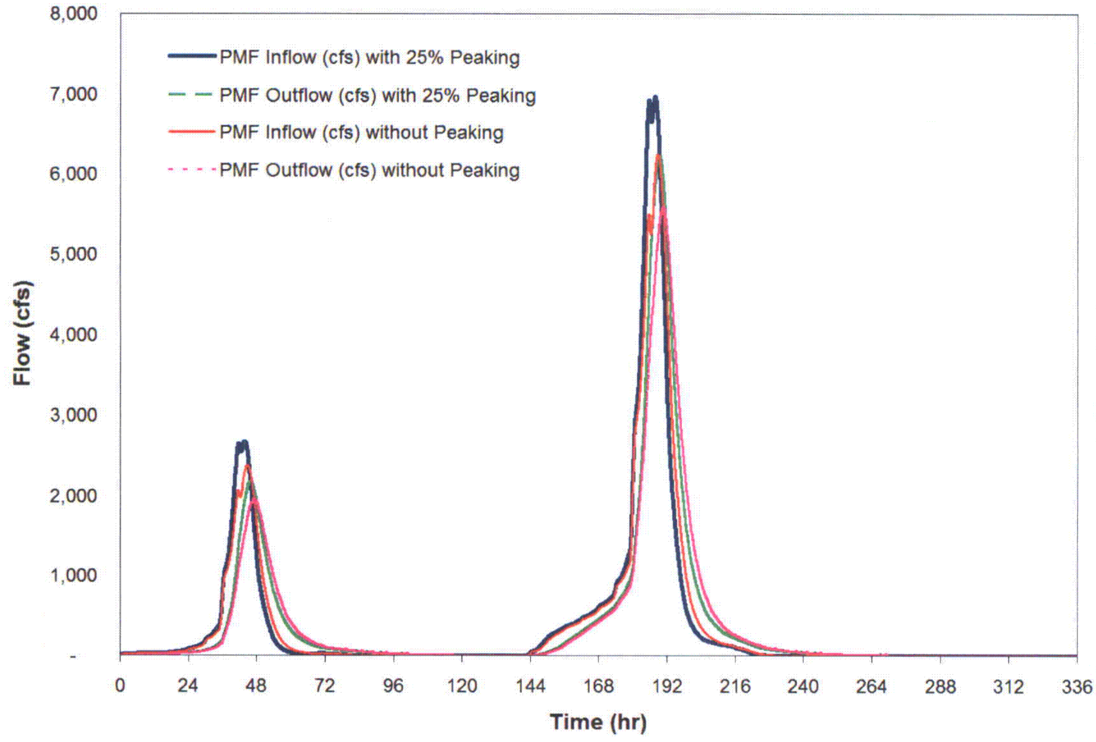


Figure 8b: Comparison of Auxiliary Reservoir PMF Inflows and Outflows, Peaking vs. No Peaking, Option 2, Scenario 2 (Tainter Gate Opened at 242 feet NGVD29)

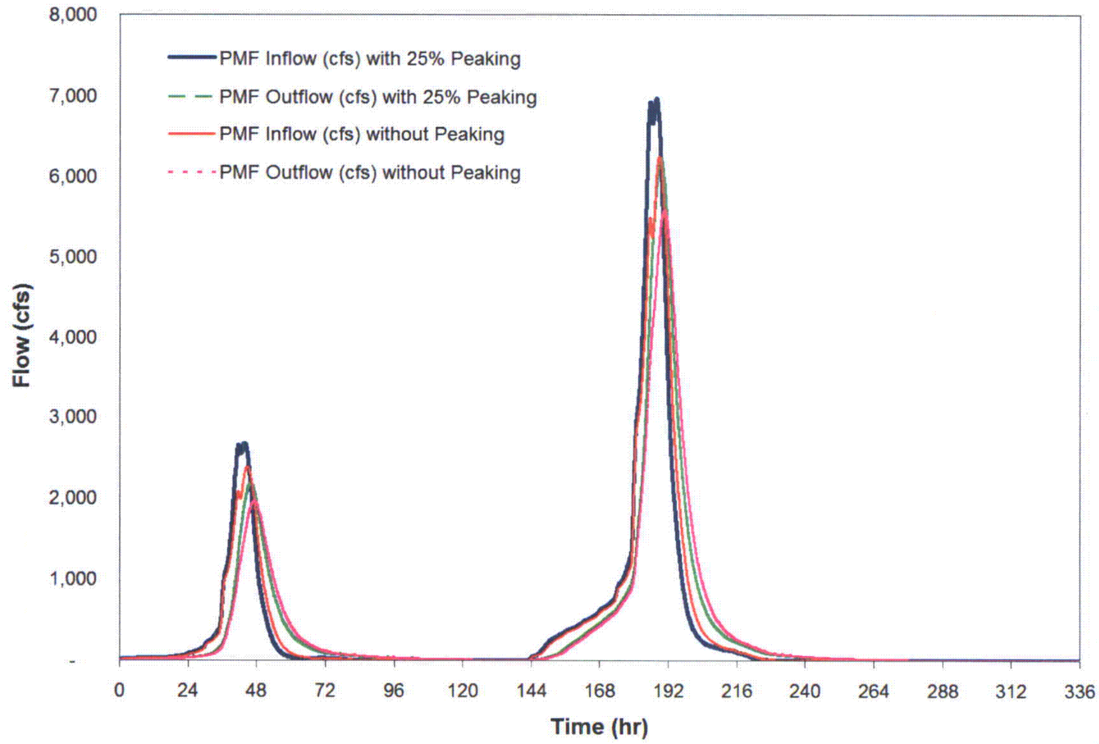


Figure 8c: Comparison of Auxiliary Reservoir PMF Inflows and Outflows, Peaking vs. No Peaking, Option 2, Scenario 3 (Tainter Gate Opened at 241 feet NGVD29)

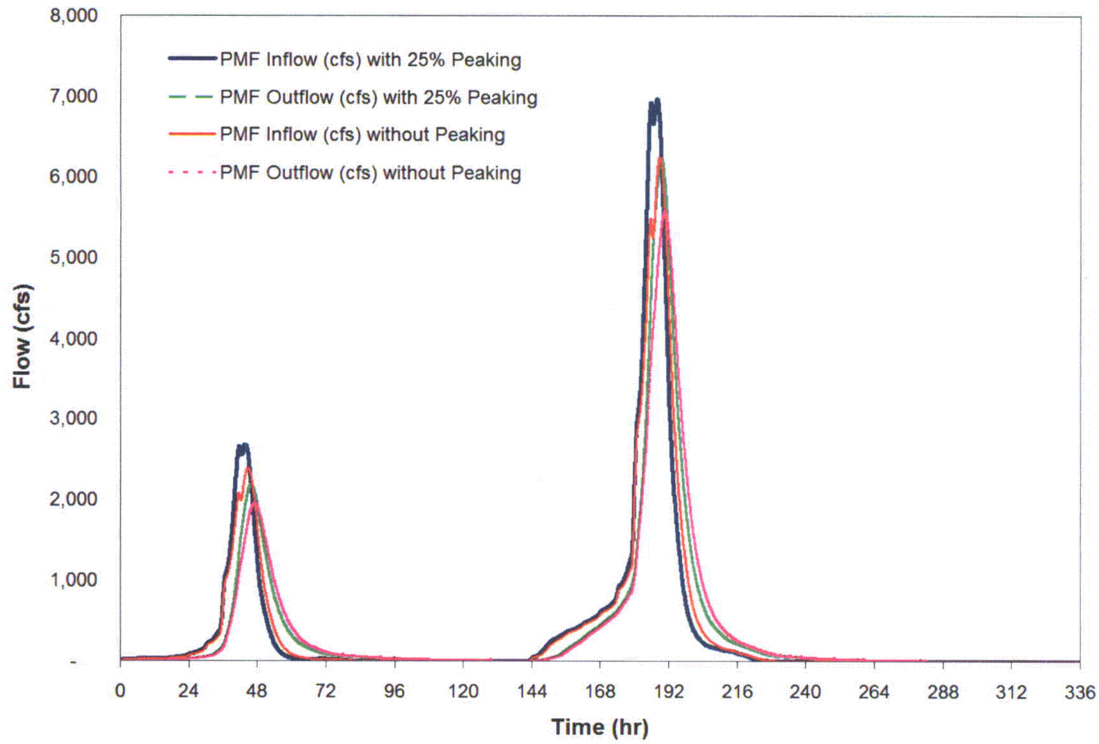


Figure 9a: Comparison of Main Reservoir PMF Inflows and Outflows, Peaking vs. No Peaking, Option 2, Scenario 1 (Tainter Gate Opened at 243 feet NGVD29)

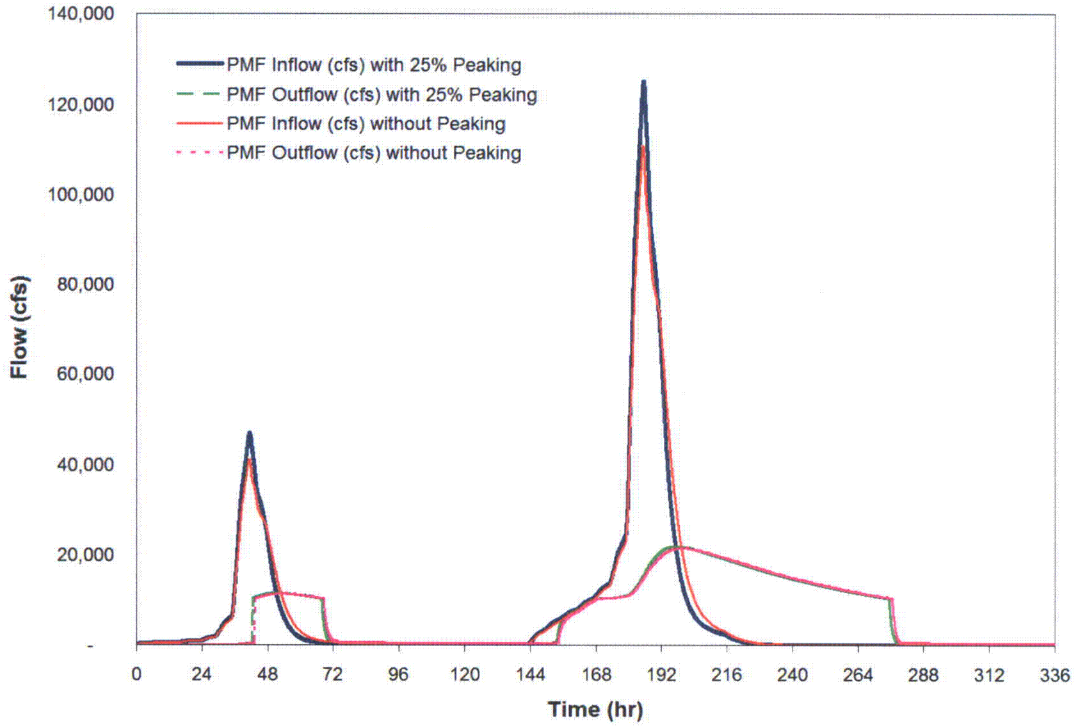


Figure 9b: Comparison of Main Reservoir PMF Inflows and Outflows, Peaking vs. No Peaking, Option 2, Scenario 2 (Tainter Gate Opened at 242 feet NGVD29)

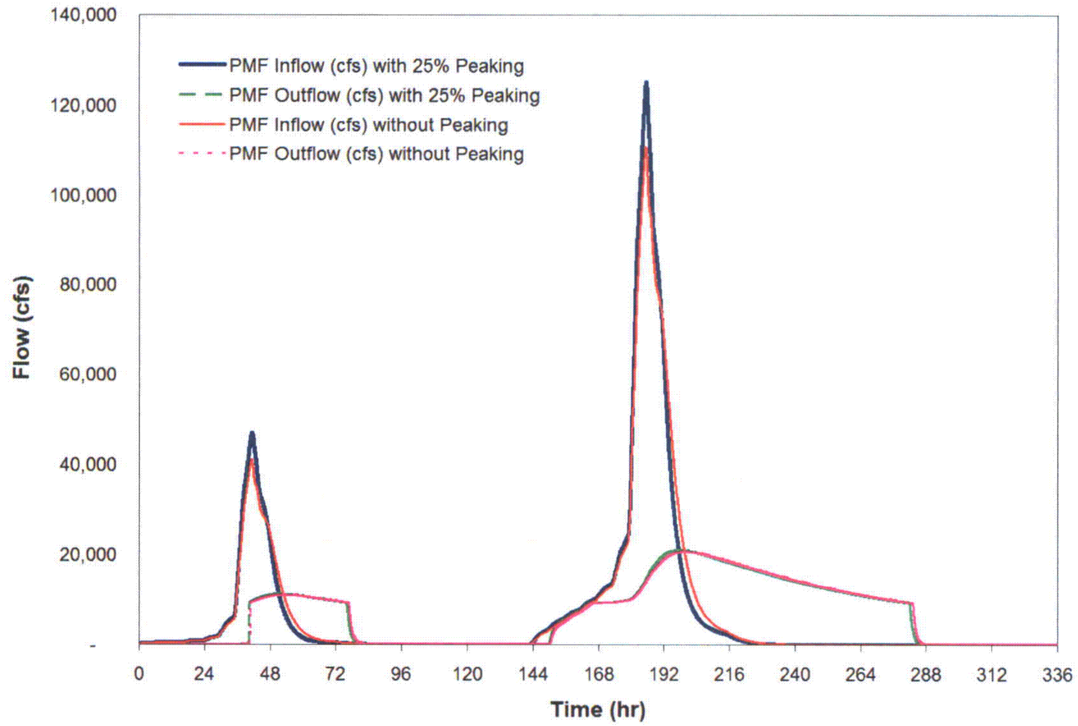


Figure 9c: Comparison of Main Reservoir PMF Inflows and Outflows, Peaking vs. No Peaking, Option 2, Scenario 3 (Tainter Gate Opened at 241 feet NGVD29)

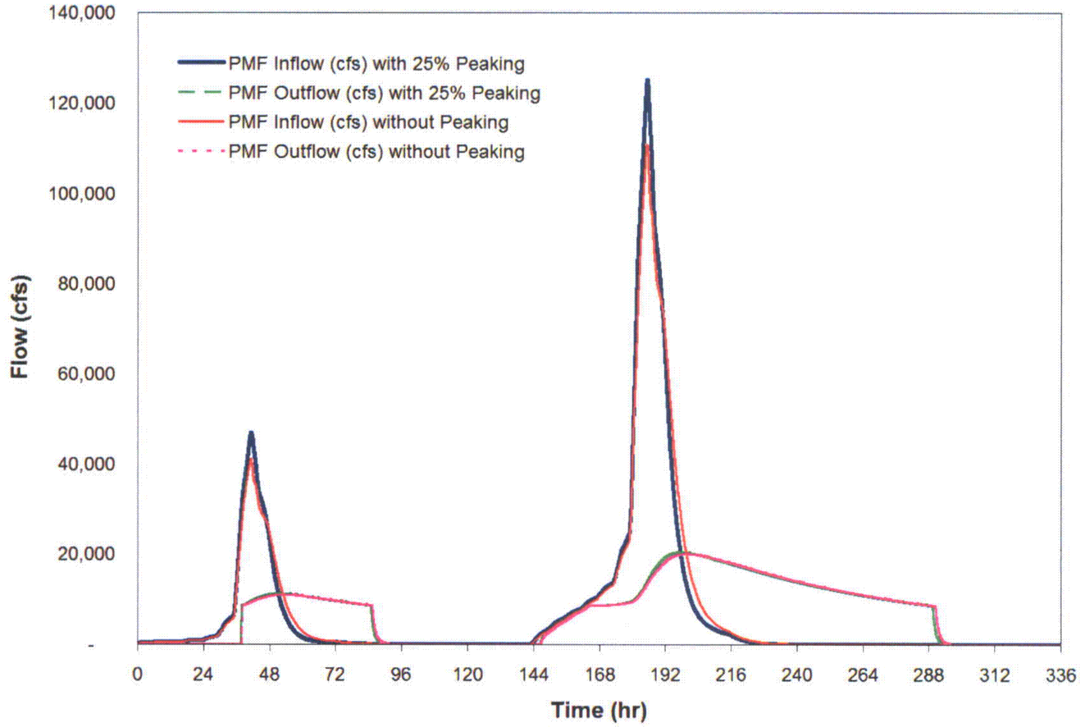


Figure 10a: Comparison of PMF Still Water Elevation in Auxiliary Reservoir, Peaking vs. No Peaking, Option 2, Scenario 1 (Tainter Gate Opened at 243 feet NGVD29)

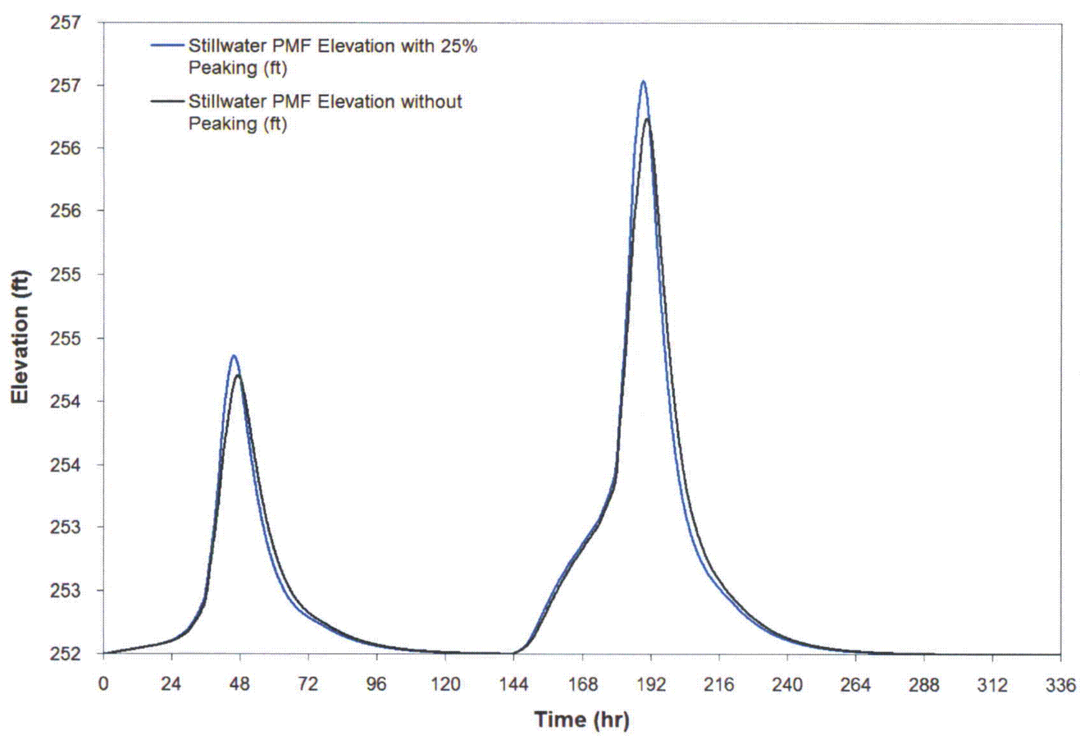


Figure 10b: Comparison of PMF Still Water Elevation in Auxiliary Reservoir, Peaking vs. No Peaking, Option 2, Scenario 2 (Tainter Gate Opened at 242 feet NGVD29)

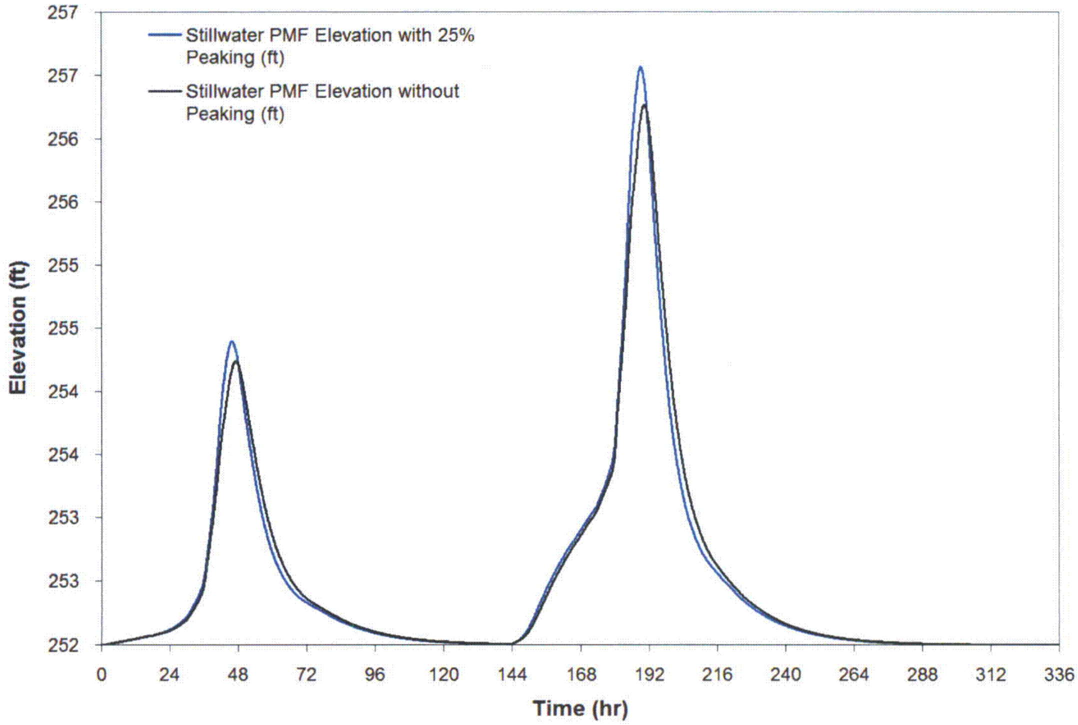


Figure 10c: Comparison of PMF Still Water Elevation in Auxiliary Reservoir, Peaking vs. No Peaking, Option 2, Scenario 3 (Tainter Gate Opened at 241 feet NGVD29)

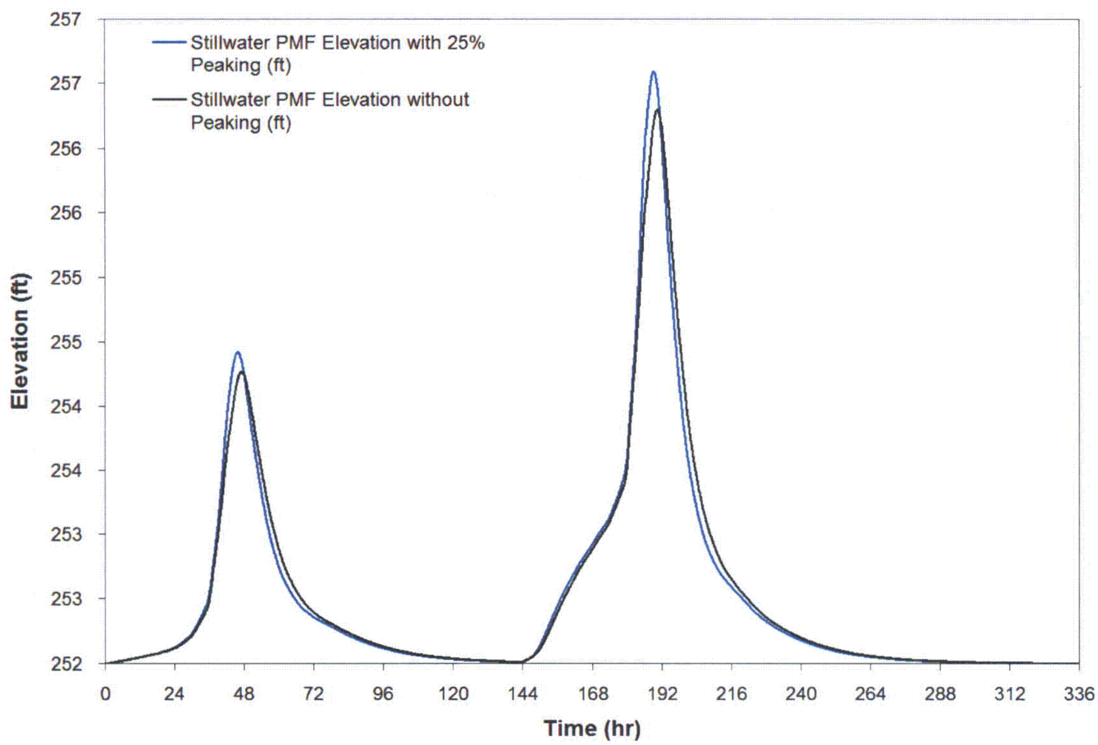


Figure 11a: Comparison of PMF Still Water Elevation in Main Reservoir, Peaking vs. No Peaking, Option 2, Scenario 1 (Tainter Gate Opened at 243 feet NGVD29)

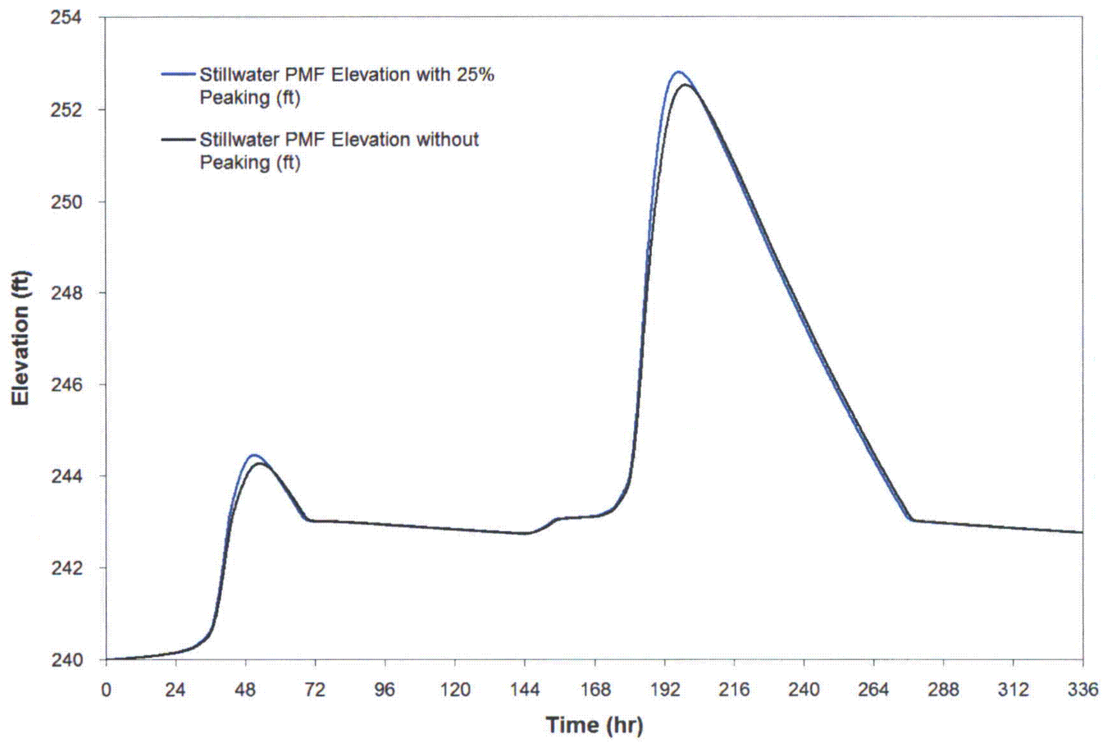


Figure 11b: Comparison of PMF Still Water Elevation in Main Reservoir, Peaking vs. No Peaking, Option 2, Scenario 2 (Tainter Gate Opened at 242 feet NGVD29)

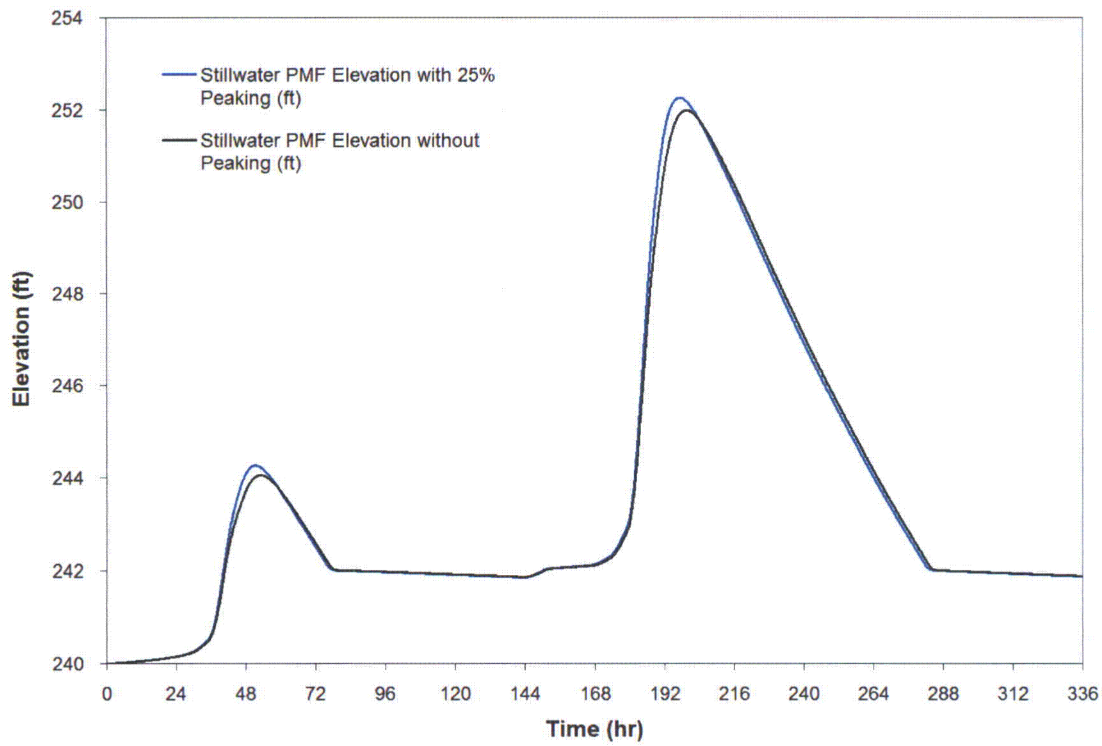
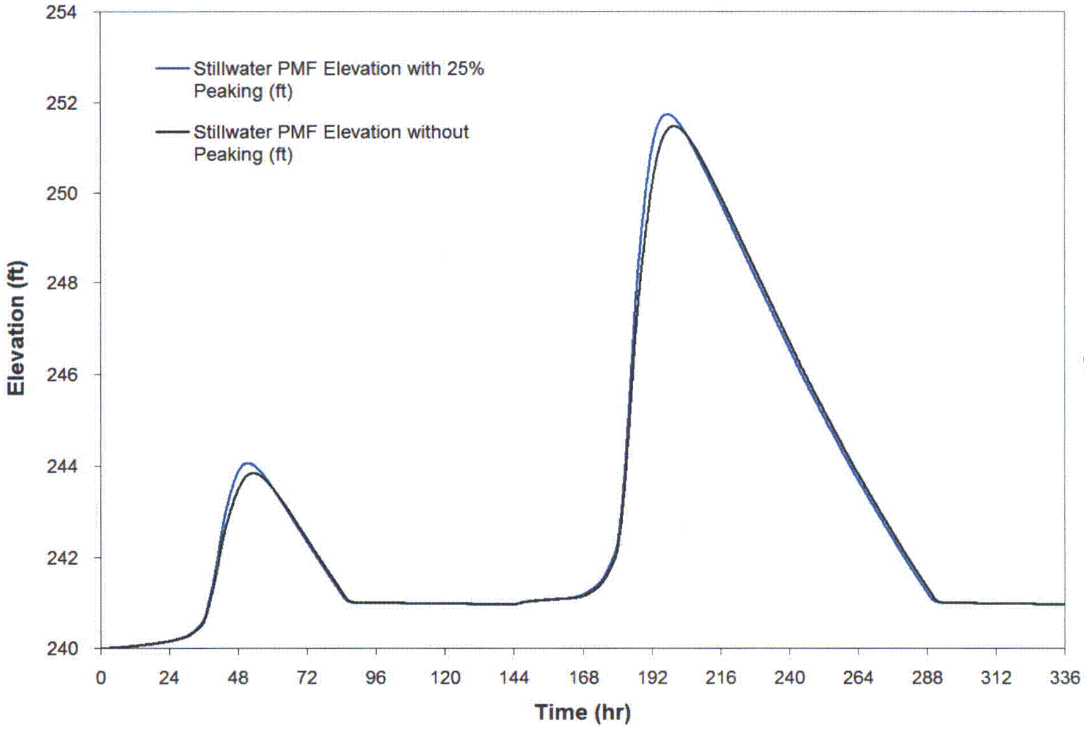


Figure 11c: Comparison of PMF Still Water Elevation in Main Reservoir, Peaking vs. No Peaking, Option 2, Scenario 3 (Tainter Gate Opened at 241 feet NGVD29)



Associated HAR COL Application Revisions:

Associated HAR COL Application revisions are detailed in HAR FSAR RAI 02.04.03-4 (supplemental response to HAR-RAI-LTR-023; NPD-NRC-2009-230).

Attachments/Enclosures:

None.