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DUKE ENERGY

10 CFR 50.54(f)

March 24, 2014
Serial: HNP-14-029

ATTN: Document Control Desk
U.S. Nuclear Regulatory Commission
Washington, DC 20555

Duke Energy Progress, Inc. (Duke Energy)
Shearon Harris Nuclear Power Plant, Unit 1
Docket No. 50-400

Subject: Response to Request for Additional Information Regarding Fukushima Lessons Learned - Flooding Hazard Reanalysis Report

References:

1. NRC Letter, *Request for Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding Recommendations 2.1, 2.3, and 9.3, of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident*; dated March 12, 2012, (Agencywide Documents Access and Management System (ADAMS) Accession No. ML12053A340)
2. Duke Energy Letter, *Flooding Hazard Reevaluation Report*, dated March 12, 2013 (ADAMS Accession No. ML 13079A253)
3. NRC Letter, *Shearon Harris Nuclear Power Plant, Unit 1 - Request for Additional Information Regarding Fukushima Lessons Learned - Flooding Hazard Reanalysis Report (TAC No. MF1103)*, dated February 10, 2014, (ADAMS Accession No. ML14030A419)

Ladies and Gentlemen:

On March 12, 2012, the Nuclear Regulatory Commission (NRC) staff issued Reference 1 requesting information pursuant to Title 10 of the Code of Federal Regulations Part 50, Section 50.54(f). Enclosure 2, of that letter, contains specific requested information associated with Near-Term Task Force Recommendation 2.1 for Flooding. In response to Required Response 2, Duke Energy submitted Reference 2, the Flooding Hazard Reevaluation Report for Shearon Harris Nuclear Power Plant, Unit 1.

By letter dated February 10, 2014 (Reference 3), the NRC staff requested additional information regarding Reference 2 in order to complete its review. The Duke Energy response to the request for additional information for Shearon Harris Nuclear Power Plant, Unit 1, is enclosed.

This letter contains no new Regulatory Commitments and no revision to existing Regulatory Commitments.

A010
NRR

Should you have any questions regarding this submittal, please contact Dave Corlett, Regulatory Affairs Manager, at (919) 362-3137.

I declare under penalty of perjury that the foregoing is true and correct.
Executed on March 24, 2014.

Sincerely,

Ernest J. Kapopoulos, Jr.

Enclosure: Response to Request for Additional Information Regarding Fukushima Lessons Learned - Flooding Hazard Reevaluation Report

cc: Mr. J. D. Austin, NRC Sr. Resident Inspector, HNP
Mr. A. Hon, NRC Project Manager, HNP
Mr. V. M. McCree, NRC Regional Administrator, Region II

Enclosure

Response to Request for Additional Information

Regarding Fukushima Lessons Learned - Flooding Hazard Reevaluation Report

Shearon Harris Nuclear Power Plant, Unit 1

Docket No. 50-400

RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

FUKUSHIMA LESSONS LEARNED

FLOODING HAZARD REEVALUATION REPORT

DUKE ENERGY PROGRESS, INC.

SHEARON HARRIS NUCLEAR POWER PLANT (HNP), UNIT 1

DOCKET NO. 50-400

By letter dated March 12, 2012, the U.S. Nuclear Regulatory Commission (NRC) issued a request for information pursuant to Title 10 of the *Code of Federal Regulations,* Section 50.54(f) (hereafter referred to as the 50.54(f) letter). The request was issued as a part of implementing lessons-learned from the accident at the Fukushima Dai-ichi nuclear power plant. Enclosure 2 to the 50.54(f) letter requested licensees to perform a flood hazard reevaluation using present-day methodologies and guidance.

By letter dated March 12, 2013 (Agencywide Documents Access and Management System (ADAMS) Accession No. ML 13079A253), Duke Energy submitted a response to Enclosure 2, Required Response 2 of the 50.54(f) letter. The NRC staff has determined that the following additional information is needed to complete its review.

RAI No. 1, Local Intense Precipitation Flooding - A

Background: Given the significant role of elevation data in defining slopes and flowpaths, the staff needs a description of the methods used to incorporate elevation measurements into its local intense precipitation-induced flood analyses, in order to complete the staff's independent evaluations.

Request: Provide descriptions of the sources of elevation data, the methods used to incorporate elevation measurements into local intense precipitation flood analysis, and the likely magnitude of the errors associated with these elevations.

Response: The Digital Elevation Model (DEM) data for HNP site was generated from Photogrammetric survey data provided in the format of CAD drawing which included 2-ft contours and elevation points. The contours and point data were processed by ARC/INFO to generate a DEM raster grid file for the HNP site. The DEM data was used to delineate the watershed into subbasins (see RAI No. 1 Figure 1). By using the standard of National Standard for Spatial Data Accuracy (NSSDA), the accuracy of the DEM was best estimated to have root mean square error (RMSE) of 0.61 ft for the 2-ft contour.

RAI No. 1 Figure 1: Delineated watershed for HNP Unit 1 site including flowpaths.

RAI No.2. Local Intense Precipitation Flooding - B

Background: Given the significant role of estimated times-of-concentration in the determination of the rainfall intensity and therefore the estimation of discharge during the local intense precipitation event, the staff needs the data used to estimate times-of-concentrations for all subbasins, in order to complete the staff's independent evaluations.

Request: Provide the data used to estimate times-of-concentration for each subbasin.

Response: The data used to estimate times-of-concentration for each subbasin is provided in RAI No. 2, Table 1

RAI No. 2 Table 1. Sub-Basin Outlet Time of Concentration Estimates

Basin	U/S Basins	Slope	L_{ov}, ft	K	N-Retardance	T_{ov}, min	L_{ch}, ft	K_{ch}	S_{ch}	T_{ch}, min	$T_c = T_{ov} + T_{ch}$	Rain Intensity, in/hr
1		0.0029	1200	0.828	0.02	14.4	315.0	0.0078	0.0190	3.0	17.4	36.80
2		0.0050	830	0.828	0.02	10.7	0.0	0.0078		0.0	10.7	48.31
3*	Cooling Tower											
4		0.0010	1200	0.828	0.02	18.5	1620.0	0.0078	0.0012	30.4	48.9	20.66
5*	Containment bldg, NW.											
6*	Containment bldg. SW.											
7		0.0028	720	0.828	0.02	11.5					11.5	46.45
8*	Containment bldg. SE.											
9		0.0047	1200	0.828	0.02	12.9	450.0	0.0078	0.0211	3.8	16.7	37.69
10	14	0.0017	1200	0.828	0.02	16.4	0.0	0.0078	0.0000			20.15
11		0.0070	427	0.828	0.02	7.2	0.0	0.0078		0.0	7.2	60.12
12		0.0010	1200	0.828	0.02	18.5	718.0	0.0078	0.0002	33.5	52.0	19.97
13		0.0044	740	0.828	0.02	10.4	0.0	0.0078		0.0	10.4	49.04
14		0.0037	1200	0.828	0.02	13.6	2200.0	0.0078	0.0013	37.6	51.2	20.15
15		0.0100	1200	0.828	0.02	10.8	156.0	0.0078	0.0925	1.0	11.7	45.88
16*	Containment bldg. NE.											
17		0.0053	946	0.828	0.02	11.2	0.0	0.0078	0.0000	0.0	11.2	47.07
18		0.0022	526	0.828	0.02	10.4	0.0	0.0078	0.0000	0.0	10.4	48.96

*Basins 3, 5, 6, 8, and 16 are permanent structures and retain any rainfall in the Cooling Tower basin, respective sumps, or Tank Building.

RAI No. 3, Local Intense Precipitation Flooding - C

Background: Given the significant role of depressions in Subbasins 5, 6, 8, and 16 in the estimation of overall discharge conveyed downstream, the staff needs more detailed description of these areas and why these depressions would remain unchanged in future, in order to complete the staff's independent evaluations.

Request: Provide detailed description of the areas with depressions in Subbasins 5, 6, 8, and 16 and why these depressions would remain unchanged in future.

Response: The description of Subbasins 5, 6, 8, and 16 are provided below:

Area 5 (HNP Unit 3 area)

Runoff is directed to a sump area that has dimensions of 70 ft. x 200 ft (Fuel Handling Building - Retaining Wall Cavity Area). The invert of this sump is at elevation 216 ft and the sump extends to elevation 261 ft before overflowing onto the site.

The sump provides (70 ft x 200 ft x 1 ft) = 14,000 ft^3 for each foot of storage. The runoff volume of 45,111 ft^3 will require 3.22 ft of storage depth, filling the sump to an elevation of 219.22 ft. All of the runoff volume from the PMP event will be retained within the sump.

Area 6 (HNP Unit 4 and part of Unit 1 area)

Runoff is directed to a sump area that has dimensions of 70 ft. x 200 ft (Fuel Handling Building - Retaining Wall Cavity Area). The invert of this sump is at elevation 216 ft and the sump extends to elevation 261 ft before overflowing onto the site.

The sump provides (70 ft x 200 ft x 1 ft) = 14,000 ft^3 for each foot of storage. The runoff volume of 136,087 ft^3 will require 9.72 ft of storage depth, filling the sump to an elevation of 225.72 ft. All of the runoff volume from the PMP event will be retained within the sump.

Area 16 (HNP Unit 2 area)

Runoff is directed to a sump area of variable dimensions. The lowest portion of that sump has dimensions of 175 ft. x 200 ft. The invert of this sump is at elevation 236 ft and the sump extends to elevation 242 ft before extending into additional sump area. The sump will not overflow onto the site until elevation 261.5 ft.

The sump provides (175 ft x 200 ft x 1 ft) = 35,000 ft^3 for each foot of storage. The runoff volume of 131,865 ft^3 will require 3.76 ft of storage depth, filling the lower sump to an elevation of 239.76 ft. All of the runoff volume from the PMP event will be retained within the sump.

Area 8 (Part of Unit 1 area)

Runoff from the building roofs in this area ends up within the walls enclosing the Tank Building. This consists of two separate spaces; both of which have an invert of 261.0 ft and a top-of-wall of 286.0 ft.

The larger space is 50 ft x 55 ft and contains a 45 ft diameter tank. The space available for water storage is the area outside of the tank. This area provides a storage of ([50 ft x 55 ft] – π [(45 ft)^2/4]) = 1,160 ft^3 for each foot of storage.

The smaller space is 35 ft x 35 ft and contains a 27 ft diameter tank. The space available for water storage is the area outside of the tank. This area provides a storage of ([35 ft x 35 ft] – π [(27 ft)^2/4])= 653 ft^3 for each foot of storage.

The total storage volume is (1,160 ft^3 + 653 ft^3) = 1,813 ft^3 for each foot of storage. The runoff volume of 27,301 ft^3 will require 15.1 ft of storage depth, filling the space in the Tank Building to an elevation of 276.1 ft. All of the runoff volume from the PMP event will be retained within the Tank Building.

The four areas (5, 6, 8, and 16) with depressions mentioned above are described in Figure 2.4.2-5 of the HNP UFSAR and thus any modifications that would change them would be reviewed in accordance with the 10 CFR 50.59 process for impact on roof flooding before implementation.

RAI No. 4, Local Intense Precipitation Flooding - D

Background: Given the significant role of the HEC-RAS model in the licensee's analysis and the need to review the formulation of its complex spatially and temporally distributed input, the staff needs the licensee's HEC-RAS input files used for local intense precipitation analysis, in order to complete the staff's independent evaluations.

Request: Provide electronic versions of the input files used for HEC-RAS analysis in the flooding hazard reanalysis report (FHRR) related to local intense precipitation analyses. Also, provide a list or map showing which flowpath reaches used in the HEC-RAS model simulation are located in gravel-covered areas and which are located in concrete or asphalt-covered areas.

Response: The electronic versions of the input files are contained on the attached electronic media.

A list of which flowpath reaches used in the HEC-RAS model simulation are located in gravel-covered areas and which are located in concrete or asphalt-covered areas is provided below.

RAI No. 4 Table 1: Ground Surface Cover Used in Model

XS-SEC	Ground Surface Cover		XS-SEC	Ground Surface Cover
XS_1_2	gravel		XS_10_7	Gravel
XS_1_3	gravel		XS_10_8	Gravel
XS_1_4	Gravel/asphalt		XS_10_9	Gravel
XS_1_5	Asphalt		XS_10_10	Gravel
XS_1_6	Concrete/asphalt		XS_10_11	Gravel

XS-SEC	Ground Surface Cover	XS-SEC	Ground Surface Cover
XS_1_7	Concrete/asphalt	XS_10_12	Gravel/concrete
XS_1_8	Concrete/asphalt	XS_11_2	Gravel/asphalt
XS_1_9	Concrete/asphalt	XS_11_3	Gravel
XS_1_10	Concrete/asphalt	XS_11_4	Gravel
XS_1_11	Concrete/asphalt	XS_11_5	Gravel
XS_2_2	Gravel/asphalt	XS_12_2	Asphalt
XS_2_3	Asphalt	XS_12_3	Asphalt
XS_2_4	Gravel	XS_12_4	Asphalt
XS_4_2	Gravel	XS_12_5	Asphalt
XS_4_3	Concrete/asphalt	XS_12_6	Concrete/asphalt
XS_4_4	Concrete/asphalt	XS_12_7	Asphalt/concrete
XS_4_5	Concrete/asphalt	XS_13_2	Gravel
XS_4_6	Concrete/asphalt	XS_13_3	Asphalt/concrete
XS_4_7	Concrete/asphalt	XS_13_4	Asphalt/concrete
XS_4_8	Concrete/asphalt	XS_13_5	Asphalt/concrete
XS_4_9	Concrete/asphalt	XS_13_6	Asphalt/concrete
XS_4_10	Concrete/asphalt	XS_13_7	Asphalt/concrete
XS_4_11	Concrete/asphalt	XS_14_2	Gravel
XS_4_12	Concrete/asphalt	XS_14_3	Gravel/asphalt
XS_4_13	Concrete/asphalt	XS_14_4	Asphalt/concrete
XS_4_14	Concrete/asphalt	XS_14_5	Gravel
XS_7_2	Gravel/asphalt	XS_14_6	Gravel
XS_7_3	Concrete/asphalt	XS_15_2	Gravel
XS_7_4	Concrete/asphalt	XS_15_3	Gravel
XS_7_5	Concrete/asphalt	XS_15_4	Gravel
XS_9_2	gravel	XS_15_5	Gravel
XS_9_3	gravel	XS_15_6	Gravel
XS_9_4	Gravel	XS_15_7	Asphalt
XS_9_5	Gravel/asphalt	XS_15_8	Asphalt
XS_9_6	Gravel/asphalt	XS_15_9	Asphalt
XS_9_7	Asphalt/concrete	XS_17_2	Concrete/asphalt
XS_9_8	Concrete/asphalt	XS_17_3	Concrete/asphalt
XS_9_9	Concrete/asphalt	XS_17_4	Gravel/concrete/asphalt
XS_9_10	Concrete/asphalt	XS_17_5	Gravel/concrete/asphalt

XS-SEC	Ground Surface Cover	XS-SEC	Ground Surface Cover
XS_10_2	Gravel	XS_17_6	Asphalt
XS_10_3	Gravel	XS_18_2	Gravel
XS_10_4	Gravel/concrete	XS_18_3	Gravel/asphalt
XS_10_5	Gravel/concrete/asphalt	XS_18_4	Asphalt
XS_10_6	Gravel	XS_18_5	Asphalt

RAI No. 5, Local Intense Precipitation Flooding - E

Background: The staff noted that mean sea level (MSL) and National Geodetic Vertical Datum of 1929 (NGVD29) datums are used in the FHRR such that they appear interchangeable. However, the FHRR does not clearly state if this is the case. Given the significant role the elevation data have in relating water-surface elevations and site topography through the use of consistent datums, the staff needs a consistent description of the relationship between MSL and NGVD29 used in its analysis, in order to complete the staff's independent evaluations.

Request: Clarify the relationship between the vertical elevation datums used in the FHRR.

Response: HNP is situated at a plant grade elevation of 260 ft NGVD29 (National Geodetic Vertical Datum of 1929, commonly referred to as Mean Sea Level). Therefore, MSL and NGVD29 are interchangeable in the FHRR.

RAI No. 6, Local Intense Precipitation Flooding - F

Background: In its review of the FHRR, the staff noted that the licensee did not provide full citations of technical references used in the local intense precipitation analysis. A full citation to the HNP Unit 1 Final Safety Analysis Report (FSAR) is also missing.

Request: Provide full citations to technical references and the HNP Unit 1 FSAR used in the FHRR.

Response: The full technical references used in the FHRR for Local Intense Precipitation Flooding are shown below:

1. Carolina Power and Light Company (CP&L), Shearon Harris Nuclear Power Plant Final Safety Analysis Report, Amendment 58.

2. Carolina Power Plant & Light Company Shearon Harris Nuclear Power Plant (CP&L) (2010). Site Plan. Drawing CAR-2165 G-003. Rev. 19. (PDF from Hard Copy only. No electronic CAD file is available.)

3. Chow, V. T. (1959), Open-Channel Hydraulics, McGraw-Hill.

4. CH2M Hill (2012), HEC-RAS Analysis for Local Probable Maximum Precipitation (PMP), Cal. No. HAG-0000-X7C-044, Rev. 2. Shearon Harris Nuclear Power Plant, Harris Advanced Reactor Units 2 & 3 (HAR 2 & HAR 3). Feb. 29, 2012.

5. ESRI (2011) Arc Hydro Tool, version 2.

6. National Oceanic and Atmospheric Administration (NOAA) (1978). "Probable Maximum Precipitation Estimates – United States East of the 105th Meridian," Hydrometeorological Report No. 51 (HMR 51), National Weather Service, NOAA.

7. National Oceanic and Atmospheric Administration (NOAA) (1982) "Application of Probable Maximum Precipitation Estimates – United States East of the 105th Meridian," Hydrometeorological Report No. 52 (HMR 52), National Weather Service, NOAA.

8. Texas Department of Transportation (2011). Hydraulic Design Manual.

9. U.S. Army Corps of Engineers (2010), Hydrologic Engineering Center, HEC-RAS, River Analysis System, Version 4.1.0, January 2010.

10. U.S. Army Corps of Engineers (1994). *Flood-Runoff Analysis*, EM 1110-2-1417.

11. U.S. Department of Agriculture (USDA), Soil Conservation Service (now known as Natural Resources Conservation Service) (1986), Technical Release 55, *Urban Hydrology for Small Watersheds.*

12. U.S. Nuclear Regulatory Commission (NRC) (2011). Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America. NUREG/CR-7046/PNNL-20091. Office of Nuclear Regulatory Research.

13. URS Corporation (2013). Site Walkdown. Rev. 0. Report prepared for Duke Energy. Study Number: 30958-096-12-05-100-001.

RAI No.7, Streams and Rivers Flooding- A

Background: The licensee used the analyses from the combined license application (COLA) for the proposed Harris Advance Reactor (HAR) Units 2 and 3 (Duke 2013) as the basis for reevaluating flooding from streams and rivers at the current HNP Unit 1 site. The COLA was based on a Main Dam configuration different from that used for developing the current licensing basis (CLB) (Duke 2013). In addition to the FHRR and HNP FSAR, NRC staff used the latest revision (Rev. 4) of the HAR COLA (Progress Energy 2012) for its review.

Request: Provide an explanation why the HAR Units 2 and 3 COLA analyses are appropriate for the reanalysis of HNP CLB for flooding from streams and rivers.

Response: The methodologies used in COLA for HAR Units 2 and 3 sites for basin-wide PMP calculation and PMF on Streams and Rivers calculation are consistent with present-day methodologies described in NUREG/CR-7046. Also, the existing HNP Unit 1 site is co-located with the proposed HAR Units 2 and 3 sites, so it is justified to use applicable model inputs/outputs from HAR Units 2 and 3 for HNP's existing Unit 1 flood hazard re-evaluation.

RAI No. 8. Streams and Rivers Flooding - B

Background: Same as RAI No. 7

Request: Provide an explanation of changes to the COLA analyses to correspond to the site and reservoir configurations with only HNP Unit 1 present.

Response:

Revisions to the COLA analysis model input were made to represent the present configuration of the Main Reservoir as follows:

- Stage-Storage: The Main Reservoir storage data was updated with respect to the HNP FSAR.
- Spillway Discharge Rating Curves: Spillway discharge rating curves were based on Figures 2.4.3-3 and 2.4.3-4 of the HNP FSAR. The input table for the Auxiliary Dam spillway for the HEC-HMS model was taken directly from HAR Units 2 and 3 COLA analyses as shown in RAI No. 8 Table 1. There is no applicable Main Dam spillway rating curve in the analysis mentioned above since it is for future conditions (which included raising the main dam spillway to an elevation of 240 ft.). RAI No. 8 Table 3 shows the calculation of an extended rating curve for the Main Dam spillway, (at its current configuration at an elevation of 220 ft.), based on Figure 2.4.3-4 of the HNP FSAR. This was necessary for reservoir elevations above 239.5 ft NGVD29. The model input for the Main Dam is shown in RAI No. 8 Table 2 and the plot of the data in RAI No. 8 Table 3 is shown in RAI No. 8 Figure 1.
- Emergency Spillway: The proposed emergency spillway on the west abutment of the Main Dam, (to be built if the main reservoir were to be raised to an elevation of 240 ft. if HAR Units 2 and 3 were constructed), was not included the model for Unit 1.

RAI No. 8 Table 1: Elevation-Outflow Rating Curve for Auxiliary Reservoir

Ordinate	STAGE	FLOW
Labels		
Units	FT	CFS
Type	UNT	UNT
1	252.00	0.0
2	252.76	0.0
3	253.14	632.0
4	253.52	1,049.0
5	253.90	1,520.0
6	254.28	2,050.0
7	254.66	2,678.0
8	255.04	3,319.0
9	255.42	4,013.0
10	255.80	4,759.0
11	256.18	5,556.0
12	256.56	6,402.0
13	256.94	7,297.0
14	257.32	8,238.0
15	257.70	9,227.0
16	258.08	10,262.0
17	258.46	11,342.0
18	258.84	12,466.0
19	259.22	13,635.0
20	259.60	14,848.0
21	260.00	16,172.0

RAI No. 8 Table 2: Elevation-Outflow Rating Curve for Main Reservoir

Ordinate	STAGE	FLOW
Labels		
Units	FT	CFS
Type	UNT	UNT
1	220.0	0.0
2	220.5	57.0
3	221.0	160.0
4	221.5	294.0
5	222.0	452.0
6	222.5	631.0
7	223.0	829.0
8	223.5	1,045.0
9	224.0	1,276.0
10	224.5	1,522.0
11	225.0	1,782.0
12	225.5	2,055.0
13	226.0	2,340.0
14	226.5	2,638.0
15	227.0	2,947.0
16	227.5	3,267.0
17	228.0	3,597.0
18	228.5	3,938.0
19	229.0	4,289.0
20	229.5	4,649.0
21	230.0	5,176.0
22	230.5	5,567.0
23	231.0	5,967.0
24	231.5	6,376.0
25	232.0	6,793.0
26	232.5	7,219.0
27	233.0	7,769.0
28	233.5	8,219.0
29	234.0	8,676.0
30	234.5	9,223.0
31	235.0	9,786.0
32	235.5	10,306.0
33	236.0	10,836.0
34	236.5	11,409.0
35	237.0	11,996.0
36	237.5	12,560.0

RAI No. 8 Table 2: Elevation-Outflow Rating Curve for Main Reservoir (cont.)

Ordinate	STAGE	FLOW
37	238.0	13,134.0
38	238.5	13,719.0
39	239.0	14,273.0
40	239.5	14,834.0
41	240.0	15,402.0
42	240.5	15,997.0
43	241.0	16,558.0
44	241.5	17,146.0
45	242.0	17,740.0
46	242.5	18,341.0
47	243.0	18,948.0
48	243.5	19,561.0
49	244.0	20,181.0
50	244.5	20,806.0
51	245.0	21,438.0
52	245.5	22,075.0
53	246.0	22,718.0
54	246.5	23,367.0
55	247.0	24,022.0
56	247.5	24,682.0
57	248.0	25,348.0
58	248.5	26,019.0
59	249.0	26,696.0
60	249.5	27,378.0
61	250.0	28,065.0
62	250.5	28,756.0
63	251.0	29,456.0
64	251.5	30,159.0
65	252.0	30,867.0
66	252.5	31,581.0
67	253.0	32,299.0
68	253.5	33,022.0
69	254.0	33,750.0
70	254.5	34,484.0
71	255.0	35,221.0
72	255.5	35,964.0
73	256.0	36,711.0
74	256.5	37,463.0
75	257.0	38,220.0

Ordinate	STAGE	FLOW
76	257.5	38,981.0
77	258.0	39,747.0
78	258.5	40,517.0
79	259.0	41,292.0
80	259.5	42,072.0
81	260.0	42,855.0

RAI No. 8 Table 3. Present Main Dam Spillway Rating Curve Extension Calculation

(Figures 9.23 and 9.24 are from the Design of Small Dams Manual)

filename = HNPMainSpway.xlsx

Discharge Rating Curve for Main Dam Spillway

Existing Spillway, Crest El. 220, Extend Rating Curve up to El. 260

Effective Crest Length:

L = L'-2(NKp + Ka)He — where L is net length = 50 feet
(COLA FSAR P. 2.4-31) — He = total head on spillway, =res. WL - 220
Kp = Ka = 0.01
N = number of piers = 1
Thus effective crest length = Leff = 50-2*0.02*(WL-220)

Q = C*L*He^1.5

C varies with head. From Design of Small Dams Figures 9.23 and 9.24,

P/Ho = 3; Co = 3.95 from COLA FSAR page 2.4-31

C = Co*(0.86242+ 0.13731*(He/Ho)^0.5)^2

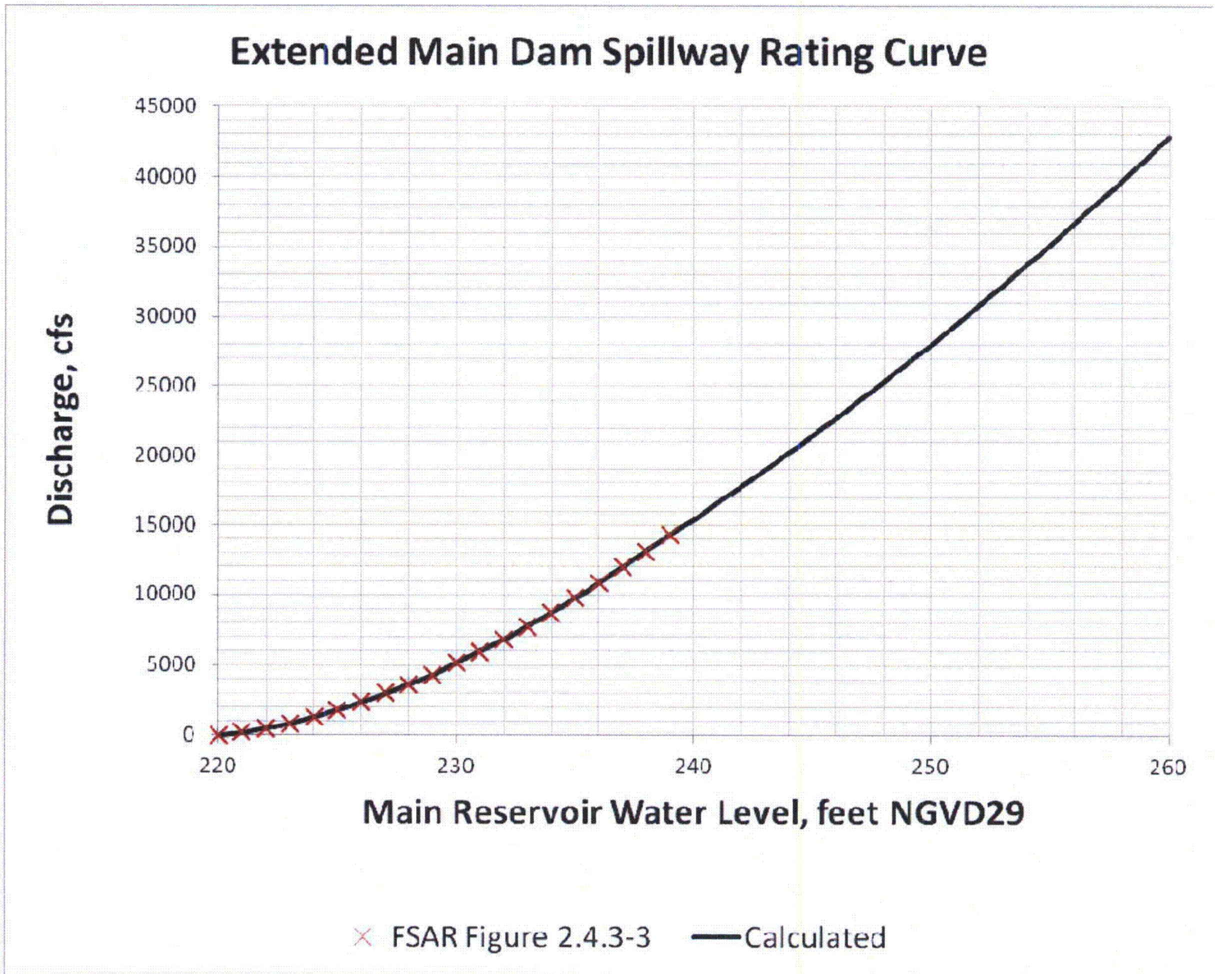
However from the existing rating curve, HNP FSAR Figure 2.4.3-3, C is clearly less than 3.95. Original spillway rating curve calculation is not available. Find C that fits Figure 2.4.3-3 and use highest value to extend rating curve from El. 240 to El. 260.

Reservoir still water level in feet NGVD29

		Q read from FSAR Fig. 2.4.3-3		Assumed C to fit FSAR Figure 2.4.3-3	
Res. WL	He, ft.	Q(FSAR)	Leff, ft.	Cfsar	Calculated Q, cfs
220	0	0	50	0	0
220.5	0.5		49.98	3.2	57
221	1	200	49.96	3.2	160
221.5	1.5		49.94	3.2	294
222	2	500	49.92	3.2	452
222.5	2.5		49.9	3.2	631
223	3	820	49.88	3.2	829
223.5	3.5		49.86	3.2	1045
224	4	1270	49.84	3.2	1276
224.5	4.5		49.82	3.2	1522
225	5	1770	49.8	3.2	1782
225.5	5.5		49.78	3.2	2055
226	6	2350	49.76	3.2	2340
226.5	6.5		49.74	3.2	2638
227	7	3000	49.72	3.2	2947
227.5	7.5		49.7	3.2	3267
228	8	3600	49.68	3.2	3597
228.5	8.5		49.66	3.2	3938
229	9	4300	49.64	3.2	4289
229.5	9.5		49.62	3.2	4649
230	10	5150	49.6	3.3	5176
230.5	10.5		49.58	3.3	5567
231	11	5930	49.56	3.3	5967
231.5	11.5		49.54	3.3	6376
232	12	6800	49.52	3.3	6793
232.5	12.5		49.5	3.3	7219
233	13	7750	49.48	3.35	7769

RAI No. 8 Table 3. Present Main Dam Spillway Rating Curve Extension Calculation (cont.)

Res. WL	He, ft.	Q(FSAR)	Leff, ft.		Cfsar	Calculated Q, cfs	Storage, acre-feet
233.5	13.5		49.46		3.35	8219	138600
234	14	8700	49.44		3.35	8676	141400
234.5	14.5		49.42		3.38	9223	144200
235	15	9800	49.4		3.41	9786	147000
235.5	15.5		49.38		3.42	10306	149800
236	16	10850	49.36		3.43	10836	152600
236.5	16.5		49.34		3.45	11409	155400
237	17	12000	49.32		3.47	11996	158200
237.5	17.5		49.3		3.48	12560	161000
238	18	13100	49.28		3.49	13134	163800
238.5	18.5		49.26		3.5	13719	166600
239	19	14300	49.24		3.5	14273	169400
239.5	19.5		49.22		3.5	14834	172000
240	20		49.2		3.5	15402	174591
240.5	20.5		49.18		3.5	15977	
241	21		49.16		3.5	16558	
241.5	21.5		49.14		3.5	17146	
242	22		49.12		3.5	17740	
242.5	22.5		49.1		3.5	18341	
243	23		49.08		3.5	18948	
243.5	23.5		49.06		3.5	19561	
244	24		49.04		3.5	20181	
244.5	24.5		49.02		3.5	20806	
245	25		49		3.5	21438	
245.5	25.5		48.98		3.5	22075	
246	26		48.96		3.5	22718	
246.5	26.5		48.94		3.5	23367	
247	27		48.92		3.5	24022	
247.5	27.5		48.9		3.5	24682	
248	28		48.88		3.5	25348	
248.5	28.5		48.86		3.5	26019	
249	29		48.84		3.5	26696	
249.5	29.5		48.82		3.5	27378	
250	30		48.8		3.5	28065	
250.5	30.5		48.78		3.5	28758	
251	31		48.76		3.5	29456	
251.5	31.5		48.74		3.5	30159	
252	32		48.72		3.5	30867	
252.5	32.5		48.7		3.5	31581	
253	33		48.68		3.5	32299	
253.5	33.5		48.66		3.5	33022	
254	34		48.64		3.5	33750	
254.5	34.5		48.62		3.5	34484	
255	35		48.6		3.5	35221	
255.5	35.5		48.58		3.5	35964	
256	36		48.56		3.5	36711	
256.5	36.5		48.54		3.5	37463	
257	37		48.52		3.5	38220	
257.5	37.5		48.5		3.5	38981	
258	38		48.48		3.5	39747	
258.5	38.5		48.46		3.5	40517	
259	39		48.44		3.5	41292	
259.5	39.5		48.42		3.5	42072	
260	40		48.4		3.5	42855	



RAI No. 8 Figure 1: Extended Main Dam Spillway Rating Curve

RAI No. 9, Streams and Rivers Flooding - C

Background: The licensee determined that a 259 km^2 (100 mi^2) isohyetal pattern maximized the precipitation depth over the Buckhorn Creek watershed. Staff were unable to find a figure illustrating the distribution of the Probable Maximum Precipitation (PMP) pattern from U.S. Weather Bureau Hydro-meteorological Report (HMR 52) over the Buckhorn Creek watershed.

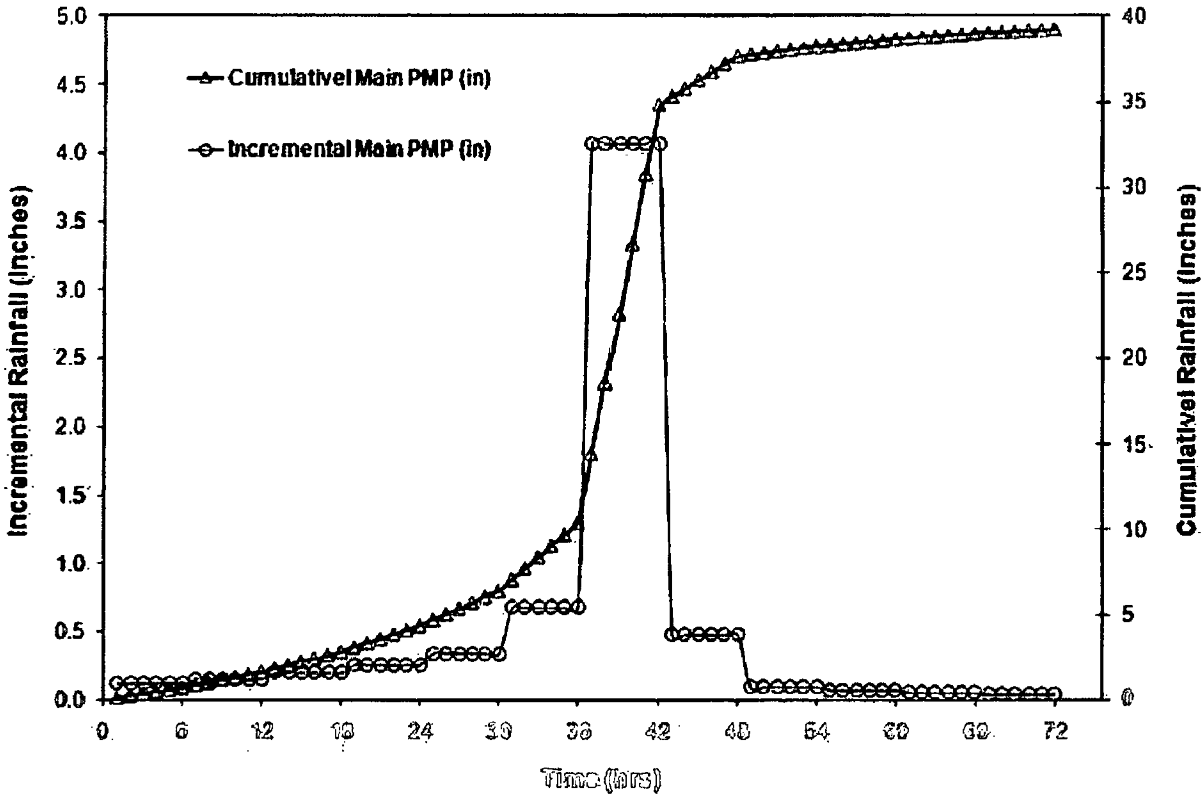
Request: Provide a figure showing the distribution of the PMP pattern derived from HMR 52 over the Buckhorn Creek watershed.

Response: The resulting hourly PMP rainfall distribution for the Buckhorn Creek drainage basin above the Main Dam is tabulated in RAI No. 9 Table 1 and plotted on RAI No. 9 Figure 1. Similarly, RAI No. 9 Figure 2 presents the PMP rainfall distribution for the drainage basin above the Auxiliary Dam.

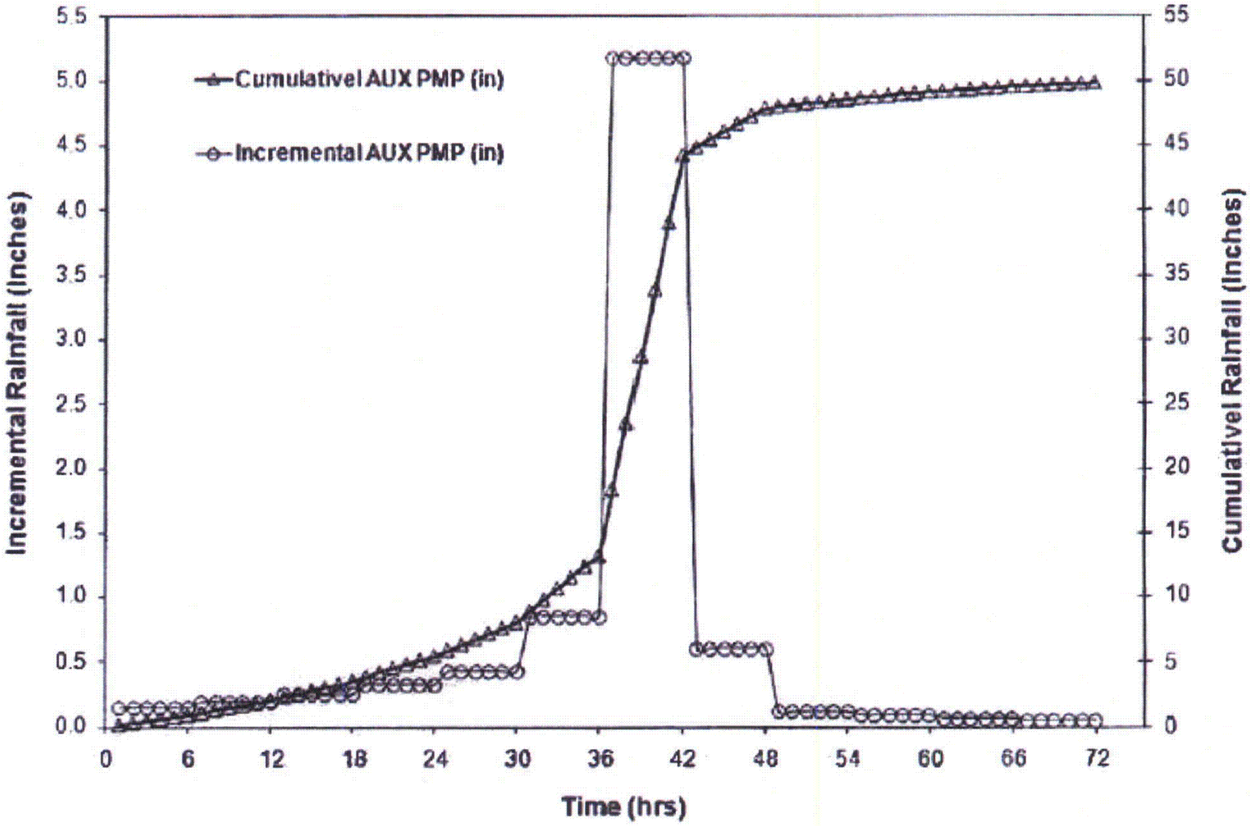
RAI No. 9 Table 1

Incremental Probable Maximum Precipitation for the Basins above 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	0.12	0.15	37	4.08	5.18
2	0.12	0.15	38	4.08	5.18
3	0.12	0.15	39	4.08	5.18
4	0.12	0.15	40	4.08	5.18
5	0.12	0.15	41	4.08	5.18
6	0.12	0.15	42	4.08	5.18
7	0.15	0.19	43	0.48	0.6
8	0.15	0.19	44	0.48	0.6
9	0.15	0.19	45	0.48	0.6
10	0.15	0.19	46	0.48	0.6
11	0.15	0.19	47	0.48	0.6
12	0.15	0.19	48	0.48	0.6
13	0.2	0.25	49	0.09	0.12
14	0.2	0.25	50	0.09	0.12
15	0.2	0.25	51	0.09	0.12
16	0.2	0.25	52	0.09	0.12
17	0.2	0.25	53	0.09	0.12
18	0.2	0.25	54	0.09	0.12
19	0.26	0.33	55	0.07	0.09
20	0.26	0.33	56	0.07	0.09
21	0.26	0.33	57	0.07	0.09
22	0.26	0.33	58	0.07	0.09
23	0.26	0.33	59	0.07	0.09
24	0.26	0.33	60	0.07	0.09
25	0.34	0.43	61	0.05	0.07
26	0.34	0.43	62	0.05	0.07
27	0.34	0.43	63	0.05	0.07
28	0.34	0.43	64	0.05	0.07
29	0.34	0.43	65	0.05	0.07
30	0.34	0.43	66	0.05	0.07
31	0.67	0.85	67	0.04	0.05
32	0.67	0.85	68	0.04	0.05
33	0.67	0.85	69	0.04	0.05
34	0.67	0.85	70	0.04	0.05
35	0.67	0.85	71	0.04	0.05
36	0.67	0.85	72	0.04	0.05



RAI No. 9 Figure 1: Incremental and Cumulative Probable Maximum Precipitation for the Basin above the Main Dam



RAI No. 9 Figure 2; Incremental and Cumulative Probable Maximum Precipitation for the Basin above the Auxiliary Dam

RAI No. 10. Streams and Rivers Flooding - E

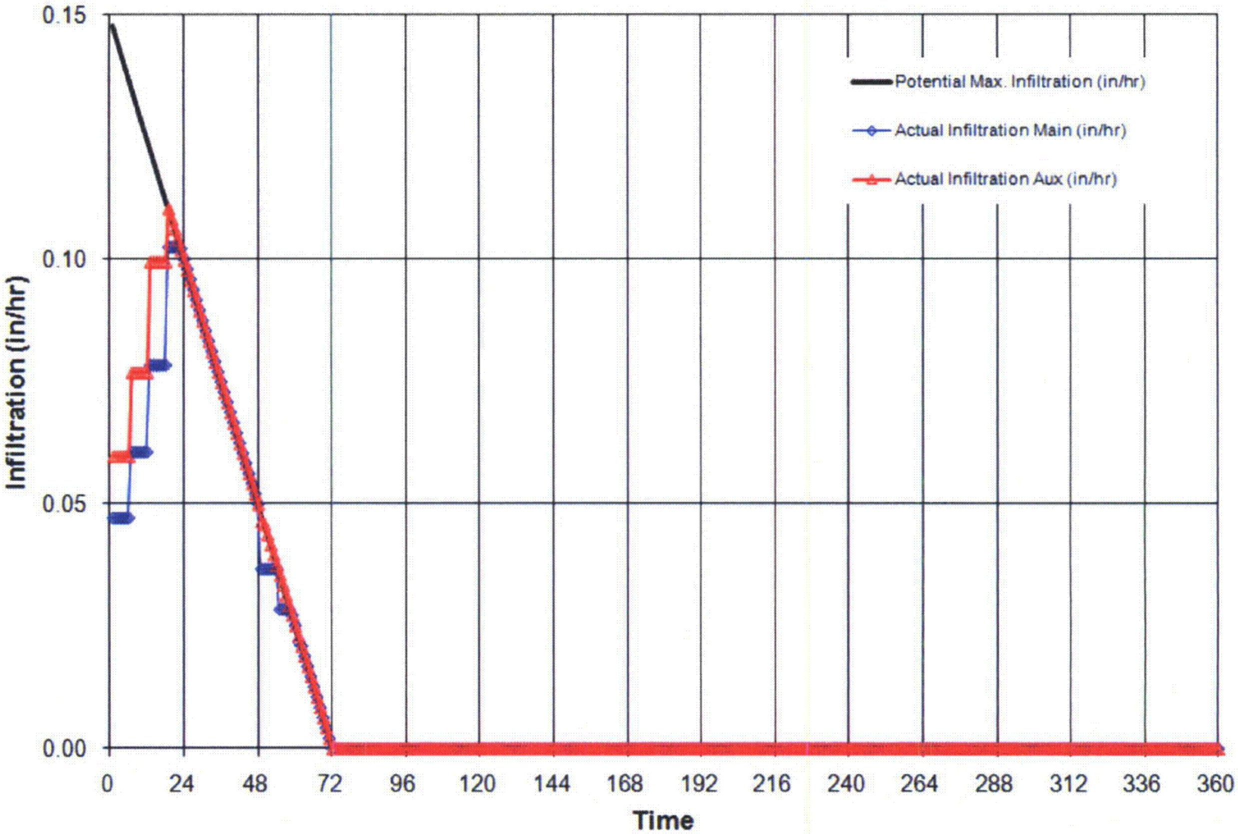
Background: Infiltration rates were estimated by the licensee to be in a range of 0.13 to 0.38 mm/hr (0.05 to 0.15 in./hr), with the larger value used as the initial infiltration rate. While the staff found that the infiltration calculation methods are appropriate, the conservatism of the infiltration rates needs to be demonstrated.

Request: Provide an explanation of how an initial infiltration rate of 0.38 mm/hr (0.15 in./hr) is conservative for a range of estimated infiltration rates of 0.13 to 0.38 mm/hr (0.05 to 0.15 in./hr) for the predominant soil type.

Response: The initial infiltration rate used in the PMF calculation is lower than 0.15 in/hr.

The equations used in section 3.2.2 of the FHRR give the maximum potential infiltration loss rate. 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 formula in section 3.2.2 of the FHRR, and (2) the rate of rainfall during a given hour of the 72-hour antecedent storm.

The figure below depicts the actual infiltration rate during the 72-hour antecedent storm and the two succeeding 72-hour periods of the PMP event for the design storms associated with both the Main Reservoir and the Auxiliary Reservoir drainage basins.



RAI No. 10 Figure 1: Potential and Actual Infiltration Losses

RAI No. 11, Streams and Rivers Flooding - F

Background: The geometric characteristics of the residual area (i.e., the land area that drains directly to the reservoirs) were presumably based on distances to the reservoir, though the method for developing them is not described by the licensee.

Request: Provide an explanation of the method used for hydrograph development of the residual area around the Main and Auxiliary Reservoirs.

Response: In accordance with HAR Units 2 and 3, COL Application, Final Safety Analysis Report, Rev. 4, Chapter 2 "Site Characteristics" Section 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):

$$t_L = CC_t\ (LL_C)^{0.3}$$

$$Q_p = (640\ C_p A)\ /\ t_L$$

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, 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 [Sub-basin numbers from HAR Units 2 and 3 COLA are different from the numbers used for the HNP Unit 1 analysis] 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 km^2 (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 2.4-260). 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 above. 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."

RAI No. 12. Streams and Rivers Flooding - G

Background: The licensee also states that the Auxiliary Dam spillway rating curve was taken from the HAR COLA (Progress Energy 2012); however, it is unclear whether the Auxiliary Dam spillway rating curve includes tailwater effects from discharge to the Main Reservoir.

Request: Provide an explanation of the effects of tailwater effects from Auxiliary Dam spillway discharges to the Main Reservoir and how they were incorporated into the rating curve of the Auxiliary Dam.

Response: The auxiliary dam spillway crest elevation is 252.00 ft NGVD29, the main dam PMF maximum stillwater level is 243.84 ft NGVD29. The Auxiliary Dam spillway crest elevation is 8.16 ft higher than the PMF maximum stillwater level in the Main Reservoir. Therefore, there are no tailwater effects from the Main Reservoir on to the Auxiliary Dam outflow discharge.

RAI No. 13, Streams and Rivers Flooding - H

Background: The licensee states that the primary discharge coefficient values from the HAR COLA are 3.95 and 3.92 for the Main Dam and the Auxiliary Dam, respectively. The licensee did a curve fit to the rating curve from HNP FSAR Figure 2.4.3-3 (PEC 2009) using a coefficient of 3.95 but it did not provide an adequate fit. By back calculation to the HNP rating curve, the licensee computed a primary discharge coefficient of 3.5.

Request: Provide an explanation for the discrepancy between the two discharge rating curve coefficients for the Main Dam spillway.

Response:

As explained in the FHRR section 3.2.3.4, the Main Dam spillway discharge coefficient C varies with the ratio of upstream dam height P to water depth above the spillway crest H_0 and with the ratio of total head H_e to design head H_0.

The coefficient C is also a function of the ogee crest coefficient C_0 which has a value of 3.95 for the Main Dam. The C_0 is the discharge coefficient C only when $H_e = H_0$.

Using the following weir equation (Q = C*L*He^1.5), the Main Dam spillway discharge coefficient "C" was calculated that fits the known discharge "Q" from Figure 2.4.3-4 of the HNP FSAR for pool level from 220.5 ft to 239.0 ft NGVD29. (RAI No. 13 Table 1)

The calculated spillway discharge coefficient "C" ranges from 3.2 for pool level of 220.5 ft to 3.50 for pool level 239.0 ft (RAI No. 13 Table 1). The spillway discharge coefficient "C" increases in value as head water increases from pool level 220.5 ft to 239.0 ft.

The Main Dam spillway discharge capacity was extended for pool level from 239.5 ft to 260.0 ft by applying the weir equation (Q = C*L*He^1.5) using the highest discharge coefficient "C" calculated above (3.50) which correspond to pool level 239.0 ft.

According to the weir equation (Q = C*L*He^1.5), the approach described above is conservative because using a lower discharge coefficient "C" equal to 3.50 instead of 3.95 results in a lower spillway discharge capacity "Q" for the Main Dam, causing the flood elevation to be higher.

RAI No. 13 Table 1. Present Main Dam Spillway Rating Curve Extension Calculation

(Figures 9.23 and 9.24 are from the Design of Small Dams Manual)

filename = HNPMainSpway.xlsx

Discharge Rating Curve for Main Dam Spillway

Existing Spillway, Crest El. 220, Extend Rating Curve up to El. 260

Effective Crest Length:

L = L'-2(NKp + Ka)He (COLA FSAR P. 2.4-31)

where L is net length = 50 feet
He = total head on spillway, =res. WL - 220
Kp = Ka = 0.01
N = number of piers = 1

Thus effective crest length = Leff = 50-2*0.02*(WL-220)

Q = C*L*He^1.5

C varies with head. From Design of Small Dams Figures 9.23 and 9.24,

P/Ho = 3; Co = 3.95 from COLA FSAR page 2.4-31

C = Co*(0.86242+ 0.13731*(He/Ho)^0.5)^2

However from the existing rating curve, HNP FSAR Figure 2.4.3-3, C is clearly less than 3.95. Original spillway rating curve calculation is not available. Find C that fits Figure 2.4.3-3 and use highest value to extend rating curve from El. 240 to El. 260.

Reservoir still water level in feet NGVD29

		Q read from FSAR Fig. 2.4.3-3		Assumed C to fit FSAR Figure 2.4.3-3	
Res. WL	He, ft.	Q(FSAR)	Leff, ft.	Cfsar	Calculated Q, cfs
220	0	0	50	0	0
220.5	0.5		49.98	3.2	57
221	1	200	49.96	3.2	160
221.5	1.5		49.94	3.2	294
222	2	500	49.92	3.2	452
222.5	2.5		49.9	3.2	631
223	3	820	49.88	3.2	829
223.5	3.5		49.86	3.2	1045
224	4	1270	49.84	3.2	1276
224.5	4.5		49.82	3.2	1522
225	5	1770	49.8	3.2	1782
225.5	5.5		49.78	3.2	2055
226	6	2350	49.76	3.2	2340
226.5	6.5		49.74	3.2	2638
227	7	3000	49.72	3.2	2947
227.5	7.5		49.7	3.2	3267
228	8	3600	49.68	3.2	3597
228.5	8.5		49.66	3.2	3938
229	9	4300	49.64	3.2	4289
229.5	9.5		49.62	3.2	4649
230	10	5150	49.6	3.3	5176
230.5	10.5		49.58	3.3	5567
231	11	5930	49.56	3.3	5967
231.5	11.5		49.54	3.3	6376
232	12	6800	49.52	3.3	6793
232.5	12.5		49.5	3.3	7219
233	13	7750	49.48	3.35	7769

RAI No. 13 Table 1. Present Main Dam Spillway Rating Curve Extension Calculation (cont.)

Res. WL	He, ft.	Q(FSAR)	Leff, ft.	Cfsar	Calculated Q, cfs	Storage, acre-feet
233.5	13.5		49.46	3.35	8219	138600
234	14	8700	49.44	3.35	8676	141400
234.5	14.5		49.42	3.38	9223	144200
235	15	9800	49.4	3.41	9786	147000
235.5	15.5		49.38	3.42	10306	149800
236	16	10850	49.36	3.43	10836	152600
236.5	16.5		49.34	3.45	11409	155400
237	17	12000	49.32	3.47	11996	158200
237.5	17.5		49.3	3.48	12560	161000
238	18	13100	49.28	3.49	13134	163800
238.5	18.5		49.26	3.5	13719	166600
239	19	14300	49.24	3.5	14273	169400
239.5	19.5		49.22	3.5	14834	172000
240	20		49.2	3.5	15402	174591
240.5	20.5		49.18	3.5	15977	
241	21		49.16	3.5	16558	
241.5	21.5		49.14	3.5	17146	
242	22		49.12	3.5	17740	
242.5	22.5		49.1	3.5	18341	
243	23		49.08	3.5	18948	
243.5	23.5		49.06	3.5	19561	
244	24		49.04	3.5	20181	
244.5	24.5		49.02	3.5	20806	
245	25		49	3.5	21438	
245.5	25.5		48.98	3.5	22075	
246	26		48.96	3.5	22718	
246.5	26.5		48.94	3.5	23367	
247	27		48.92	3.5	24022	
247.5	27.5		48.9	3.5	24682	
248	28		48.88	3.5	25348	
248.5	28.5		48.86	3.5	26019	
249	29		48.84	3.5	26696	
249.5	29.5		48.82	3.5	27378	
250	30		48.8	3.5	28065	
250.5	30.5		48.78	3.5	28758	
251	31		48.76	3.5	29456	
251.5	31.5		48.74	3.5	30159	
252	32		48.72	3.5	30867	
252.5	32.5		48.7	3.5	31581	
253	33		48.68	3.5	32299	
253.5	33.5		48.66	3.5	33022	
254	34		48.64	3.5	33750	
254.5	34.5		48.62	3.5	34484	
255	35		48.6	3.5	35221	
255.5	35.5		48.58	3.5	35964	
256	36		48.56	3.5	36711	
256.5	36.5		48.54	3.5	37463	
257	37		48.52	3.5	38220	
257.5	37.5		48.5	3.5	38981	
258	38		48.48	3.5	39747	
258.5	38.5		48.46	3.5	40517	
259	39		48.44	3.5	41292	
259.5	39.5		48.42	3.5	42072	
260	40		48.4	3.5	42855	

RAI No. 14, Streams and Rivers Flooding - H

Background: The licensee also used HEC-HMS to compute the probable maximum flood (PMF) water surface elevations in the Main and Auxiliary Reservoirs using level-pool routing of runoff along with the reservoir stage-storage and storage-discharge curves.

Request: Provide an explanation of the conservatism in using level-pool routing as opposed to hydraulic routing for the PMF elevation computations in the Main and Auxiliary Reservoirs.

Response:

From NUREG/CR-7046, PNNL-20091, "Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plant Sites in the United States of America," Section C.5 page C-17:

> *Although an approximate PMF water-surface elevation may be estimated if a stage-discharge relationship for the stream near the site is available, the preferred approach is to use the HEC-HMS estimated flood discharges as input to the USACE Hydrologic Engineering Center River Analysis System (HEC-RAS) model.*

Using the HEC-RAS model to calculate water-surface elevation is preferred for sites located near streams because HEC-RAS model accounts for velocity head losses, and HEC-HMS uses the storage-discharge, elevation-storage-discharge, and elevation-area-discharge methods to calculate water-surface elevation of the reservoir.

HEC-RAS calculates a slightly higher water-surface elevation in the Main Reservoir compared to the water-surface elevation computed by HEC-HMS. The following is a comparison of calculated water-surface elevation in the Main Reservoir using HEC-HMS and HEC-RAS (HAR Units 2 and 3, COL Application, Final Safety Analysis Report, Rev. 4, Chapter-2 "Site Characteristics," Section 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 Main Reservoir within HEC-HMS model, PMF stillwater elevations. The peak stillwater elevation in the Main Reservoir was determined to be 252.48 ft NGVD29.
>
> The peak stillwater elevation using the HEC-RAS model run for the PMF is 252.76 ft NGVD29.
>
> From the result caparison above, HEC-RAS model water-surface elevation in the Main Reservoir is 0.28 ft higher than water-surface elevation computed by HEC-HMS. (See Table 2.4.3-227 of HAR Units 2 and 3, COLA).

HAR site grade is at elevation 260.0 ft NGVD29 which is about 16.0 ft above stillwater level in the Main Reservoir. Therefore, while slightly less conservative, calculating water-surface elevation using HEC-HMS model is appropriate.

RAI No. 15, Streams and Rivers Flooding - I

Background: Staff examined the stage capacity curve for the Main Reservoir (HNP FSAR Figure 2.4.3-6; PEC 2009) and found that it only extends up to an elevation of 76.2 m (250 ft). Staff could not find a figure or table corresponding to the extension of the Main Dam spillway rating curve.

Request: Clarify if the reservoir capacity curve was extended to 79.2 m (260 ft) as done for the spillway discharge rating curve. If the capacity curve was not extended, provide an explanation of the method used to handle level-pool routing if the stage exceeded 76.2 m (250 ft).

Response: The Main Reservoir stage capacity curve is extended to elevation 260 ft NGVD29, see RAI No. 15 Table 1.

RAI No. 15 Table 1: Stage-Storage Rating Curve for Main Reservoir

Ordinate	ELEVATION	STORAGE
Labels		
Units	FT	ACRE-FT
Type	UNT	UNT
1	215.0	55,000.0
2	220.0	73,201.0
3	222.0	80,805.0
4	224.0	89,024.0
5	226.0	97,915.0
6	228.0	107,460.0
7	230.0	117,739.0
8	232.0	128,793.0
9	234.0	140,605.0
10	236.0	153,187.0
11	238.0	166,597.0
12	240.0	174,591.0
13	242.0	189,433.0
14	244.0	205,261.0
15	246.0	222,121.0
16	248.0	240,059.0
17	250.0	259,122.0
18	252.0	279,359.0
19	254.0	300,819.0
20	256.0	323,552.0
21	258.0	347,609.0
22	260.0	373,041.0

RAI No. 16, Streams and Rivers Flooding- J

Background: In its review of the FHRR, the staff noted that the licensee did not provide full citations of technical references used in the flooding analysis for streams and rivers. A full citation to the HNP Unit 1 FSAR and the HAR Units 2 and 3 COLA are also missing.

Request: Provide full citations to technical references, to the HNP Unit 1 FSAR, and to the HAR Units 2 and 3 COLA used in the FHRR.

Response: The full technical references used in the FHRR for Streams and Rivers Flooding are shown below:

1. ANSI/ANS 2.8, "American National Standard for Determining Design Basis Flooding at Power Reactor Sites", Prepared by the American Nuclear Society Standards Committee Working Group ANS-2.8, La Grange Park, Illinois, 1992.

2. SHNPP Unit 1 Final Safety Analysis Report (FSAR) Amendment No. 58, Sections 2.4 and 3.4

3. HAR Units 2 and 3, COL Application, Final Safety Analysis Report Rev. 4, Chapter 2: Site Characteristics

4. Calculation No. HAG-0000-X7C-003, Rev. 6, Calculation Title & Description: Probable Maximum Flood (PMF) on Streams and Rivers, July 14, 2011.

5. Calculation No. HAG-0000-X7C-031, Rev. 2, Calculation Title & Description: Probable Maximum Precipitation (PMP) Estimate for the Buckhorn Creek Drainage Basin above the Main Reservoir, HAR Site, North Carolina, January 20, 2011.

6. Hydrologic Engineering Center, 2010, "Hydrologic Modeling System HEC-HMS, User's Manual," Version 3.5, August 2010, US Army Corps of Engineers, Davis, CA.

7. Hydrometeorological Report No. 51, "Probable Maximum Precipitation Estimates, United States East of the 105th Meridian". U.S. Department of Commerce, National Oceanic and Atmospheric Administration, June 1978.

8. Hydrometeorological Report No. 52, "Application of Probable Maximum Precipitation Estimates, United States East of the 105th Meridian". U.S. Department of Commerce, National Oceanic and Atmospheric Administration, August1982.

9. NUREG/CR-7046, PNNL-20091, "Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plant Sites in the United States of America.

10. U.S. Nuclear Regulatory Commission, 2012, "Request for Information Pursuant to Title 10 CFR 50.54(f) Regarding Recommendations 2.1, 2.3 and 9.3, of the Near-Term Task Force Review of Insights from the Fukushima Dai-Ichi Accident," Eric Leeds and Michael Johnson, March 12, 2012.

11. Post-Fukushima Near-Term Task Force Recommendation 2.1: Supplemental Guidance for the Evaluation of Dam Failures, Draft Rev. B, 8-21-2012.

12. ML12097A509, letter from NRC, "Prioritization of Response Due Dates for Request For Information Pursuant to Title 10 of The Code of Federal Regulations 50.54(F) Regarding Flooding Hazard Reevaluations for Recommendation 2.1 of The Near-

Term Task Force Review of Insights From The Fukushima Dai-Ichi Accident", May 11, 2012

13. U.S. Department of the Interior, Bureau of Reclamation (USBR), "Design of Small Dams, a Water Resources Technical Publication," Third Edition, 1987

RAI No. 17, Storm Surge Flooding

Background: In its review of the FHRR, the staff noted that the licensee cited the Nuclear Energy Institute (NEI) white paper Post-Fukushima Near-Term Task Force Recommendation 2.1, Supplemental Guidance for the Evaluation of Dam Failures, Rev. B, for the estimation of maximum wave run-up.

Request: Explain how the use of the NEI white paper fulfills the requirement specified in the March 12, 2012 50.54(f) letter to use current applicable Commission requirements and guidance in performing the flood hazard re-evaluation of storm surge.

Response: Reg Guide 1.59, NUREG-0800 and NUREG/CR-7046 were reviewed for guidance on failure of downstream dams. The NEI white paper provided additional information and was used as discussed below. Reg Guide 1.59, NUREG-0800 and NUREG/CR-7046 address upstream dam failure which is not a flooding hazard for HNP. The NEI white paper was the most applicable guidance at the time of submittal.

From section 3.3.4 of the FHRR

Wave runup can be calculated for waves of 10%, 2%, 1% and 0.1% frequency, meaning the percent of waves in a given wave train that reach that elevation. According to the NEI white paper Post-Fukushima Near-Term Task Force Recommendation 2.1 Supplemental Guidance for the Evaluation of Dam Failures, Draft, Rev. B, maximum wave runup should be taken as the 1% wave, meaning the highest of 100 waves.

From NEI white paper, page 6:

This paper is intended to clarify how dam failure should be considered when reevaluating the bounding PMF in response to Enclosure 2 (Recommendation 2.1: Flooding) of the March 12, 2012 50.54(f) letter. This paper provides added detailed guidance to supplement the NUREG/CR-7046, Sections 3.4 and 3.9 and Appendix H.2, related to dam failure considerations

Also, from NEI white paper, page 12:

Dam overtopping may be investigated for these two conditions:

- *Probable maximum flood surcharge level plus maximum (1%) average height resulting from sustained 2-year wind speed applied in the critical direction; or*
- *Normal operating level plus maximum (1%) wave height based on the probable maximum gradient wind*

RAI No. 18, Hazard Input for the Integrated Assessment - A

Background: The March 12, 2012, 50.54(f) letter, Enclosure 2, requests the licensee to perform an integrated assessment of the plant's response to the re-evaluated hazard if the re-evaluated flood hazard is not bounded by the current design basis.

Request: The licensee is requested to provide the applicable flood event duration parameters (see definition and Figure 6 of the Guidance for Performing an Integrated Assessment, JLD-SG-2012-05) associated with mechanisms that trigger an Integrated Assessment. This includes (as applicable) the warning time the site will have to prepare for the event, the period of time the site is inundated, and the period of time necessary for water to recede off the site for the mechanisms that are not bounded by the current design basis. The licensee is also requested to provide a basis for the flood event duration parameters. The basis for warning time may include information from relevant forecasting methods (e.g., products from local, regional, or national weather forecasting centers).

Response: In accordance with Table 6 of the HNP FHRR, the flooding mechanisms that are not bounded by the CLB and require an integrated assessment are: Local Intense Precipitation (at Waste Process Building (WPB) and Diesel Fuel Oil Storage Tank Building (DFOSB)), Flooding in Streams and Rivers (at the Main Dam, Auxiliary Dam and Plant Island), Storm Surge (at the Auxiliary Dam) and Combined Effects (at the Main Dam and Auxiliary Dam).

For the flooding mechanism On-Site Local Intense Precipitation (PMP) based Probable Maximum Flood (PMF), the flood event duration parameters are as follows:

- Warning time is 0 hours as local PMP could be caused by a tropical storm or frontal precipitation. It is assumed, for conservatism that the storm occurs without warning. The source is National Weather Service, Weather Forecasting Center.
- The period of time that the site is inundated is 1 hour, based on the 1-hour PMP rainfall duration obtained from the HMR-51, and HMR-52 (NOAA, 1978 & 1982).
- The period of time necessary for water to recede is up to 1-hour, based on the 1-hour PMP event duration.

For the flooding mechanism Flooding in Streams and Rivers, the flood event duration parameters are as follows:

- Warning time is 36 hours in accordance with the National Weather Service, National Hurricane Center.
- The reevaluated flood level is below the protected level; therefore, the site is not inundated.

For the flooding mechanism Storm Surge (PMSS), the flood event duration parameters are as follows:

- Warning time is 36 hours in accordance with the National Weather Service, National Hurricane Center.
- The reevaluated flood level is below the protected level; therefore, the site is not inundated.

For the flooding mechanism Combined Effects, the flood event duration parameters are as follows:

- Warning time is 36 hours in accordance with the National Weather Service, National Hurricane Center.
- The reevaluated flood level is below the protected level; therefore, the site is not inundated.

RAI No. 19, Hazard Input for the Integrated Assessment - B

Background: The March 12, 2012, 50.54(f) letter, Enclosure 2, requests the licensee to perform an integrated assessment of the plant's response to the re-evaluated hazard if the re-evaluated flood hazard is not bounded by the current design basis.

Request: Provide the flood height and associated effects (as defined in Section 9 of JLD-ISG-2012-05) that are not described in the flood hazard re-evaluation report for mechanisms that trigger an Integrated Assessment. This includes the following quantified information for each mechanism (as applicable):

- Hydrodynamic loading, including debris,
- Effects caused by sediment deposition and erosion (e.g., flow velocities, scour),
- Concurrent site conditions, including adverse weather, and
- Groundwater ingress.

Response: In accordance with Table 6 of the HNP FHRR, the flooding mechanisms that are not bounded by the CLB and require an integrated assessment are: Local Intense Precipitation (at the Waste Process Building (WPB) and Diesel Fuel Oil Storage Tank Building (DFOSB)), Flooding in Streams and Rivers (at the Main Dam, Auxiliary Dam and Plant Island), Storm Surge (at the Auxiliary Dam) and Combined Effects (at the Main Dam and Auxiliary Dam).

For the flooding mechanism On-Site Local Intense Precipitation (PMP) based Probable Maximum Flood (PMF), the flood height and associated effects are as follows:

- Site grade level is 261 ft. The flood elevation at WPB and the DFOSB is 261.36 ft and 261.41 ft, respectively. This gives a flood height of 0.36 ft at the WPB and 0.41ft at the DFOSB.

- Hydrodynamic Impact Forces and Hydrostatic Pressure:

  The only buildings under the potential risk of increased flood impact force are the WPB and the DFOSB. The impact force (Hydrodynamic Loading) is calculated by the following equation (Reference 1):

  $$P_i = k\rho_f V^2 \qquad \text{(Eq.1)}$$

  Where Pi is the impact force in lb/ft, coefficient k is 1.28 for both English and SI units, ρf = water density in (62.42 lb/ft^3) and V is the maximum velocity ft/sec regardless of direction.

The maximum hydrostatic force is the hydrostatic pressure at the bottom of the building wall as given by the following equation:

$$P_s = \gamma \cdot h \qquad \text{(Eq. 2)}$$

Where: P_s is the hydrostatic pressure (lb/ft^2); γ is the specific weight of water (62.42 lb/ft^3) and h is the water depth for the flood.

The maximum velocity adjacent to the WPB is 0.85 ft/sec and the velocity for the DFOSB is 0.06 ft/sec. Thus, the impact force on the building walls is given by Eq.1 as follows:

P_i = 1.28 * 62.4 * 0.85^2 = 57.7 lb/ft at the WPB

P_i = 1.28 * 62.4 * 0.06^2 = 0.29 lb/ft at the DFOSB

The hydrostatic pressure on the building walls is obtained from Eq. 2 as follows:

Ps = 62.4 * 0.36 = 22.5 lb/ft^2 at the WPB

Ps = 62.4 * 0.41 = 25.6 lb/ft^2 at the DFOSB

Both forces are smaller than the strength of concrete 3,000 pound per square inch or 432,000 pound per square foot.

(Reference 1): Deng, Z., 1996. "Impact of Debris Flows and Its Mitigation," Ph.D. Dissertation submitted to the Dept. of Civil and Environmental Engineering, Univ. of Utah, Salt Lake City, Utah.

- Debris: There is no source of debris as the site is higher in elevation than the surrounding areas and the site surface cover is either pavement, concrete, or gravel.
- Sediment deposition or erosion: There is no source of sediment as the site is higher in elevation than the surrounding areas and the site surface cover is either pavement, concrete, or gravel. Also, the flow velocity generated from the On-Site Local Intense Precipitation (PMP) based Probable Maximum Flood (PMF) ranges from 0.06 ft/sec to 1.27 ft/sec. This velocity range is lower than 2.5 ft/sec critical/permissible velocities for bare soil (Soil and Water Conservation Engineering, Schwab, *et al.)*
- No adverse weather impact to the site.
- No groundwater impact.

The following information is obtained from, HAR Units 2 and 3, COL Application, Final Safety Analysis Report, Rev. 4, Chapter 2 "Site Characteristics," Section 2.4.12.1.2.2 Triassic Rocks:

1. Before the HNP was constructed, groundwater elevations across the site occurred principally within jointed, fractured bedrock, generally at depths of 9.1 to 27.4 m (30 to 90 ft) below the original ground surface.

2. Following completion of the HNP, surface water pressure from the Main Reservoir and the Auxiliary Reservoir created new groundwater elevations at the HAR site. Groundwater depths now range between 0.6 and 9.1 m (2 and 30 ft) below ground surface.

Though the flood mechanisms Flooding in Streams and Rivers, Storm Surge (PMSS), and Combined Effects require an Integrated Assessment, the reevaluated flood levels are below the protected levels; therefore, hydrodynamic loading, effects caused by sediment deposition and erosion, concurrent site conditions, and groundwater ingress do not apply.