# Exelon Generation.

~~Security-Related Information – Withhold Under 10 CFR 2.390~~

10 CFR 50.54(f)

RS-14-173

July 3, 2014

U.S. Nuclear Regulatory Commission
ATTN: Document Control Desk
Washington, DC 20555-0001

Quad Cities Nuclear Power Station, Units 1 and 2
Renewed Facility Operating License Nos. DPR-29 and DPR-30
NRC Docket Nos. 50-254 and 50-265

Subject: Response to Request for Additional Information Regarding Fukushima Lessons Learned - Flood Hazard Reevaluation Report

References:

1. Exelon Generation Company, LLC Letter to USNRC, Response to March 12, 2012 Request for Information Enclosure 2, Recommendation 2.1, Flooding, Required Response 2, Flooding Hazard Reevaluation Report, dated March 12, 2013 (RS-13-047)
2. 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
3. NRC Letter, Request for Additional Information Regarding Fukushima Lessons Learned-Flood Hazard Reevaluation Report, dated June 25, 2014

In Reference 1, Exelon Generation Company, LLC (EGC) provided the Quad Cities Nuclear Power Station, Units 1 and 2, Flooding Hazard Reevaluation Report in response to the March 12, 2012 Request for Information Enclosure 2, Recommendation 2.1, Flooding, Required Response 2, (Reference 2).

The purpose of this letter is to provide the response to the NRC request for additional information (RAI) (Reference 3) regarding the Quad Cities Nuclear Power Station, Units 1 and 2 Flooding Hazard Reevaluation Report. Enclosure 1 provides the response to each NRC RAI. Enclosures 2 and 3 provide the electronic information files requested by the respective RAIs.

Enclosure 2 to this letter contains Sensitive Unclassified Non-Safeguards Information (SUNSI) and the information should be withheld from public disclosure in accordance with the requirements of 10 CFR 2.390. Enclosure 2 has been marked accordingly with the notation *"Security-Related Information – Withhold Under 10 CFR 2.390."*



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NRR

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This letter contains no new regulatory commitments. If you have any questions regarding this report, please contact Ron Gaston at (630) 657-3359.

I declare under penalty of perjury that the foregoing is true and correct. Executed on the 3rd day of July 2014.

Respectfully submitted,

Glen T. Kaegi

Glen T. Kaegi
Director - Licensing & Regulatory Affairs
Exelon Generation Company, LLC

Enclosures:

1. Quad Cities Nuclear Power Station, Units 1 and 2 - Response to Request for Additional Information Regarding Fukushima Lessons Learned – Flood Hazard Reevaluation Report
2. DVD # 1 of RS-14-173 for RAI Response No. 4 – Regarding Fukushima Lessons Learned-Flood Hazard Reevaluation Report
3. DVD # 2A of RS-14-173 for RAI Response No. 11, 12, 13, 15, 17, 20, 22, and 23– Regarding Fukushima Lessons Learned-Flood Hazard Reevaluation Report
   DVD # 2B of RS-14-173 for RAI Response No. 15– Regarding Fukushima Lessons Learned-Flood Hazard Reevaluation Report
   DVD # 2C of RS-14-173 for RAI Response No. 15– Regarding Fukushima Lessons Learned-Flood Hazard Reevaluation Report
   DVD # 2D of RS-14-173 for RAI Response No. 15– Regarding Fukushima Lessons Learned-Flood Hazard Reevaluation Report
   DVD # 2E of RS-14-173 for RAI Response No. 15– Regarding Fukushima Lessons Learned-Flood Hazard Reevaluation Report
   DVD # 2F of RS-14-173 for RAI Response No. 15– Regarding Fukushima Lessons Learned-Flood Hazard Reevaluation Report

~~*Security-Related Information – Withhold Under 10 CFR 2.390*~~

cc: Director, Office of Nuclear Reactor Regulation
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# Enclosure 1

**Quad Cities Nuclear Power Station, Units 1 and 2**

**Response to Request for Additional Information**

**Regarding Fukushima Lessons Learned - Flood Hazard Reevaluation Report**

(55 pages)

**RAI 1: Hazard Input for the Integrated Assessment – Flood Event Duration Parameters**

Background: Enclosure 2 of the 50.54(f) letter requests the licensee to perform an integrated assessment of the plant's response to the reevaluated hazard if the reevaluated flood hazard is not bounded by the current design basis. Flood scenario parameters from the flood hazard reevaluation serve as the input to the integrated assessment. To support efficient and effective evaluations under the integrated assessment, NRC staff will review flood scenario parameters as part of the flood hazard reevaluation and document results of the review as part of the NRC staff assessment of the flood hazard reevaluation.

Request: Provide the applicable flood event duration parameters (see definition and Figure 6 of the NRC interim staff guidance document JLD-ISG-2012-05, "Guidance for Performing an Integrated Assessment," November 2012 (ADAMS Accession No. ML12311A214), associated with mechanisms that trigger an integrated assessment using the results of the flood hazard reevaluation. This includes (as applicable) the warning time the site will have to prepare for the event (e.g., the time between notification of an impending flood event and arrival of floodwaters on site) and the period of time the site is inundated for the mechanisms that are not bounded by the current design basis. The licensee is also requested to provide the basis or source of information for the flood event duration, which may include a description of relevant forecasting methods (e.g., products from local, regional, or national weather forecasting centers) and/or timing information derived from the hazard analysis.

**Response:**

The flood event duration parameters shown in Figure 1.1 are determined for the critical flood causing mechanism in terms of maximum water surface elevation and fastest arrival time. For the Quad Cities Nuclear Power Station (QCNPS), the flooding scenarios with coincident dam failures (i.e., Probable Maximum Flood (PMF) + hydrologic dam failure) bound the flooding scenarios without dam failures both for critical timing and maximum water surface elevation. After evaluating the US Army Corps of Engineers, Hydrologic Engineering Center, Riverine Analysis System (HEC-RAS) results for all the storm centers that flood the QCNPS (i.e., flood elevation greater than an elevation of 595.0 feet MSL 1912), the controlling PMF in terms of maximum water surface elevation and the fastest arrival time is the cool-season PMP centered over the McGregor sub-watershed, snowmelt from an antecedent 100-year snowpack, and hydrologic dam failure.

Flood event durations are calculated as the temporal difference between the end of the Probable Maximum Precipitation (PMP) and arrival time of flood water at QCNPS for various flood causing mechanisms and critical flood elevations.

Flood event duration parameters are based on the HEC-RAS model developed to support the flood hazard analysis of the QCNPS. The PMF flood hydrograph for the flood event duration parameters calculation was obtained at HEC-RAS cross section 506.9 (Mississippi River, Reach #13), which crosses the northern portion of QCNPS.

The controlling PMF stillwater elevation (PMF+ hydrologic dam failure) is 600.9 feet MSL 1912. QCNPS enters flood emergency procedure immediately when the river level exceeds 586 feet or when the river level is predicted to be greater than 594.0 feet MSL 1912 in less than 96 hours.

Flood event duration parameters for the controlling flooding mechanism for elevations above 595.0 feet MSL 1912 are calculated based on the shape of the PMF hydrograph. Flood event duration parameters for the controlling flood causing mechanism (PMF+ hydrologic dam failure) are summarized as follows:

1. The flood duration from the end of the PMP to elevation 595.0 feet MSL 1912 is 172 hours according to the HEC-RAS output time series table for the controlling PMF simulation (PMF + hydrologic dam failure).

2. The duration of inundation above elevation 595.0 feet MSL 1912 is 240 hours according to the HEC-RAS output time series table for the controlling PMF simulation (PMF + hydrologic dam failure).

As identified by JLD-ISG-2012-05, the critical warning time, inundation time, and recession time for the controlling flooding mechanism (PMF+ hydrologic dam failure) are provided in Figure 1.1 and Table 1.1. Timing parameters for other analyzed flood causing mechanisms are summarized in Table 1.1 as well.

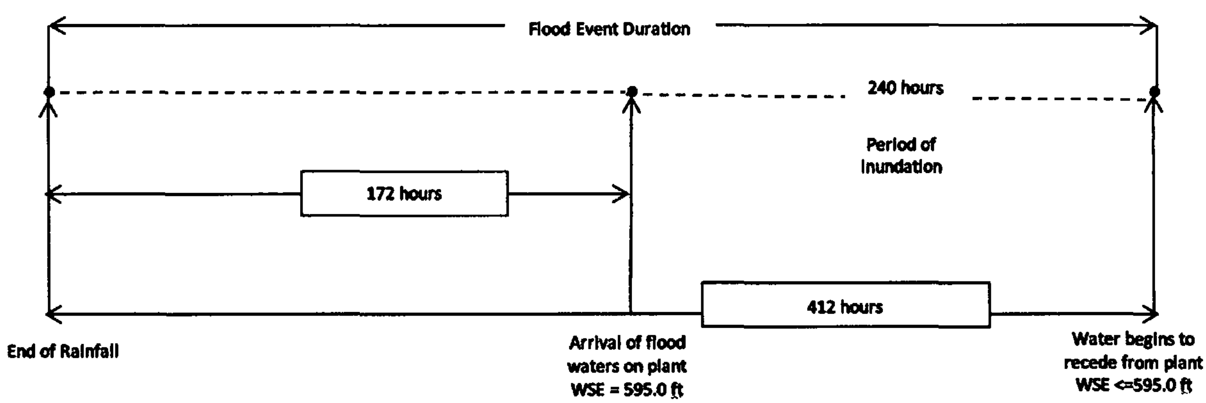



**Figure 1.1 - Flood Event Duration Parameters (Elevations in MSL 1912)**

**Table 1.1 – Timing Parameters for Various Flood Mechanisms (Elevations in MSL 1912)**

| | Critical Flood Criteria | Corresponding Conservative Flood Scenario | Parameter Value |
|---|---|---|---|
| **1** | **Flooding in Streams and Rivers (HEC-RAS Modeling)** | | |
| 1a | Time to reach elevation 595.0 feet from the end of the PMP | Cool-season PMP centered over the McGregor sub-watershed, with snowmelt from an antecedent 100-year snowpack | 173 hours |
| 1b | Time to reach the highest water surface elevation at the plant of 600.5 feet | | 256 hours |
| 1c | Duration of inundation above elevation of 595.0 feet | | 238 hours |
| **2** | **PMF with Hydrologic Dam Failures (HEC-RAS Modeling)** | | |
| 2a | Time to reach elevation 595.0 feet from the end of the PMP | | 172 hours |
| 2b | Time to reach the highest water surface elevation at the plant of 600.9 feet | | 250 hours |
| 2c | Duration of inundation above elevation of 595.0 feet | | 240 hours |
| 3 | **Seismic Dam Failure (HEC-RAS Modeling)** | | |
| 3a | Time to reach the highest water surface elevation at the plant of 589.6 feet | | 256 hours |
| 3b | Time to reach elevation 595.0 feet from the end of the PMP | | N/A* |

* Elevation of 595.0 feet is not reached due to seismic dam failure

**RAI 2: Hazard Input for the Integrated Assessment – Flood Height and Associated Effects**

Background: Enclosure 2 of the 50.54(f) letter requests the licensee to perform an integrated assessment of the plant's response to the reevaluated hazard if the flood hazard is not bounded by the current design basis. Flood scenario parameters from the flood hazard reevaluation serve as the input to the integrated assessment. To support efficient and effective evaluations under the integrated assessment, NRC staff will review flood scenario parameters as part of the flood hazard reevaluation and document results of the review as part of the staff assessment of the flood hazard reevaluation.

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 reevaluation 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:**

a. Flood Height

The maximum flood elevation for the local intense precipitation (LIP) event, as reported in the March 12, 2103 submittal, ranges between 593.74 and 597.81 feet NAVD-88. The LIP analysis was conducted using a two dimensional model. Therefore, maximum flood elevations vary throughout the plant. More details on the results are provided with the March 12, 2013 submittal.

The maximum (stillwater) flood elevation for the Combined Effects river flood (including upstream dam failure), as reported in the March 12, 2103 submittal, is 600.9 feet MSL 1912.

b. Wind Wave and Runup Effects

Consideration of wind-wave action for the LIP event is not explicitly required by NUREG/CR-7046 and is judged to be a negligible associated effect because of limited fetch lengths and flow depths.

The maximum wind-wave runup elevation for the Combined Effects river flood, as reported in the March 12, 2103 submittal, is 605.0 feet MSL 1912.

c. Hydrodynamic/debris loading

The LIP analysis resulted in hydrodynamic loads ranging between 0.01 and 271.83 lbs/foot width. The LIP analysis was conducted using a two dimensional model. Therefore, hydrodynamic loads vary throughout the plant. More details on the results are provided with the March 12, 2013 submittal. The debris load for the LIP event is negligible due to low velocities and depths, which results in a lack of power to transport heavy debris.

During the Combined Effects river flood, Quad Cities Station allows flood waters to enter and fill up the plant to equalize the associated hydrostatic loading. Therefore, during the river flood, the structural loading to consider is limited to the hydrodynamic and debris loading. Hydrodynamic and debris impact loading during the governing probable maximum flood (PMF) scenario were evaluated in Calculation QDC-0085-S-2034. The hydrodynamic forces for low velocity flow (less than 10 feet per second) are converted into an equivalent hydrostatic force. Section 7.3.1 of Calculation QDC-0085-S-2034 reports a hydrodynamic load of 3.6 lbs/ft width, equivalent static force corresponding to the PMF at elevation 598 feet MSL 1912 for Approach 1 and 4.1 lbs/ft acting at elevation 598.9 feet MSL 1912 for Approach

2. (See below for description of two dam failure approaches.) Debris impact loading was analyzed using the guidelines described in FEMA P-259 and by considering debris weight recommended in ASCE/SEI-7-10 (1000 lbs). Section 7.3.2 of Calculation QDC-0085-S-2034 reports a debris impact load of 480 lbs.

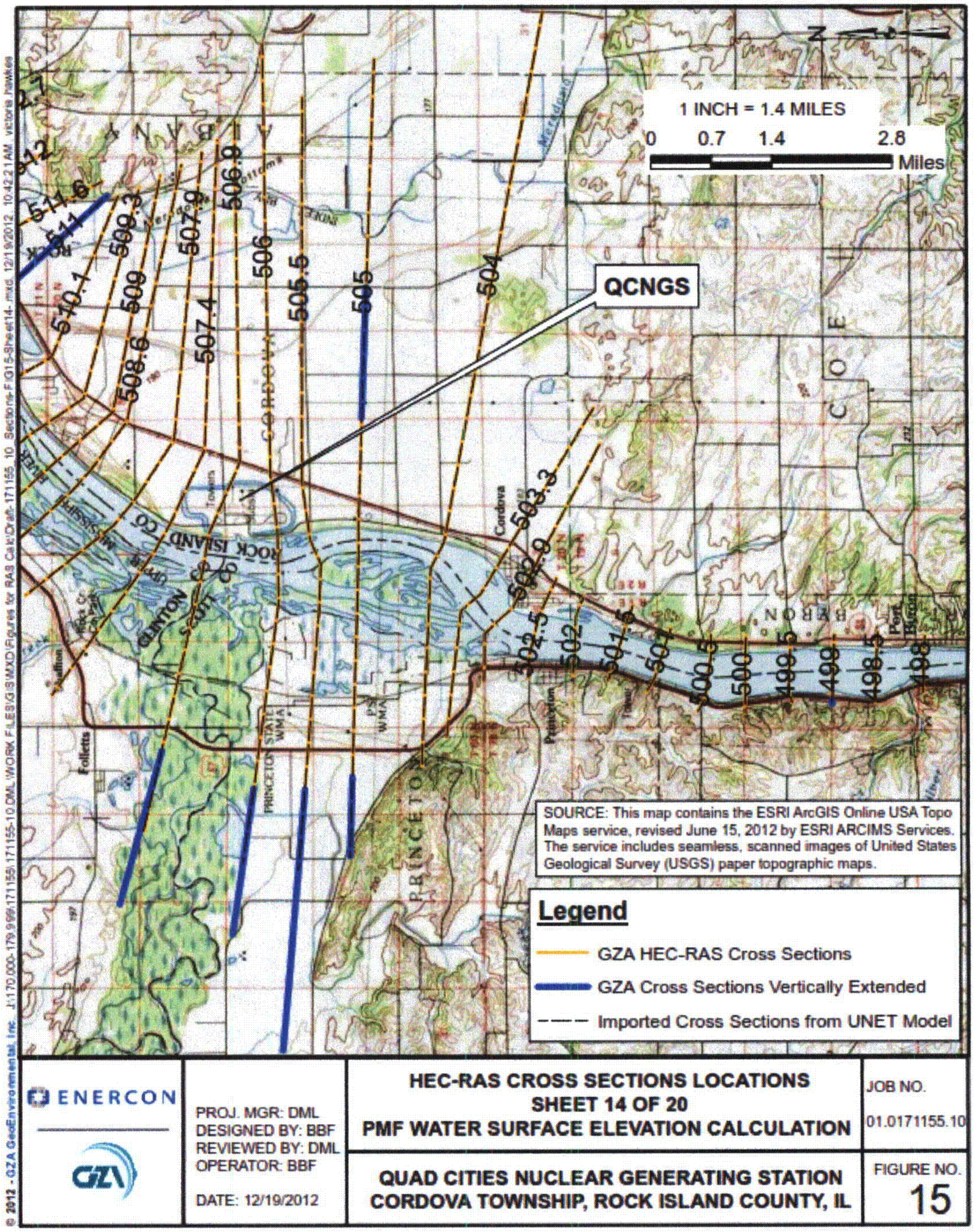
Please note that two approaches were considered to evaluate flooding from upstream dam failure. Approach 1 considered failure of a subset of the upstream dams to develop a conservative but representative upstream dam failure scenario based on ANSI/ANS 2.8 guidance, which states that some dams can be eliminated from dam failure analysis based on "*low head differential, small volume, distance from plant site, and major intervening natural or reservoir detention capacity.*" Smaller and more remote dams, judged to be unlikely to significantly contribute to flooding at the site, were excluded in Approach 1. Approach 2 was applied for sensitivity purposes only and introduced additional conservatisms by evaluating failure of all 1,558 upstream dams within the watershed, represented in the hydrologic model as hypothetical dams. The NRC Dam Failure Interim Staff Guidance (July 31, 2013) was released after Quad Cities Flood Hazard Reevaluation Report was submitted and, therefore, was not applicable to the reevaluation. The approach was developed based on ongoing discussions, at the time the reevaluation was being conducted, of dam failure methodology between the NRC and the Nuclear Energy Institute (NEI). The information provided in response to this Request for Information is based on results from Dam Failure Approach 1.

The Mississippi River is a navigable water body and, therefore, the potential for a barge to impact critical structures during the PMF event was assessed. As shown in Table 2.1, velocities in the main channel are much higher than overbank velocities at the site. Also, as shown in Figure 2.1, the plant is located in the left (looking downstream) overbank and inner curve of bend in the river channel. The velocity differential and configuration of the river at the site indicates that inertia alone would keep floating barges in the main channel, making any direct or indirect strike on critical structures not credible. Furthermore, topographic features between the site and river would protect the plant from barge impacts.

**Table 2.1: Summary of Channel and Overbank Velocities (for PMF + Dam Failure Approach 1)**

| River | Reach # | River Station | Average Velocity- Left (looking downstream) Overbank (fps) | Average Channel Velocity (fps) |
|---|---|---|---|---|
| Mississippi | 13 | 506.9 | 0.55 | 4.94 |
| Mississippi | 13 | 506.0 | 0.07 | 3.55 |

**Figure 2.1: HEC-RAS Cross-Section Locations (Calculation # QDC-0085-S-1991, Attachment 1, Figure 15)**




d. Effects of sediment deposition and erosion

The sediment supply is expected to limit the amount of deposition that could occur during a LIP event. Velocities around the plant during the LIP event range between 0.19 and 6.29 feet per second (fps). The maximum velocity is well below permissible velocities for paved

surfaces, which is the dominant surface in the power block area, of 12 to 30 fps (U.S. Army Corps of Engineers (USACE), Engineer Manual EM 1110-3-136, Drainage and Erosion Control Mobilization Construction, April 1984) so erosion and localized scour is also not expected to be a significant effect of LIP flooding.

A detailed sediment transport analysis was not performed as part of the flood hazard reevaluation for the PMF river flood. However, a qualitative evaluation was conducted to assess the potential impacts of sediment deposition and erosion on flooding and at plant structures. Table 2.1 shows a reduction in the left overbank velocity from the upstream to downstream side of the plant, indicating that some deposition of sediment may occur during the flood. However, the magnitude of the left overbank velocities are low and expected to only transport very fine particle sizes (e.g. clay) with very low settling velocities. Therefore, deposition is expected to be minimal and would not affect flood levels at the site. The low overbank velocities, shown in Table 2.1, also indicate that scour and erosion is not expected, even where localized eddies form around plant structures. The site is largely covered with asphalt and concrete pavement, which is able to withstand velocities between 12 and 30 fps (USACE 1984). Even the most erodible bare soil can withstand velocities up to 2 fps or more (USACE 1984), which is still much greater than the applied velocities.

e. Concurrent site conditions

The meteorological events that could potentially result in significant rainfall of the LIP and probable maximum precipitation (PMP) magnitude are squall lines, thunderstorms with capping inversion, and mesoscale convective systems. These meteorological events are typically accompanied by hail, strong winds, and even tornadoes. The flood hazard reevaluation calculations indicate that the site is also subject to flooding from a rain-on-snow event in the watershed, which can produce concurrent high winds, ice, and snow conditions on the site. The riverine PMF can be accompanied with debris loads, which may impact site accessibility once the flood waters recede.

f. Groundwater ingress

During a LIP event, impervious cover immediately around the power block buildings and the short-duration (1-hour precipitation) will keep infiltration of precipitation and groundwater seepage to a minimum. Therefore, groundwater level changes are not expected to occur during a LIP flood.

The river flood may cause groundwater levels to surcharge at the plant. The impact of this surcharge on the plant's ability to protect against the ingress from surcharged groundwater levels will be evaluated in the Integrated Assessment, which will consider groundwater levels to rise with the river flood up to plant grade.

**RAI 3: Site Information**

Background: The March 12, 2013, letter (Section c) and FHRR (Sections 2.a and 4) state that the current design-basis flood elevation is 603 feet mean sea level (MSL, 1912 datum) and that this corresponds to the probable maximum flood (PMF) elevation (FHRR, Section 4, Table 1). However, Section 4.a of the flooding walkdown report for QCNPS (ADAMS Accession No. ML12332A307) states that the original design-basis flood elevation is 589 ft MSL, which was

based on the 200-year flood that was considered to be the PMF at the time of the plant design. The flooding walkdown report further states that floods in excess of the 200-year event are plausible and notes that the updated final safety analysis report provides a stage discharge curve indicating that PMF is estimated to reach an elevation of 601 feet MSL. The walkdown report (Section 4.a) discusses the 603 feet MSL elevation in the context of an elevation to which the plant can mitigate flood effects, but does not identify 603 feet MSL as the design-basis flood elevation.

The comparison between the design basis and the reevaluated hazard is key for determining which hazards, if any, should be evaluated in the integrated assessment report.

Request: Provide clarification regarding the apparent discrepancy between the FHRR and the flooding walkdown report with respect to the design-basis PMF elevation.

**Response:**

The 200-year flood corresponding to an elevation of 589 feet was considered to be the Probable Maximum Flood in the original FSAR. Ensuing discussions between ComEd and the AEC during original approval – documented in FSAR Amendments 13, 16, 18 and 23, expanded the discussion on Probable Maximum Flood to encompass a flood up to elevation 603 feet based on lower probability scenarios.

The Safety Evaluation Report (August 25, 1971) that approved the Operating License for the plant reviews the data discussed above and includes discussion that the plant selected the 200-year flood as the Probable Maximum Flood but also provides additional discussion that "More recently for construction permit reviews, we have used the larger "Probable Maximum Flood" (PMF), as defined by the U.S. Corps of Engineers, as a basis for establishing the maximum flood level for which a facility should be designed. Using this criterion, the applicant's flood analysis predicted that the highest level that would be reached by a PMF would reach about 8 feet above plant site grade. The applicant has described emergency measures that can be taken under these circumstances to protect the plant against the effects of flooding, and to achieve and maintain a safe shutdown condition without resulting in any structural damage or release of radioactivity from the reactor system." A PMF of 8 feet above plant grade corresponds to an elevation of 603 feet. Therefore, the design basis PMF from the original SER for Quad Cities is 603 feet.

**RAI 4: Local Intense Precipitation – Supporting Analysis and Electronic Files**

Background: The information provided in the LIP evaluation report does not adequately describe modeling assumptions and key features of the modeling implementation such as the representation of topography and land cover, model input parameters, and model output accuracy. The NRC staff audit found that some of this information is described in Calculation Package LIP-QDC-001, Rev. 3, "Quad Cities Local Intense Precipitation Evaluation."

Request: Provide the following information:

1) The body of the supporting analysis (LIP-QDC-001, Rev 3, pages 1-19)
2) Appendix A (Figures) of the supporting analysis (LIP-QDC-001, Rev 3, pages 20-65)
3) Electronic versions of input and output files for the LIP analysis, including:

a. Digital Elevation Model (DEM) or other x-y-z data files used to produce the ground surface elevation map (Figure A-01 in Calculation Package LIP-QDC-001, Rev. 3).
b. An electronic version of the ground surface elevation map (Figure A-01 in Calculation Package LIP-QDC-001, Rev. 3)
c. An electronic version of the map showing land cover and Manning's n roughness coefficients (Figure A-02 in LIP-QDC-001, Rev. 3)
d. Electronic versions of all FLO-2D flood routing model input files (including model execution and numerical solution control files) used for surface flow modeling of the LIP probable maximum precipitation (PMP) event described in LIP-QDC-001, Rev. 3, and all FLO-2D output files listed in Appendix D of Calculation Package LIP-QDC-001, Rev. 3.

**Response:**

1) The body of the supporting analysis (LIP-QDC-001, Rev 3, pages 1-19) – See Enclosure 2, DVD #1 under file labeled "1 – LIP-QDC-001_Rev 3_Narrative.pdf"

2) Appendix A (Figures) of the supporting analysis (LIP-QDC-001, Rev 3, pages 20-65) – See Enclosure 2, DVD #1 under file labeled "2 – LIP-QDC-001_Rev 3_Appendix A.pdf"

3) Electronic versions of input and output files for the LIP analysis, including:

   a. Digital Elevation Model (DEM) or other x-y-z data files used to produce the ground surface elevation map (Figure A-01 in Calculation Package LIP-QDC-001, Rev. 3) – See Enclosure 2, DVD #1 under folder labeled "3 – LIP Analysis Input and Output Files", subfolder labeled "a – Digital Elevation Model".

   b. An electronic version of the ground surface elevation map (Figure A-01 in Calculation Package LIP-QDC-001, Rev. 3) – See Enclosure 2, DVD #1 under folder labeled "3 – LIP Analysis Input and Output Files", subfolder labeled "b – Ground Surface Elevation Map".

   c. An electronic version of the map showing land cover and Manning's n roughness coefficients (Figure A-02 in LIP-QDC-001, Rev. 3) – See Enclosure 2, DVD #1 under folder labeled "3 – LIP Analysis Input and Output Files", subfolder labeled "c – Land Cover and Manning's n Roughness Map".

   d. Electronic versions of all FLO-2D flood routing model input files (including model execution and numerical solution control files) used for surface flow modeling of the LIP probable maximum precipitation (PMP) event described in LIP-QDC-001, Rev. 3, and all FLO-2D output files listed in Appendix D of Calculation Package LIP-QDC-001, Rev. 3 – See Enclosure 2, DVD #1 under folder labeled "3 – LIP Analysis Input and Output Files", subfolder labeled "d – FLO-2D Input-Output Files".

**RAI 5: Local Intense Precipitation – Storm Analysis Basis for Design Storm**

Background: The LIP analysis relied on the storm analyses performed by National Weather Service in developing Hydrometeorology Reports (HMRs) 51 and 52. However, the licensee conducted a site-specific PMP study to support the PMF analysis in the FHRR. The site-specific PMP study included storms not considered in HMRs 51 and 52.

Request: Justify using one set of storms as a basis for the LIP estimates and a different set of storms as the basis for the PMF analysis.

**Response:**

The local intense precipitation event (LIP) was analyzed for the 1-hour/1-square-mile PMP, as defined in HMR-52. (The depth-area-duration data in HMR-51 does not include the 1-hour/1-square-mile PMP. The storm with the shortest duration and smallest area in HMR-51 is the 6-hour/10-square-mile PMP.) HMR-51 is frequently used for riverine watershed studies but is limited to a watershed size of approximately 20,000 square miles. The watershed size for the Mississippi River at Quad Cities (88,000 square miles), which is far larger than the size limitation in HMR-51, was the primary driver for performing a watershed-wide site-specific study.

Because of its short duration and small area, a site-specific study for the 1-hour/1-square-mile PMP would have involved an additional analysis of storms, different than those included in the watershed-wide study. Industry's experience, including Exelon's more recent experience in Illinois, is that site-specific studies for the 1-hour/1-square-mile PMP consistently results in lower rainfall values than in HMR-52, particularly in inland areas. In this submittal, Exelon chose to accept a more conservative value in HMR-52 for the 1-hour/1-square-mile PMP.

**RAI 6: Local Intense Precipitation – Design Storm Duration and Temporal Distribution**

Background: Although the basic approach for developing the design storm is outlined in HMR 52, several key details are left to the discretion of the analyst. Among these are the duration of the design storm (e.g., in relation to watershed characteristics such as the time of concentration) and the temporal distribution of the rainfall within the selected duration.

Request: Describe the rational, including any sensitivity analysis, that indicates whether the 1-hr PMP scenario used in the LIP analyses (Figure 3-2) bounds the effects of LIP in comparison with alternative duration PMP scenarios (e.g. 6-hr, 12-hr, 48-hr, or 72-hr PMP scenarios). The licensee is requested to evaluate the bounding LIP scenarios in terms of the severity of flood level as well as inundation duration. The licensee is also requested to describe the rationale for evaluating LIP using a temporal rainfall distribution in which the peak rainfall intensity occurs at the beginning of the PMP event and declines thereafter (e.g., in comparison with another temporal distribution, such as a centered distribution).

**Response:**

Per NUREG/CR-7046 Section 3.2 (Reference 1): *"Local intense precipitation is a measure of the extreme precipitation at a given location.* ***The duration of the event and the support area are needed to qualify an extreme precipitation event fully.*** *Generally, the amount of extreme precipitation decreases with increasing duration and increasing area. The PMP values for areas of the United States east of the 105th meridian are presented in HMRs 51 (Schreiner and Riedel 1978) and 52 (Hansen et al. 1982). The 1-hr, 2.56-km$^2$ (1-mi$^2$) PMP was derived using single-station observations of extreme precipitation, coupled with theoretical methods for moisture maximization, transposition, and envelopment. HMR 52 recommended that no increase in PMP values for areas smaller than 2.56 km$^2$ (1 mi$^2$) should be considered over the 1-hr, 2.56-km$^2$ (1-mi$^2$) PMP.* ***The local intense precipitation is, therefore, deemed equivalent to the 1-hr, 2.56-km$^2$ (1-mi$^2$) PMP at the location of the site."***

Since the 1-hr, 1-mi$^2$ LIP event would fully encompass the contributing drainage area of the Quad Cities Nuclear Station, the evaluation of a longer duration and larger storm event (6-hr, 10-mi$^2$) was not warranted. This approach is in accordance with the definition of the LIP event per NUREG/CR-7046, as described above. In addition, because of the rainfall intensity during the first hour of the storm event, the amount of precipitable water available for a longer duration storm event would be minimal compared to the first hour. Therefore, any increase in maximum flood levels due to a longer duration storm event is unlikely.

The 1-hr PMP event temporal distribution was developed in accordance with HMR 52 (Reference 2), which provides a set of multiplication factors for the 5-, 15-, and 30-minute time intervals relative to the 1-hr, 1-mi$^2$ PMP depths. While HMR 52 does not specifically state the time intervals be arranged in this particular order, with the typical west-east flow across North America, the type of storm set-up that would provide an LIP would likely be a mesoscale convective system (such as a squall line for example). Using the conceptual model of this type of system (Reference 3) the initial precipitation is associated with the mature cells and a zone of convergence and as such will be very intense. The storm motion and nature of the system would then see a decrease in the precipitation after the initial burst as the rear trailing stratiform region with the cold pool moves over the area. This type of meteorological system fits with the front loaded distribution.

References:

1. United States Nuclear Regulatory Commission (2011) NUREG/CR-7046, "Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America."
2. NOAA Hydrometeorological Report No. 52 (HMR-52) (1982), Application of Probable Maximum Precipitation Estimates – United States East of the 105th Meridian, U.S. Department of Commerce, National Oceanic and Atmospheric Administration, and U.S. Department of the Army Corps of Engineers.
3. Houze, Robert A., Jr. (2004), "Mesoscale Convective Systems." Review of Geophysics, 42, RG4003/2004. Paper number 2004RG000150.

**RAI 7: Local Intense Precipitation – Precipitation onto Buildings**

Background: The LIP evaluation report does not describe how precipitation onto building roofs was modeled. The NRC staff audit found that some of this information is described in LIP-QDC-001, Rev. 3.

Request: Provide a detailed description of how rainfall onto buildings is modeled in the LIP analysis, including a description of how water draining from roofs is routed and how this is implemented in the FLO-2D model. Provide justification for any assumptions regarding water storage by buildings.

**Response:**

To represent the buildings in the FLO-2D model, FLO-2D Area Reduction Factors (ARFs) and Width Reduction Factors (WRFs) functions were utilized. ARFs and WRFs are coefficients that modify the individual grid element surface area storage and flow width, respectively. ARFs can be used to reduce the flood volume storage on grid elements due to buildings and enhance the accuracy of the flood simulation. ARFs are specified as a percentage of the total grid element surface area (less than or equal to 100%). WRFs are specified as a percentage of the grid element side (less than or equal to 100%). In the FLO-2D model version used for the analysis, when an element's ARF is set from 0.95 to 1.0, any rainfall volume is assumed to go into the storm drain system and will not route through the model.

In the Quad Cities LIP FLO-2D model, the elements requiring ARF and WRF values were selected using a shapefile created in ArcGIS software representing the outline of the buildings. The ARF value for buildings was set to 0.94 to ensure that stormwater falling on top of roof buildings is accounted for in the model and not removed from the model domain. A WRF value of 1.0 was set for grids adjacent to the buildings in the four critical directions to prevent inflows from the surrounding elements and to ensure that the building elements do not provide storage for floodwaters from the surrounding elements. Water was allowed to pond on building roofs due to roof parapets, which assumes any roof drains are completely blocked. The justification for this assumption was based on the existence of roof parapets on the Turbine Building and the Reactor Building. The height of roof parapets on the Reactor Building is 17 inches on the east side and 29 inches on the west side (Reference 1), with the roof sloped towards the west. The height of roof parapets on the Turbine Building is 17 inches with additional storage provided towards the center of the roof where the height between the top of the parapet and the roof is 24 inches (Reference 2). The blockage of roof drains is consistent with Case 3 scenario.

References:

1. Exelon Drawing B-764, Quad Cities Nuclear Station, "Reactor Building Roof Plan."
2. Exelon Drawings B-716 and B-717, Quad Cities Nuclear Station, "Turbine Building Roof Plan."

**RAI 8: Local Intense Precipitation – Modeling Approach for Spray Canal**

Background: The LIP evaluation report indicates that the spray canal is included within the FLO-2D modeling domain. However, no discussion of the modeling approach used for the spray canal is provided. A Manning's n roughness coefficient for the canal water surface is provided instead of a Manning's n roughness coefficient based on the canal sides and bottom, which implies that the canal is not treated a flow element.

Request: Provide clarification regarding the modeling approach used for the spray canal.

**Response:**

The spray canal geometry is included in the FLO-2D model as part of the gridded digital elevation model (DEM) surface based on publically available LiDAR survey data (Reference 1). The LiDAR survey coverage of the spray canal reflects the side slopes and water surface elevation in the canal at the time of the LiDAR survey. The LiDAR survey did not penetrate the water surface in the spray canal to capture bathymetry and, therefore, the spray canal bottom elevation in the DEM is represented by the water surface elevation at the time of the LiDAR survey. This method of depicting the spray canal is conservative since it does not treat the spray canal as a flow element and reflects the NUREG/CR-7046 Case 3 scenario for the LIP analysis (all drainage canals blocked). The Manning's n roughness coefficient for water surface (0.02) was assigned to the elements representing the spray canal bottom since the elevation of the grid element reflects water surface. The Manning's n roughness coefficient for grass (0.32) was assigned to the grassed-lined canal side slopes. It should be noted that the inner and outer berms around the canal provide topographic relief and, therefore, the selection of the Manning's n roughness coefficient is not critical in the LIP evaluation. Runoff from adjacent drainage areas would flow away from the site and would not contribute to higher flood levels in the power block. In addition, with the culverts modeled as completely blocked, the spray canal would provide storage for runoff flowing directly into the canal without causing a backwater condition on the power block.

References:

1. Aero-Metric Photogrammetry and Geospatial Data Solutions (2010). Vertical Accuracy Report for State of Illinois Department of Transportation Rock Island County Illinois. Available at http://www.isgs.uiuc.edu/nsdihome/webdocs/ilhmp/county/rockisland.html (accessed on June 6, 2012).

**RAI 9: Local Intense Precipitation – Modeling of Concrete Security Barriers**

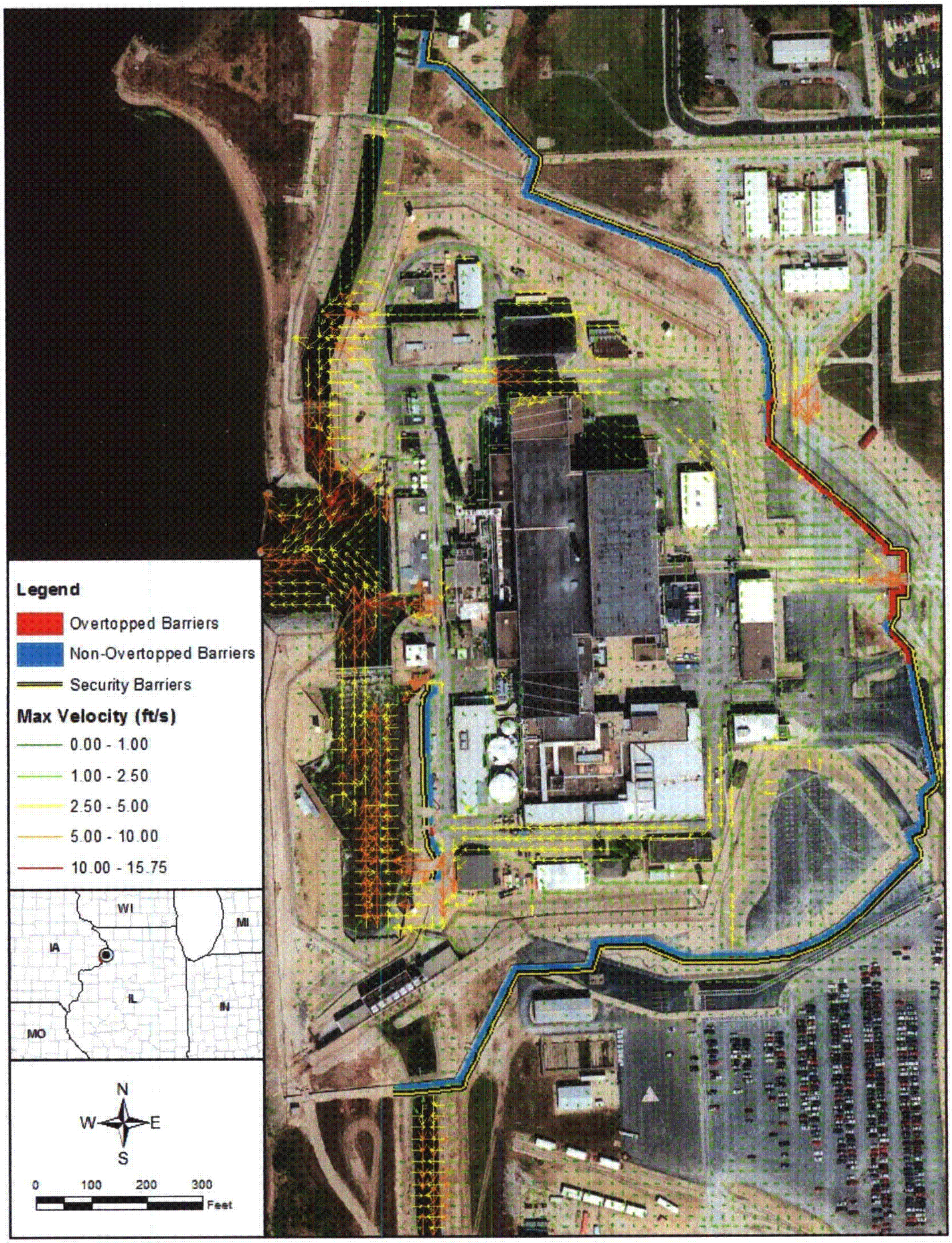
Background: The LIP evaluation report does not describe how flow over (or around) concrete security barriers is modeled. The NRC staff audit found that some of this information is described in LIP-QDC-001, Rev. 3. The staff is aware that FLO-2D treats flow over structures such as levees as flow over a broad-crested weir, with a fixed weir coefficient of 2.85. A weir coefficient of 2.85 is within the range found in several hydrology text books. However, weir coefficients provided in standard hydrology and hydraulics texts are for flow measurement weirs. The concrete security barriers are not flow measurement devices. In addition, the weir coefficient is a function of the weir breadth and the head upstream of the weir.

Request: Provide justification for modeling the flow over concrete security barriers using the FLO-2D levee function (i.e., as flow over a broad-crested weir with a fixed weir coefficient of 2.85), including a discussion of any physical characteristics which prompt such a selection and any sensitivity analysis that was performed.

**Response:**

The levee feature was used to represent the concrete jersey barriers and concrete blocks in the FLO-2D model to account for the security barrier's effects on local drainage patterns. The use of the levee feature allowed for modeling of the gaps between the security barriers to accurately represent potential flow paths around and over the structures. Weir flow will occur until the tailwater depth is 85% of the headwater depth, and at that point the model will calculate the exchange across the levee using the difference in water surface elevation. This modeling approach allows for better representation of the security barrier compared to changing the ground elevation of each individual grid element to reflect the top elevation of the security barrier. Furthermore, the breadth of the security barrier (concrete block width and jersey barrier configuration) is more representative of a broad crested weir and justifies the use of the broad crested weir coefficient for estimating overtopping flow in this analysis. In addition, only a limited segment of the security barriers is overtopped during the LIP event, as shown in Figure 1.

**Figure 1 – Overtopped Security Barriers during FLO-2D LIP Model Run**



**RAI 10: Consistent Vertical Datums**

Background: The submittals do not use a consistent vertical datum. The LIP evaluation report provides elevations with respect to the North American Vertical Datum of 1988 (NAVD88). Analyses for other flooding phenomena in the FHRR report elevations with respect to the legacy General Adjustment of 1912 (MSL 1912) datum. Furthermore, the submittals do not provide a conversion between the two datums. The NRC staff audit found that datum conversions are provided in Calculation Package QDC-0085-S-1991, "Calculation of Probable Maximum Flood (PMF) Water Surface Elevation: Evaluation of Riverine Hydraulics for the Upper Mississippi River at QCNGS."

Request: Provide a consistent set of vertical datum for the submittals or provide a conversion between the various datums used. In particular, describe what relationships were used to convert between modern datums such as NAVD88 and legacy datums such as MSL 1912.

**Response:**

The LIP evaluation was performed utilizing NAVD88 because current surrounding plant topographical information was available in NAVD88 at the time of the evaluation. The flooding depths as provided in the LIP evaluation are the water surface depths corresponding to topographical information in NAVD88.

Subsequent to the completion of the LIP evaluation, an elevation survey was performed to correlate NAVD88 to the plant grade elevation. The elevation survey was performed by a Professional Land Surveyor licensed in the State of Illinois. Door sill elevations were captured and the elevations ranged from 594.51 feet to 594.61 feet (NAVD88).

Therefore, plant grade elevation of 595.0 feet, corresponds to a plant grade elevation of 594.55 (NAVD88). Thus, an equation to convert the datum shift for NAVD88 at Quad Cities Station is concluded to be:

Plant Grade – NAVD88 (feet) = 0.45 feet

QDC-0085-S-1991, Rev. 0, Attachment 2E (Datum Conversion between MSL 1912 and NAVD88 Equation Sheet and Results) indicates the datum shift (MSL 1912 – NAVD88) to be 0.70 feet at the River Mile 506.9 (corresponding to Quad Cities Station). This indicates that the Plant Grade is 0.25 feet above MSL 1912 (0.70 feet – 0.45 feet). Thus an equation to convert the datum shift for MSL 1912 at Quad Cities Station is concluded to be:

Plant Grade – MSL 1912 (feet) = 0.25 feet

**RAI 11: Probable Maximum Precipitation Analysis**

Background: The discussion of the PMP analysis in FHRR Section 3.1 does not adequately describe the overall logic, key assumptions, methods, inputs, and results. The NRC staff audit found that this information is documented, in part, in Calculation Package QDC-0085-S-1989,

"Probable Maximum Precipitation (PMP) for the Upper Mississippi River Watershed Contributory to QCNGS."

Request: Provide the following portions of QDC-0085-S-1989:

- Main text (Pages 1-36)
- *Attachment 1, Figures*
- Attachment 2, Section 10 (Storm Dimensions)
- Attachment 2, Section 13 (Recommendations for Applications)
- Attachment 2, Appendix E (SPAS System Description)

Also provide electronic editable files for the following portions of QDC-0085-S-1989:

- Depth-Area Duration Tables (Attachment 7)
- Percentage of 6-hr PMP Increment tables (Attachment 8)
- Average 6-hr Incremental PMP Spreadsheets (Attachment 9)
- All Season and cool season hyetographs (Attachment 10)
- 100-year snowpack calculation spreadsheet (Attachment 11)
- Meteorological Time Series Data for Snowmelt (Attachment 13)
- Snowmelt Spreadsheet (Attachment 14)

**Response:**

The requested excerpts from Calculation QDC-0085-S-1989 and electronic editable files are included in Enclosure 3 DVD #2A under folder labeled "RAI 11". Note that NDCD Stations 14923 and 479304 conservatively reported the March statistics in the PMP calculation. The March statistics for these two gages are included in Enclosure 3 DVD #2A.

**RAI 12: Site-Specific PMP Estimates**

Background: FHRR Section 3.1 states that a site-specific PMP analysis was performed to calculate PMP values specific to the 88,000 square-mile ($mi^2$) contributory watershed of the Mississippi River upstream of the QCNPS site because HMR 51 does not provide PMP estimates for areas in excess of 20,000 $mi^2$. The FHRR further states that the site-specific PMP analysis used techniques and databases that differ from those used in HMR 51. However, the FHRR does not provide adequate detail to evaluate the differences in techniques and data used. The NRC staff audit found that some of this information is documented in QDC-0085-S-1989.

Request: Provide the following information:

- A detailed description of the techniques and databases used, including storm selection. Describe any difference between techniques and databases used in the current analysis and those used in HMR 51.
- A detailed comparison between PMP results in HMR 51 and the FHRR analysis for those areas and duration common to both analyses. Include an explanation for differences that may exist.

- A detailed description of the alternate dewpoint climatology used, including data sources, methods, and resulting maps or databases. An existing report that includes this information would suffice.
- Electronic versions of the storm analysis spreadsheets developed for the site-specific PMP study (QDC-0085-S-1989, Attachment 2, Appendix F)
- Electronic versions of the spreadsheets used to perform the depth-area and depth-duration envelopments
- Electronic versions of the final all-season and cool-season PMP maps (QDC-0085-S-1989, Attachment 2, Appendices A and B)

**Response:**

- *A detailed description of the techniques and databases used, including storm selection. Describe any difference between techniques and databases used in the current analysis and those used in HMR 51.*

The storm selection process employed during the analysis of the Quad Cities Nuclear Power Station (QCNPS) PMP work included analysis of several databases and previous storm lists derived during previous and ongoing PMP work in the region considered transpositionable to any point within the overall watershed. Applied Weather Associates (AWA) has completed numerous PMP studies in the region where storms were considered transpositionable to the QCNPS watershed. This region covers most of the Midwest below 3,000 feet east through the initial upslope region of the Appalachians and within about 6° latitudinal extent. Storms comprising these storm lists were queried from National Climatic Data Center (NCDC) hourly and daily datasets, US Army Corps of Engineers (USACE) storm studies, National Weather Service Hydrometeorological Reports (HMRs), United State Geological Survey (USGS) flood reports, journal articles, various government and private mesonets, weather books, and other sources. All data have been quality controlled to verify the accuracy of the given rainfall report(s).

Storms which comprised the initial list of storms had to have been at least equivalent to or greater than the 100-year precipitation frequency value for the given duration and given location and/or have resulted in major flooding at given location. This resulted in hundreds of potential rainfall events. This list then needed to be analyzed further to produce a manageable list of storms which would then be fully analyzed to derive the PMP values. This final list is known as the short storm list.

The final short storm list used to determine the PMP values for QCNPS basin was derived using the results of previous PMP studies in regions similar to this basin. These include the EPRI Michigan/Wisconsin Regional PMP study (accepted by FERC), the Nebraska Statewide PMP study (accepted by FERC and Nebraska dam safety), the Ohio Statewide PMP study (accepted by FERC and Ohio dam safety), Tarrant Regional Water District PMP (accepted by Texas dam safety), and the Wyoming Statewide PMP study (in progress).

During this process, the final short storm lists used in each of these studies was combined and evaluated. The first set of parameters used to delineate the storms was whether they were transpositionable to any grid point used to derive the PMP values for the QCNPS basin.

Factors such as elevation differences of more than +/- 1,000 feet and/or distances of more than +/- 6° latitude were considered. Next, the storm type was evaluated. Storm types which would not result in a PMP/PMF scenario for the large QCNPS watershed were not considered. This included storms which were individual thunderstorms, Mesoscale Convective Systems (MCS) and storms which were directly associated with remnant tropical systems.

This storm search and storm selection process and methods were generally the same as those used and described in HMR 51. However, no notes or working papers are available from HMR 51, so explicit comparisons are not possible. However, all storms used in HMR 51 which occurred within the region considered transpositionable were included. Differences include updating the storm database to include all storms through 2013 (adding more than 40 years of storm record to the previous design basis). In addition, more databases were queried and utilized because several did not exist when the original analysis was completed.

Finally, the transpositioning of storms to each of the grid points was an area where potential differences between this study and HMR 51 exist. AWA has the original copy of all transposition limits maps produced by the NWS. In addition, maps of transposition limits for specific storms are provided in various HMRs (e.g. HMR 52 Figure 26) and included for several storms in HMR 53 Table 2.1. AWA utilized this information along with our updated analysis to determine explicit transposition limits for each storm considering the QCNPS watershed characteristics. AWA's analysis included the guidance provided in HMR 51 Section 2.4.2 and updated understanding of storm dynamics, available moisture sources, variations in dew point climatologies by season, interactions with topography, and differences by storm type.

These analyses and engineering judgments were applied to each storm considering each grid point, with specific meteorological and topographical characteristics considered. This general analysis process is similar to that described in HMR 51. However, the major difference is AWA does not allow implicit transpositioning to occur as was done in HMR 51. This occurs in HMR 51 during the smoothing and regionalization process employed to produce consistent PMP isolines across the entire region covered by HMR 51. In order to envelope all data across the entire region covered by HMR 51 and avoid bulls-eyes or inconsistent PMP isolines, HMR 51 had to allow storms to influence PMP values far beyond their intended explicit transposition limits. It was not within HMR 51 scope to consider explicit characteristics of individual basins, but instead they were required to provide a generalized PMP estimate across the entire region. This updated site-specific analysis was not constrained by this consideration and was able to explicitly consider the unique meteorological and topographic characteristics of the basin and apply updated understanding to the study not available to HMR 51.

A specific example of the implicit transpositioning that effects HMR 51 PMP values across the entire domain is demonstrated by the maximized values of the Smethport, PA July 1942 storm and how those values were enveloped. The NWS explicitly states that this storm should only be transpositioned east to the crest of the Appalachians, south to 35°N, north to 43°N and west along the first upslopes of the Appalachians (Figure 1). However, the PMP isolines across the entire region covered by HMR 51 envelope the data from this storm, thereby allowing a storm which is not transpositionable to much of the domain to control PMP values inappropriately. This is evidenced by comparing all storms used in HMR 51 to the PMP values over the regions where they are transpositionable and noting that no storm data supports the values provided except the maximized Smethport storm.

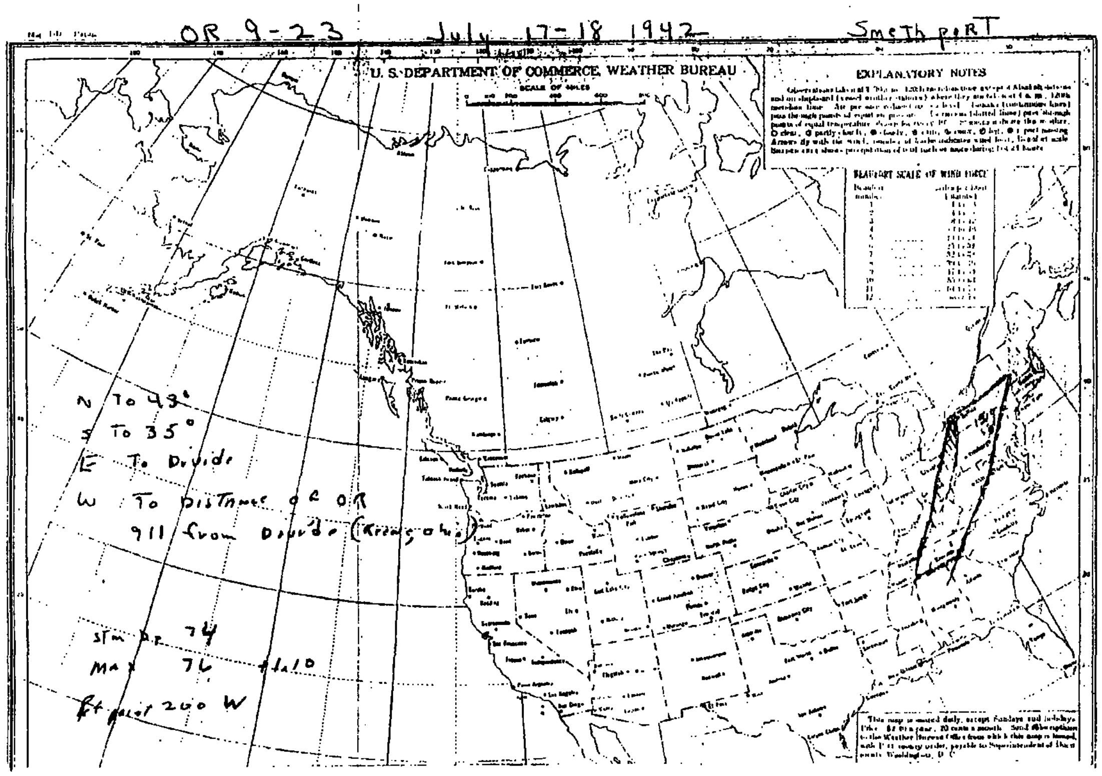



**Figure 1 – Smethport, PA July 1942 NWS transposition limits with notes detailing constraints**

**Response:**

- *A detailed comparison between PMP results in HMR 51 and the FHRR analysis for those areas and duration common to both analyses. Include an explanation for differences that may exist.*

A direct comparison to HMR 51 PMP values for duration from 24- through 72-hours and area sizes from 1000- through 20,000-square miles is provided at each grid point analyzed during the study in Table 12.1-12.3. In the tables, values greater than zero represent reductions from the HMR 51 values and values less than zero represent increases from the HMR 51 values. Figure 12.1 displays the grid point locations used in this study and referenced in the Tables 12.1-12.3. Comparisons are only directly valid at these area sizes and durations because the storms used in this study to compute PMP values were of the storm type relevant for producing PMF over large area sizes and long durations. Therefore, short duration, high intensity storms (MCCs and thunderstorms) were not included in the analysis. This is because those storms types are unique and would not co-occur with large scale, long duration storms. Therefore, there should not be a comingling of storm types that would result in a PMP design storm which is not physically possible, as this would violate the HMR definition of PMP. Comparisons for cool-season PMP values are not possible because no explicit cool-season PMP values exist in HMR 51. HMR 52 does provide seasonality adjustments which can be applied to HMR 51 all-season PMP values. However, these data do not result in true cool-season PMP, as they are a result of

ratio applied to all-season type storm events which are different than cool-season rain-on-snow events.

Quad Cities Nuclear Generating Station
Grid Point Locations

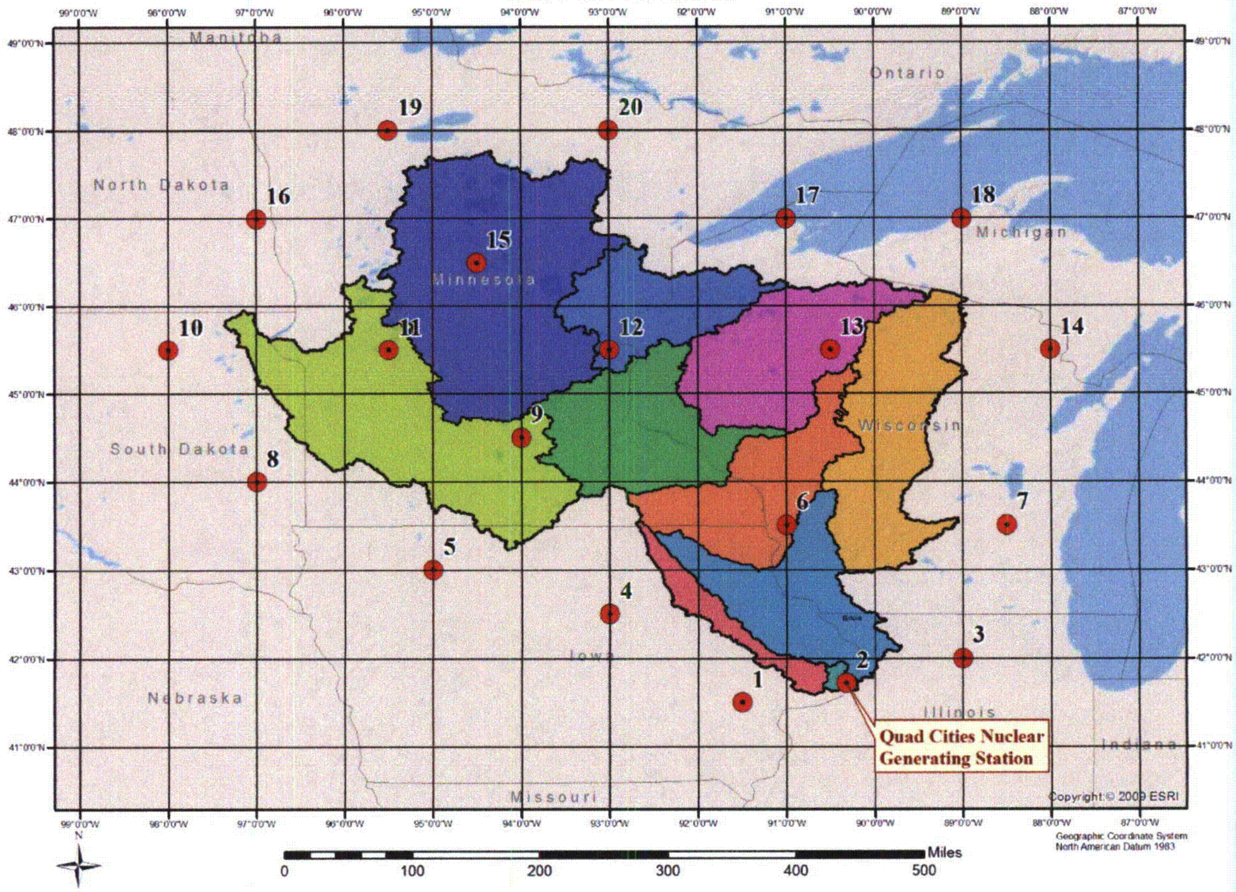



**Figure 12.1 – Grid points used in the study with QCNPS watershed and sub-basins delineated for reference**

**Table 12.1 –Percent difference between the all-season PMP values at each grid point and the basin centroid at the 24-hour duration vs HMR 51 PMP values. Positive values represent reductions from HMR 51.**

| GRIDPOINT | LAT | LON | 1,000mi² | 5,000mi² | 10,000mi² | 20,000mi² |
|---|---|---|---|---|---|---|
| 1 | 41.50 | -91.50 | 13% | 11% | 10% | 8% |
| 2 | 41.72 | -90.32 | 12% | 10% | 7% | 8% |
| 3 | 42.00 | -89.00 | 11% | 10% | 7% | 6% |
| 4 | 42.50 | -93.00 | 14% | 10% | 10% | 9% |
| 5 | 43.00 | -95.00 | 15% | 11% | 10% | 10% |
| 6 | 43.50 | -91.00 | 11% | 8% | 8% | 7% |
| 7 | 43.50 | -88.50 | 7% | 6% | 6% | 4% |
| 8 | 44.00 | -97.00 | 13% | 11% | 9% | 14% |
| 9 | 44.50 | -94.00 | 8% | 5% | 3% | 4% |
| 10 | 45.50 | -98.00 | 10% | 6% | 4% | 8% |
| 11 | 45.50 | -95.50 | 10% | 7% | 6% | 6% |
| 12 | 45.50 | -93.00 | 7% | 4% | 4% | 4% |
| 13 | 45.50 | -90.50 | 8% | 7% | 7% | 5% |
| 14 | 45.50 | -88.00 | 2% | 2% | 3% | 0% |
| 15 | 46.50 | -94.50 | 9% | 7% | 6% | 9% |
| 16 | 47.00 | -97.00 | 8% | 4% | 3% | 8% |
| 17 | 47.00 | -91.00 | 1% | 3% | 3% | -1% |
| 18 | 47.00 | -89.00 | 0% | 2% | 2% | -1% |
| 19 | 48.00 | -95.50 | 10% | 7% | 7% | 11% |
| 20 | 48.00 | -93.00 | 8% | 7% | 7% | 5% |
| Basin Centroid | 44.89 | -92.69 | 10% | 8% | 7% | 5% |

**Table 12.2 –Percent difference between the all-season PMP values at each grid point and the basin centroid at the 48-hour duration vs HMR 51 PMP values. Positive values represent reductions from HMR 51.**

| GRIDPOINT | LAT | LON | 1,000mi$^2$ | 5,000mi$^2$ | 10,000mi$^2$ | 20,000mi$^2$ |
|---|---|---|---|---|---|---|
| 1 | 41.50 | -91.50 | 11% | 3% | -2% | 1% |
| 2 | 41.72 | -90.32 | 10% | 0% | -5% | -3% |
| 3 | 42.00 | -89.00 | 9% | -1% | -5% | -3% |
| 4 | 42.50 | -93.00 | 13% | 5% | -1% | 1% |
| 5 | 43.00 | -95.00 | 14% | 8% | 3% | 2% |
| 6 | 43.50 | -91.00 | 8% | -1% | -4% | -2% |
| 7 | 43.50 | -88.50 | 5% | -3% | -7% | -5% |
| 8 | 44.00 | -97.00 | 15% | 10% | 12% | 10% |
| 9 | 44.50 | -94.00 | 7% | 0% | -6% | -5% |
| 10 | 45.50 | -98.00 | 13% | 10% | 8% | 7% |
| 11 | 45.50 | -95.50 | 11% | 5% | -1% | 2% |
| 12 | 45.50 | -93.00 | 6% | 0% | -6% | -5% |
| 13 | 45.50 | -90.50 | 7% | 0% | -5% | -4% |
| 14 | 45.50 | -88.00 | 3% | -5% | -11% | -11% |
| 15 | 46.50 | -94.50 | 10% | 5% | 0% | -1% |
| 16 | 47.00 | -97.00 | 11% | 8% | 7% | 6% |
| 17 | 47.00 | -91.00 | 3% | -4% | -10% | -11% |
| 18 | 47.00 | -89.00 | 1% | -5% | -11% | -13% |
| 19 | 48.00 | -95.50 | 11% | 10% | 10% | 10% |
| 20 | 48.00 | -93.00 | 5% | 0% | -4% | -4% |
| Basin Centroid | 44.89 | -92.69 | 8% | 1% | -4% | -2% |

**Table 12.3 –Percent difference between the all-season PMP values at each grid point and the basin centroid at the 72-hour duration vs HMR 51 PMP values. Positive values represent reductions from HMR 51.**

| GRIDPOINT | LAT | LON | 1,000mi$^2$ | 5,000mi$^2$ | 10,000mi$^2$ | 20,000mi$^2$ |
|---|---|---|---|---|---|---|
| 1 | 41.50 | -91.50 | 6% | 0% | 0% | 3% |
| 2 | 41.72 | -90.32 | 5% | 0% | 1% | 4% |
| 3 | 42.00 | -89.00 | 5% | 3% | 3% | 6% |
| 4 | 42.50 | -93.00 | 7% | 1% | 0% | 2% |
| 5 | 43.00 | -95.00 | 7% | 0% | -1% | 2% |
| 6 | 43.50 | -91.00 | 5% | 1% | 0% | 3% |
| 7 | 43.50 | -88.50 | 3% | 3% | 4% | 6% |
| 8 | 44.00 | -97.00 | 7% | 0% | 0% | 0% |
| 9 | 44.50 | -94.00 | 1% | -4% | -7% | -4% |
| 10 | 45.50 | -98.00 | 16% | 12% | 10% | 9% |
| 11 | 45.50 | -95.50 | 1% | -3% | -4% | -2% |
| 12 | 45.50 | -93.00 | -2% | -6% | -7% | -4% |
| 13 | 45.50 | -90.50 | 4% | 4% | 5% | 8% |
| 14 | 45.50 | -88.00 | 7% | 6% | 5% | 4% |
| 15 | 46.50 | -94.50 | 11% | 11% | 11% | 9% |
| 16 | 47.00 | -97.00 | 14% | 13% | 12% | 9% |
| 17 | 47.00 | -91.00 | 5% | 4% | 5% | 4% |
| 18 | 47.00 | -89.00 | 4% | 4% | 4% | 4% |
| 19 | 48.00 | -95.50 | 15% | 17% | 15% | 14% |
| 20 | 48.00 | -93.00 | 10% | 10% | 9% | 9% |
| Basin Centroid | 44.89 | -92.69 | 2% | -2% | -2% | -1% |

Because no working papers or notes exists for HMR 51, explicit comparisons are not possible for many of the components. However, comparisons can be made for some of the data and general comparisons and reasons for differences are discussed. The following areas were treated differently and/or updated in this study versus the HMRs:

1. HMR 51 provides generalized and smoothed PMP values over a large geographic domain covering the United States east of the 105$^{th}$ meridian. Specific characteristics unique to individual basins, such as QCNPS, were not addressed. This study considered characteristics specific to the basin, and produced PMP values explicitly considering the topography of the basin and the meteorology of the PMP storm type which would results in the PMF for the basin.

2. Each storm's inflow vector was re-evaluated and combined with an updated set of dew point climatology data and when necessary, updated storm representative dew point values were used for the in-place maximization and computation of the total adjustment factors. The HYSPLIT trajectory model was used to evaluate and verify moisture inflow vectors for storms on the short storm list. Trajectory models were not available in previous HMR studies. The use of HYSPLIT allowed for a high degree of confidence when evaluating moisture inflow vectors and storm representative dew points.

3. Several new storms have been analyzed and included in this site-specific PMP study that were not included in HMR 51. This provided a higher level of confidence in the final site-specific PMP values. Further, this allowed for a refined set of values that better represent the PMP values for both the all-season and cool-season PMP scenarios, as the data set used to derive PMP has been expanded to include a larger set of more recent storms.

4. The site-specific PMP study provided adjustments for storm elevation to the nearest 100 feet of elevation, whereas HMR 51 made no explicit adjustment for elevation for PMP value over the basin. This adjustment depends on the elevation of the historic storm's maximum rainfall location and therefore varies from storm to storm. Further, the average basin elevation for each grid point was evaluated in this study using GIS, providing a much more accurate representation and calculation to account for loss of available moisture up to that elevation.

5. SPAS was used in conjunction with NEXRAD data (when available) to evaluate the spatial and temporal distribution of rainfall. Use of NEXRAD data generally produced higher point rainfall amounts than were observed using only rain gauge observations and provides objective spatial distributions of storm rainfall for locations among rain gauges. SPAS results provided storm depth area durations (DADs), total storm precipitation patterns, and mass curves for the newly analyzed storms. Using these technologies, significant improvements of the storm rainfall analyses were achieved.

6. Previously analyzed storm events that occurred prior to 1948 that used 12-hour persisting dew points were adjusted using storm representative dew point adjustments of 2°F for synoptic type storm events and 7°F for MCS type storm events. This was done to adjust for using average dew point values for varying durations vs. 12-hour persisting dew point values. Recent evaluations of 12-hour persisting storm representative dew points show those used in HMR 51 underestimated the storm representative values. An updated set of maximum dew point climatology maps were produced. These maps have higher maximum dew point values than those used in HMR studies and therefore compensate for the higher storm representative dew points.

7. Interpolation of PMP values at each duration between each of the 20 grid point displayed in Figure 1 was completed using GIS and manual engineering judgment. This resulted in a consistent and meteorologically reasonable spatial and temporal distribution of PMP values across the entire domain analyzed. HMR 51 employed a similar process where PMP isolines were contoured across a much larger domain (which also encompassed the QCNPS watershed) using engineering judgment. Computer technologies, such a GIS interpolation, were not available in HMR 51. Therefore, more refined analysis in this study specific to the site, increased understanding of meteorology and topography in the region, and use of computer interpolation technologies resulted in PMP distributions that are specific to the QCNGS watershed that were not available in HMR 51.

**Response:**

- *A detailed description of the alternate dew point climatology used, including data sources, methods, and resulting maps or databases. An existing report that includes this information would suffice.*

The dew point climatology used for site-specific PMP analysis is provided in detail in Attachment 2 of Calculation QDC-0085-S-1989. Appropriate excerpts from Calculation QDC-0085-S-1989 are included in Enclosure 3 DVD #2A under folder labeled "RAI 12".

**Response:**

- *Electronic versions of the storm analysis spreadsheets developed for the site-specific PMP study (QDC-0085-S-1989, Attachment 2, Appendix F)*

The requested electronic versions of the storm analysis spreadsheets for the site-specific PMP are included in Enclosure 3 DVD #2A under folder labeled "RAI 12".

**Response:**

- *Electronic versions of the spreadsheets used to perform the depth-area and depth-duration envelopments*

The requested electronic versions of the spreadsheets used to perform the depth-area and depth-duration envelopments are included in Enclosure 3 DVD #2A under folder labeled "RAI 12".

**Response:**

- *Electronic versions of the final all-season and cool-season PMP maps (QDC-0085-S-1989, Attachment 2, Appendices A and B)*

The requested electronic versions of the final all-season and cool-season PMP maps (QDC-0085-S-1989, Attachment 2, Appendices A and B) are included in Enclosure 3 DVD #2A under folder labeled "RAI 12".

**RAI 13: Probable Maximum Flood Analysis**

**Background:** The discussion of the PMF analysis in FHRR Section 3.1 does not adequately describe the overall logic, key assumptions, methods, inputs, and results. The NRC staff audit found that this information is documented, in part, in Calculation Package QDC-0085-S-1990, "Probable Maximum Flood (PMF) for the Upper Mississippi River Watershed Contributory to QCNGS."

**Request:** Provide the following portions of QDC-0085-S-1990:

- Main text (Pages 1-73)
- Attachment 1, Figures
- Attachment 6, Nonlinearity Adjustment
- Attachment 8, Muskingum-K Estimates

Also provide electronic editable files for the following:

- NRCS Soil Data (QDC-0085-S-1990, Attachment 4)
- USGS Monthly Flow Data (QDC-0085-S-1990, Attachment 5)
- Input and Output files for the HEC-HMS hydrologic model calibration and verification runs
- Input and Output files for the HEC-HMS hydrologic all-season PMF and cool-season PMF simulations

**Response:**

The requested excerpts from Calculation QDC-0085-S-1990 and electronic editable files are included in Enclosure 3 DVD #2A under folder labeled "RAI 13". The computer model names corresponding to the critical scenarios are specified in Table 13.1.

**Table 13.1 – Computer Models Corresponding to Probable Maximum Flood – Hydrology Analysis**

| Computer Software | Computer Model Name |
|---|---|
| **1. Calibration** | |
| HEC-HMS | RAI Response\RAI 13\HEC-HMS\Calibration\Upper_Mississippi_Aug95_Calibration\Upper_Mississippi_Aug95_Cal |
| HEC-HMS | RAI Response\RAI 13\HEC-HMS\Calibration\Upper_Mississippi_July2010_Calibration\Upper_Mississippi_July10_Cal |
| HEC-HMS | RAI Response\RAI 13\HEC-HMS\Calibration\Upper_Mississippi_June04_Calibration\Upper_Mississippi_Jun04_Cal |
| HEC-HMS | RAI Response\RAI 13\HEC-HMS\Calibration\Upper_Mississippi_June08_Calibration\Upper_Mississippi_June08_Cal |
| HEC-HMS | RAI Response\RAI 13\HEC-HMS\Calibration\Upper_Mississippi_June2002_Calibration\Upper_Mississippi_Jun02_Cal |
| HEC-HMS | RAI Response\RAI 13\HEC-HMS\Calibration\Upper_Mississippi_May99_Calibration |
| HEC-HMS | RAI Response\RAI 13\HEC-HMS\Calibration\Upper_Mississippi_May2002_Calibration\Upper_Mississippi_May2002_Cal |
| HEC-HMS | RAI Response\RAI 13\HEC-HMS\Calibration\Upper_Mississippi_Oct02_Calibration\Upper_Mississippi_Oct02_Cali |
| HEC-HMS | RAI Response\RAI 13\HEC-HMS\Calibration\Upper_Mississippi_Sept94_Calibration\Upper_Mississippi_Sept94_Cal |
| **2. Verification** | |
| HEC-HMS | RAI Response\RAI 13\HEC-HMS\Verification\Upper_Mississippi_Aug09_Verification\Upper_Mississippi_Aug09_Ve |
| HEC-HMS | RAI Response\RAI 13\HEC-HMS\Verification\Upper_Mississippi_July97_Verification\Upper_Mississippi_July97_Ve |
| HEC-HMS | RAI Response\RAI 13\HEC-HMS\Verification\Upper_Mississippi_July98Verification\Upper_Mississippi_July98_Ver |
| HEC-HMS | RAI Response\RAI 13\HEC-HMS\Verification\Upper_Mississippi_June00_Verification\Upper_Mississippi_June00_Ver |
| HEC-HMS | RAI Response\RAI 13\HEC-HMS\Verification\Upper_Mississippi_May03_Verification\Upper_Mississippi_May03_Ver |
| HEC-HMS | RAI Response\RAI 13\HEC-HMS\Verification\Upper_Mississippi_Oct05_Verification\Upper_Mississippi_Oct05_Ver |
| HEC-HMS | RAI Response\RAI 13\HEC-HMS\Verification\Upper_Mississippi_Oct07_Verification\Upper_Mississippi_Oct07_Ver |
| HEC-HMS | RAI Response\RAI 13\HEC-HMS\Verification\Upper_Mississippi_Oct95_Verification\Upper_Mississippi_Oct95_Ver |

**Table 13.1 – Computer Models Corresponding to Probable Maximum Flood – Hydrology Analysis (Continued)**

| Computer Software | Computer Model Name |
|---|---|
| **3. PMF** | |
| HEC-HMS | RAI Response\RAI 13\HEC-HMS\PMF\All_Season_at_Clinton |
| HEC-HMS | RAI Response\RAI 13\HEC-HMS\PMF\All_Season_PMF_at_Anoka |
| HEC-HMS | RAI Response\RAI 13\HEC-HMS\PMF\All_Season_PMF_at_McGregor |
| HEC-HMS | RAI Response\RAI 13\HEC-HMS\PMF\All_Season_PMF_at_Watershed |
| HEC-HMS | RAI Response\RAI 13\HEC-HMS\PMF\All_Season_PMF_at_Watershed_Alt2 |
| HEC-HMS | RAI Response\RAI 13\HEC-HMS\PMF\All_Season_PMF_at_Winona |
| HEC-HMS | RAI Response\RAI 13\HEC-HMS\PMF\All_Season_PMF_Moving |
| HEC-HMS | RAI Response\RAI 13\HEC-HMS\PMF\Cool_Season_PMF_at_Anoka |
| HEC-HMS | RAI Response\RAI 13\HEC-HMS\PMF\Cool_Season_PMF_at_Clinton |
| HEC-HMS | RAI Response\RAI 13\HEC-HMS\PMF\Cool_Season_PMF_at_McGregor |
| HEC-HMS | RAI Response\RAI 13\HEC-HMS\PMF\Cool_Season_PMF_at_Watershed |
| HEC-HMS | RAI Response\RAI 13\HEC-HMS\PMF\Cool_Season_PMF_at_Winona |
| HEC-HMS | RAI Response\RAI 13\HEC-HMS\PMF\Cool_Season_PMF_Moving |
| HEC-HMS | RAI Response\RAI 13\HEC-HMS\PMF\Nonlinear_UH |

**RAI 14: Flooding on Streams and Rivers – Cool-Season Baseflow**

Background: FHRR Section 3.1 states that streamflow data for March was used to estimate the cool-season mean monthly baseflow. Stream data for April was not used. The stated rational for omitting the April data was to avoid double-accounting for April snow melt.

Request: Provide justification for the inclusion of baseflow values for March instead of April, with consideration of the likelihood that streamflows for both months result from a combination of snowmelt and rainfall.

**Response:**

The March baseflow was conservatively used to reflect the river condition prior to the onset of the combined cool season probable maximum precipitation (PMP)/April snowmelt event. Based on review of historical streamflow data, the April baseflow is significantly larger than winter (December through March) baseflow, likely indicating the additional influence of snowmelt. The climate conditions necessary to accumulate and preserve a 100-year snowpack for an April PMP across the entirety of the Quad Cities Nuclear Power Station (QCNPS) watershed would not be the conditions necessary to simultaneously melt the same snowpack prior to the PMP that lead to a typically high April baseflow. These conditions would likely involve an abundance of snowfall combined with the necessary cold to preserve/avoid losing snowpack to snow melt in months prior to April. The objective of the cool season probable maximum flood (PMF) analysis was to preserve a ripe 100-year snowpack that would melt rapidly due to the cool season PMP (i.e., rain-on-snow). April baseflow was therefore excluded specifically so as to not double count snowmelt.

Consideration was given to exclude both March and April to avoid double-counting snowmelt effects. However, excluding both March and April flows from the calculation for the cool season baseflow would result in a reduction of approximately 70 percent, compared to a baseflow calculated to include both March and April. Therefore, the baseflow from March was allowed to remain as a conservative approach.

**RAI 15: Probable Maximum Flood Water Surface Calculations**

Background: The discussion of the PMF water surface calculation in FHRR Section 3.1 does not adequately describe the overall logic, key assumptions, methods, inputs, and results. The NRC staff audit found that this information is documented, in part, in Calculation Package QDC-0085-S-1991, "Calculation of Probable Maximum Flood (PMF) Water Surface Elevation: Evaluation of Riverine Hydraulics for the Upper Mississippi River at QCNGS."

Request: Provide the following portions of QDC-0085-S-1991:

- Main text (Pages 1-32)
- Attachment 1, Figures
- Attachment 2, Data Conversions
- Attachment 5, Bridge Plans

Also provide electronic editable files for the following:

- Bridge Spreadsheets (QDC-0085-S-1991, Attachment 6)
- Stream Flow Data (QDC-0085-S-1991, Attachment 7)
- USACE Observed Historical Profiles (QDC-0085-S-1991, Attachment 8)
- HEC-RAS hydraulic model input and output files for the calibration floods (QDC-0085-S-1991, Attachment 9)
- HEC-RAS hydraulic model input and output files used for water level simulation resulting from the PMF event reported in FHRR Section 3.1, including those in QDC-0085-S-1991, Attachments 10-14

**Response:**

The requested excerpts from Calculation QDC-0085-S-1991 and electronic editable files, listed below, are included in Enclosure 3 DVD #2A.

- Main text (Pages 1-32)
- Attachment 1, Figures
- Attachment 2, Data Conversions
- Attachment 5, Bridge Plans
- Bridge Spreadsheets (QDC-0085-S-1991, Attachment 6)
- Stream Flow Data (QDC-0085-S-1991, Attachment 7)
- USACE Observed Historical Profiles (QDC-0085-S-1991, Attachment 8)
- HEC-RAS hydraulic model calibration profiles (QDC-0085-S-1991, Attachment 9)

The requested HEC-RAS input-output files, listed below, are split on 5 DVDs #2B through #2F due to file size. The computer model names corresponding to the critical scenarios are specified in Table 15.1. In order for HEC-RAS Calibration model to run, all files from folders labeled "HEC-RAS- Calibration" from DVDs #2B through #2F need to be copied in the same folder.

- HEC-RAS hydraulic model input and output files for the calibration floods (QDC-0085-S-1991, Attachment 9)

- HEC-RAS hydraulic model input and output files used for water level simulation resulting from the PMF event reported in FHRR Section 3.1, including those in QDC-0085-S-1991, Attachments 10-14

Additionally, the USACE Observed Historical Profiles (QDC-0085-S-1991, Attachment 8) are graphics which are obtained through the USACE River Gages website (http://rivergages.mvr.usace.army.mil/WaterControl/new/layout.cfm) and are not available as electronic editable files. Therefore, Attachment 8 is included in Enclosure 3 DVD #2A in pdf format under folder labeled "RAI 15".

**Table 15.1 – Computer Models Corresponding to Probable Maximum Flood – Hydrology Analysis**

| Computer Software | Computer Model Name |
|---|---|
| | **Calibration - 1965 Flood** |
| HEC-RAS | HEC-RAS - Calibration |
| *Project* | Quad Cities-Mississippi River |
| *Plan* | 1965 Flood Calibration (.p66) |
| *Geometry file* | Calc QCNGS Geometry (.g09) |
| *Unsteady Flow File* | 1965 Flood (.u30) |
| *Output* | QuadCities-Mississ.dss |
| | **Calibration - 2001 Flood** |
| HEC-RAS | HEC-RAS - Calibration |
| *Project* | Quad Cities-Mississippi River |
| *Plan* | 2001 Flood Calibration (.p67) |
| *Geometry file* | Calc QCNGS Geometry (.g09) |
| *Unsteady Flow File* | 2001 Flood (.u31) |
| *Output* | QuadCities-Mississ.dss |
| | **Calibration - 1993 Flood** |
| HEC-RAS | HEC-RAS - Calibration |
| *Project* | Quad Cities-Mississippi River |
| *Plan* | 1993 Flood Calibration (.p68) |
| *Geometry file* | Calc QCNGS Geometry (.g09) |
| *Unsteady Flow File* | 1993 Flood (.u32) |
| *Output* | QuadCities-Mississ.dss |
| | **Calibration - 1969 Flood** |
| HEC-RAS | HEC-RAS - Calibration |
| *Project* | Quad Cities-Mississippi River |
| *Plan* | 1969 Flood Calibration (.p69) |
| *Geometry file* | Calc QCNGS Geometry (.g09) |
| *Unsteady Flow File* | 1969 Flood (.u33) |
| *Output* | QuadCities-Mississ.dss |
| | **Calibration - 1997 Flood** |
| HEC-RAS | HEC-RAS - Calibration |
| *Project* | Quad Cities-Mississippi River |
| *Plan* | 1997 Flood Calibration (.p70) |
| *Geometry file* | Calc QCNGS Geometry (.g09) |
| *Unsteady Flow File* | 1997 Flood (.u34) |
| *Output* | QuadCities-Mississ.dss |

**Table 15.1 – Computer Models Corresponding to Probable Maximum Flood – Hydrology Analysis (Continued)**

| Computer Software | Computer Model Name |
|---|---|
| **All Season PMF - Watershed Centroid** | |
| HEC-RAS | HEC-RAS - PMF |
| *Project* | Quad Cities-Mississippi River |
| *Plan* | PMF-All Season-Watershed Centroid at 291 (.p01) |
| *Geometry file* | Final QCNGS Geometry (.g14) |
| *Unsteady Flow File* | PMF-All Season-Watershed Centroid at 291 (.u01) |
| *Output* | QCNGS.dss |
| **All Season PMF - Clinton Subwatershed Centroid** | |
| HEC-RAS | HEC-RAS - PMF |
| *Project* | Quad Cities-Mississippi River |
| *Plan* | PMF-All Season Clinton Centroid (.p02) |
| *Geometry file* | Final QCNGS Geometry (.g14) |
| *Unsteady Flow File* | PMF-All Season Clinton Centroid (.u02) |
| *Output* | QCNGS.dss |
| **All Season PMF - McGregor Subwatershed Centroid** | |
| HEC-RAS | HEC-RAS - PMF |
| *Project* | Quad Cities-Mississippi River |
| *Plan* | PMF-All Season-McGregor Centroid (.p03) |
| *Geometry file* | Final QCNGS Geometry (.g14) |
| *Unsteady Flow File* | PMF-All Season-McGregor Centroid (.u03) |
| *Output* | QCNGS.dss |
| **All Season PMF - Winona Subwatershed Centroid** | |
| HEC-RAS | HEC-RAS - PMF |
| *Project* | Quad Cities-Mississippi River |
| *Plan* | PMF-All Season-Winona Centroid (.p04) |
| *Geometry file* | Final QCNGS Geometry (.g14) |
| *Unsteady Flow File* | PMF-All Season-Winona Centroid (.u04) |
| *Output* | QCNGS.dss |
| **All Season PMF - Anoka Subwatershed Centroid** | |
| HEC-RAS | HEC-RAS - PMF |
| *Project* | Quad Cities-Mississippi River |
| *Plan* | PMF-All Season-Anoka Centroid (.p05) |
| *Geometry file* | Final QCNGS Geometry (.g14) |
| *Unsteady Flow File* | PMF-All Season-Anoka Centroid (.u05) |
| *Output* | QCNGS.dss |

**Table 15.1 – Computer Models Corresponding to Probable Maximum Flood – Hydrology Analysis (Continued)**

| Computer Software | Computer Model Name |
|---|---|
| **All Season PMF - Watershed Centroid Alternative 2** | |
| HEC-RAS | HEC-RAS - PMF |
| *Project* | Quad Cities-Mississippi River |
| *Plan* | PMF-All Season-Watershed Centroid at 254 (.p09) |
| *Geometry file* | Final QCNGS Geometry (.g14) |
| *Unsteady Flow File* | PMF-All Season-Watershed Centroid at 254 (.u06) |
| *Output* | QCNGS.dss |
| **All Season PMF - Nonlinear** | |
| HEC-RAS | HEC-RAS - PMF |
| *Project* | Quad Cities-Mississippi River |
| *Plan* | PMF-All Season-Nonlinear (.p10) |
| *Geometry file* | Final QCNGS Geometry (.g14) |
| *Unsteady Flow File* | PMF-All Season-Nonlinear (.u07) |
| *Output* | QCNGS.dss |
| **All Season PMF - Moving Storm** | |
| HEC-RAS | HEC-RAS - PMF |
| *Project* | Quad Cities-Mississippi River |
| *Plan* | PMF-All Season-Moving Storm (.p52) |
| *Geometry file* | Final QCNGS Geometry (.g14) |
| *Unsteady Flow File* | PMF-All Season-Moving Storm (.u35) |
| *Output* | QCNGS.dss |
| **Cool Season PMF - Moving Storm** | |
| HEC-RAS | HEC-RAS - PMF |
| *Project* | Quad Cities-Mississippi River |
| *Plan* | PMF-Cool Season-Moving (.p56) |
| *Geometry file* | Final QCNGS Geometry (.g14) |
| *Unsteady Flow File* | PMF-Cool Season-Moving (.u38) |
| *Output* | QCNGS.dss |
| **Cool Season PMF - Watershed Centroid** | |
| HEC-RAS | HEC-RAS - PMF |
| *Project* | Quad Cities-Mississippi River |
| *Plan* | PMF-Cool Season Centroid (.p57) |
| *Geometry file* | Final QCNGS Geometry (.g14) |
| *Unsteady Flow File* | PMF-Cool Season Centroid (.u39) |
| *Output* | QCNGS.dss |

**Table 15.1 – Computer Models Corresponding to Probable Maximum Flood – Hydrology Analysis (Continued)**

| Computer Software | Computer Model Name |
|---|---|
| **Cool Season PMF - Nonlinear** | |
| HEC-RAS | HEC-RAS - PMF |
| *Project* | Quad Cities-Mississippi River |
| *Plan* | PMF-Cool Season-Nonlinear-McGregor (.p58) |
| *Geometry file* | Final QCNGS Geometry (.g14) |
| *Unsteady Flow File* | PMF-Cool Season-Nonlinear-McGregor (.u40) |
| *Output* | QCNGS.dss |
| **Cool Season PMF - Winona Subwatershed Centroid** | |
| HEC-RAS | HEC-RAS - PMF |
| *Project* | Quad Cities-Mississippi River |
| *Plan* | PMF-Cool Season-Winona Centroid (.p59) |
| *Geometry file* | Final QCNGS Geometry (.g14) |
| *Unsteady Flow File* | PMF-Cool Season-Winona Centroid (.u41) |
| *Output* | QCNGS.dss |
| **Cool Season PMF - Anoka Subwatershed Centroid** | |
| HEC-RAS | HEC-RAS - PMF |
| *Project* | Quad Cities-Mississippi River |
| *Plan* | PMF-Cool Season-Anoka Centroid (.p60) |
| *Geometry file* | Final QCNGS Geometry (.g14) |
| *Unsteady Flow File* | PMF-Cool Season-Anoka Centroid (.u42) |
| *Output* | QCNGS.dss |
| **Cool Season PMF - Clinton Subwatershed Centroid** | |
| HEC-RAS | HEC-RAS - PMF |
| *Project* | Quad Cities-Mississippi River |
| *Plan* | PMF-Cool Season-Clinton Centroid (.p61) |
| *Geometry file* | Final QCNGS Geometry (.g14) |
| *Unsteady Flow File* | PMF-Cool Season-Clinton Centroid (.u43) |
| *Output* | QCNGS.dss |
| **Cool Season PMF - McGregor Subwatershed Centroid** | |
| HEC-RAS | HEC-RAS - PMF |
| *Project* | Quad Cities-Mississippi River |
| *Plan* | PMF-Cool Season-McGregor Centroid (.p61) |
| *Geometry file* | Final QCNGS Geometry (.g14) |
| *Unsteady Flow File* | PMF-Cool Season-McGregor Centroid (.u43) |
| *Output* | QCNGS.dss |

**RAI 16: Flooding on Streams and Rivers – Manning's n Roughness Coefficient**

Background: The description of the PMF analysis in FHRR Section 3.1 states that the Manning's n roughness coefficient was the main parameter adjusted during HEC-RAS hydraulic model calibrations. However, the FHRR does not adequately describe how this adjustment was performed or discuss the adequacy of the final coefficient values.

Request: Provide additional details on the initial and final values of Manning's n roughness coefficient, and describe how those values compare with recommended values in standard references for site conditions existing in the watershed. For example, provide a table similar to Table 1 in the LIP evaluation report, which includes the type of surface coverage and the extent of coverage.

**Response:**

Initial Manning's Roughness Coefficients for the Mississippi River, Apple River, Wapsipinicon River, and Rock River originated from the UNET model included as part of the U.S. Army Corps of Engineers (USACE) Upper Mississippi River System Flow Frequency Study and were verified using published guidance for selection of Manning's Roughness Coefficients based on Chow, 1959 (Reference 1). The Manning's Roughness Coefficient was used as a calibration parameter. Initial and final values (calibrated) of Manning's Roughness Coefficients between Lock & Dam No. 13 (cross section 522.5) and Lock & Dam No. 14 (cross section 493.4) are tabulated in Table 16.1. Quad Cities Nuclear Power Station (QCNPS) is located at section 506.9 (Mississippi River, Reach #11), which crosses the northern portion of QCNPS. Manning's Roughness Coefficients set as follows:

1. Cap Manning's Roughness Coefficients values to 0.10 for the developed areas (i.e., residential, commercial and industrial areas)
2. Use Manning's Roughness Coefficient with 0.08 for forest areas. Chow, 1959 recommend Manning's Roughness Coefficients in the range of 0.080 to 0.120 for forest areas.
3. Use Manning's Roughness Coefficient values within 0.03 for agriculture areas. Chow, 1959 recommends Manning's Roughness Coefficients in the range of 0.030 to 0.050 for agricultural areas.

As discussed above, for some cross sections the lower end of Manning's roughness coefficients from Chow, 1959 were used. However, for calibration purposes, the Manning roughness coefficients were adjusted to better match computed profiles to observed historical water surface elevations. The results of the QCNPS HEC-RAS model calibration (Table 16.2) show that four of the simulated calibration floods (1965, 2001, 1993, and 1969) generated a peak water surface elevation at QCNPS within 0.5 foot of the observed elevations, and one calibration flood (1997) slightly exceeded the target elevation difference by 0.2 foot (i.e., +0.7 foot difference between simulated and observed). Based on the calibration results, the QCNPS model is judged to be acceptably calibrated. Using the higher end values would be more conservative, but the results would deviate from observed elevation and would not be considered sound calibration.

Based on a visual examination of aerial photographs of the QCNPS watershed, mainly residential, commercial, industrial, and forested areas exist (see Figures 16.1 through 16.4). In coomparison to published values in Chow, 1959, the calibrated Manning's roughness coefficients are representative of the site conditions existing in the QCNPS watershed.

**Table 16.1 Comparison of UNET and Calibrated Manning's Roughness Coefficients**

| | | Final (Calibrated Manning's Roughness Coefficients) | | | Initial Manning's Roughness Coefficients (USACE UNET) | | |
|---|---|---|---|---|---|---|---|
| River | River Station | Left Overbank | Channel | Right Overbank | Left Overbank | Channel | Right Overbank |
| MISSISSIPPI | 522.5 L&D 13 TAIL - RM | 0.03 | 0.0275 | 0.08 | 0.04 | 0.025 | 0.08 |
| MISSISSIPPI | 522.4 | 0.03 | 0.0275 | 0.077 | 0.04 | 0.025 | 0.07 |
| MISSISSIPPI | 522.3 | 0.03 | 0.0275 | 0.077 | 0.04 | 0.025 | 0.07 |
| MISSISSIPPI | 522.2 | 0.03 | 0.0275 | 0.077 | 0.04 | 0.025 | 0.07 |
| MISSISSIPPI | 521.7 | 0.03 | 0.0275 | 0.1 | 0.04 | 0.025 | 0.1 |
| MISSISSIPPI | 521.2 | 0.1 | 0.0275 | 0.1 | 0.1 | 0.025 | 0.12 |
| MISSISSIPPI | 521 | 0.1 | 0.0275 | 0.1 | 0.04 | 0.025 | 0.12 |
| MISSISSIPPI | 520.6 | 0.1 | 0.0275 | 0.1 | 0.04 | 0.025 | 0.12 |
| MISSISSIPPI | 520.4 | 0.1 | 0.0275 | 0.1 | 0.04 | 0.025 | 0.12 |
| MISSISSIPPI | 520 | 0.03 | 0.0275 | 0.1 | 0.04 | 0.025 | 0.12 |
| MISSISSIPPI | 519.95 | 0.03 | 0.0275 | 0.1 | 0.04 | 0.025 | 0.12 |
| MISSISSIPPI | 519.9 | 0.03 | 0.0275 | 0.1 | 0.04 | 0.025 | 0.12 |
| MISSISSIPPI | 519.75* | 0.03 | 0.0275 | 0.08 | 0.04 | 0.025 | 0.12 |
| MISSISSIPPI | 519.6 | 0.03 | 0.0275 | 0.1 | 0.04 | 0.025 | 0.12 |
| MISSISSIPPI | 519.1 | 0.03 | 0.0275 | 0.1 | 0.04 | 0.025 | 0.12 |
| MISSISSIPPI | 518.4 | 0.03 | 0.0275 | 0.1 | 0.04 | 0.025 | 0.12 |
| MISSISSIPPI | 518.15 | 0.03 | 0.0275 | 0.1 | 0.04 | 0.025 | 0.12 |
| MISSISSIPPI | 518.1 | 0.055 | 0.0275 | 0.1 | 0.05 | 0.025 | 0.12 |
| MISSISSIPPI | 518.05 | 0.055 | 0.0275 | 0.1 | 0.04 | 0.025 | 0.12 |
| MISSISSIPPI | 518 CLINTON - RM 518 | 0.03 | 0.0275 | 0.1 | 0.04 | 0.025 | 0.12 |
| MISSISSIPPI | 517.95 | 0.03 | 0.0275 | 0.1 | 0.04 | 0.025 | 0.12 |
| MISSISSIPPI | 517.7 | 0.077 | 0.0275 | 0.1 | 0.07 | 0.025 | 0.12 |
| MISSISSIPPI | 517 | 0.03 | 0.0275 | 0.1 | 0.04 | 0.025 | 0.12 |
| MISSISSIPPI | 516.6 | 0.03 | 0.0275 | 0.1 | 0.04 | 0.025 | 0.12 |
| MISSISSIPPI | 516 | 0.055 | 0.0275 | 0.1 | 0.12 | 0.025 | 0.12 |
| MISSISSIPPI | 515.5 | 0.055 | 0.0275 | 0.1 | 0.05 | 0.025 | 0.12 |
| MISSISSIPPI | 515 | 0.066 | 0.0275 | 0.1 | 0.06 | 0.025 | 0.12 |
| MISSISSIPPI | 514.4 | 0.066 | 0.0275 | 0.1 | 0.06 | 0.025 | 0.12 |
| MISSISSIPPI | 514 | 0.03 | 0.0275 | 0.1 | 0.06 | 0.025 | 0.04 |
| MISSISSIPPI | 513 | 0.1 | 0.0275 | 0.03 | 0.04 | 0.025 | 0.04 |
| MISSISSIPPI | 512.7 | 0.03 | 0.0275 | 0.03 | 0.04 | 0.025 | 0.04 |
| MISSISSIPPI | 512 CAMANCHE DS - RM | 0.03 | 0.0275 | 0.03 | 0.04 | 0.025 | 0.04 |

**Table 16.1 Comparison of UNET and Calibrated Manning's Roughness Coefficients (Continued)**

| River | River Station | Final (Calibrated Manning's Roughness Coefficients) | | | Initial Manning's Roughness Coefficients (USACE UNET) | | |
|---|---|---|---|---|---|---|---|
| | | Left Overbank | Channel | Right Overbank | Left Overbank | Channel | Right Overbank |
| MISSISSIPPI | 511.6 CAMANCHE - FLOW | 0.03 | 0.0275 | 0.03 | 0.04 | 0.025 | 0.04 |
| MISSISSIPPI | 511 | 0.03 | 0.0275 | 0.03 | 0.04 | 0.025 | 0.04 |
| MISSISSIPPI | 510.1 | 0.03 | 0.0275 | 0.066 | 0.04 | 0.025 | 0.06 |
| MISSISSIPPI | 509.3 | 0.03 | 0.0275 | 0.03 | 0.04 | 0.025 | 0.06 |
| MISSISSIPPI | 509 | 0.03 | 0.0275 | 0.03 | 0.04 | 0.025 | 0.06 |
| MISSISSIPPI | 508.6 | 0.03 | 0.0275 | 0.03 | 0.04 | 0.025 | 0.04 |
| MISSISSIPPI | 507.9 | 0.03 | 0.0275 | 0.03 | 0.04 | 0.025 | 0.05 |
| MISSISSIPPI | 507.4 | 0.03 | 0.0275 | 0.03 | 0.04 | 0.025 | 0.06 |
| MISSISSIPPI | **506.9 (QCNPS)** | **0.03** | **0.0275** | **0.08** | **0.09** | **0.025** | **0.08** |
| MISSISSIPPI | 506 | 0.99 | 0.0275 | 0.03 | 0.09 | 0.025 | 0.08 |
| MISSISSIPPI | 505.5 | 0.066 | 0.0275 | 0.03 | 0.06 | 0.025 | 0.04 |
| MISSISSIPPI | 505 | 0.03 | 0.0275 | 0.03 | 0.04 | 0.025 | 0.04 |
| MISSISSIPPI | 504 | 0.03 | 0.0275 | 0.03 | 0.04 | 0.025 | 0.04 |
| MISSISSIPPI | 503.3 | 0.03 | 0.0275 | 0.03 | 0.04 | 0.025 | 0.04 |
| MISSISSIPPI | 503.1 | 0.03 | 0.0275 | 0.03 | 0.04 | 0.025 | 0.05 |
| MISSISSIPPI | 502.9 | 0.03 | 0.0275 | 0.03 | 0.04 | 0.025 | 0.04 |
| MISSISSIPPI | 502.5 | 0.099 | 0.0275 | 0.1 | 0.09 | 0.025 | 0.12 |
| MISSISSIPPI | 502 PRINCETON - RM 5 | 0.1 | 0.0275 | 0.1 | 0.12 | 0.025 | 0.12 |
| MISSISSIPPI | 501.74 | 0.08 | 0.0275 | 0.1 | 0.08 | 0.025 | 0.12 |
| MISSISSIPPI | 501.5 | 0.055 | 0.0275 | 0.1 | 0.05 | 0.025 | 0.12 |
| MISSISSIPPI | 501 | 0.08 | 0.0275 | 0.1 | 0.08 | 0.025 | 0.11 |
| MISSISSIPPI | 500.5 | 0.1 | 0.0275 | 0.055 | 0.12 | 0.025 | 0.05 |
| MISSISSIPPI | 500 | 0.1 | 0.0275 | 0.1 | 0.1 | 0.025 | 0.12 |
| MISSISSIPPI | 499.5 | 0.1 | 0.0275 | 0.1 | 0.1 | 0.025 | 0.12 |
| MISSISSIPPI | 499 | 0.1 | 0.0275 | 0.1 | 0.12 | 0.025 | 0.12 |
| MISSISSIPPI | 498.5 | 0.1 | 0.0275 | 0.055 | 0.12 | 0.025 | 0.05 |
| MISSISSIPPI | 498 | 0.1 | 0.0275 | 0.1 | 0.12 | 0.025 | 0.12 |
| MISSISSIPPI | 497.1 LECLAIRE - RM 49 | 0.1 | 0.0275 | 0.1 | 0.12 | 0.025 | 0.12 |
| MISSISSIPPI | 496.8 | 0.1 | 0.0275 | 0.1 | 0.12 | 0.025 | 0.12 |
| MISSISSIPPI | 496.5 | 0.1 | 0.0275 | 0.1 | 0.12 | 0.025 | 0.12 |
| MISSISSIPPI | 496 | 0.1 | 0.0275 | 0.1 | 0.12 | 0.025 | 0.12 |

**Table 16.1 Comparison of UNET and Calibrated Manning's Roughness Coefficients (Continued)**

| River | River Station | Final (Calibrated Manning's Roughness Coefficients) | | | Initial Manning's Roughness Coefficients (USACE UNET) | | |
|---|---|---|---|---|---|---|---|
| | | Left Overbank | Channel | Right Overbank | Left Overbank | Channel | Right Overbank |
| MISSISSIPPI | 495.3 | 0.1 | 0.0275 | 0.1 | 0.12 | 0.025 | 0.12 |
| MISSISSIPPI | 495 | 0.1 | 0.0275 | 0.1 | 0.12 | 0.025 | 0.12 |
| MISSISSIPPI | 494.6 | 0.1 | 0.0275 | 0.1 | 0.12 | 0.025 | 0.12 |
| MISSISSIPPI | 494 | 0.1 | 0.0275 | 0.1 | 0.1 | 0.025 | 0.12 |
| MISSISSIPPI | 493.4 L&D 14 POOL | 0.1 | 0.0275 | 0.1 | 0.11 | 0.025 | 0.12 |

**Table 16.2: Observed versus Predicted Water Surface Comparison**

| Flood | Elevation, ft (MSL 1912) | | Difference |
|---|---|---|---|
| | Observed | Modeled X-Section 506.9 | (feet) |
| 1965 | 586.0 | 586.2 | 0.2 |
| 2001 | 584.9 | 584.4 | -0.5 |
| 1993 | 584.2 | 584.1 | -0.1 |
| 1969 | 583.0 | 583.0 | 0.0 |
| 1997 | 582.5 | 583.2 | 0.7 |

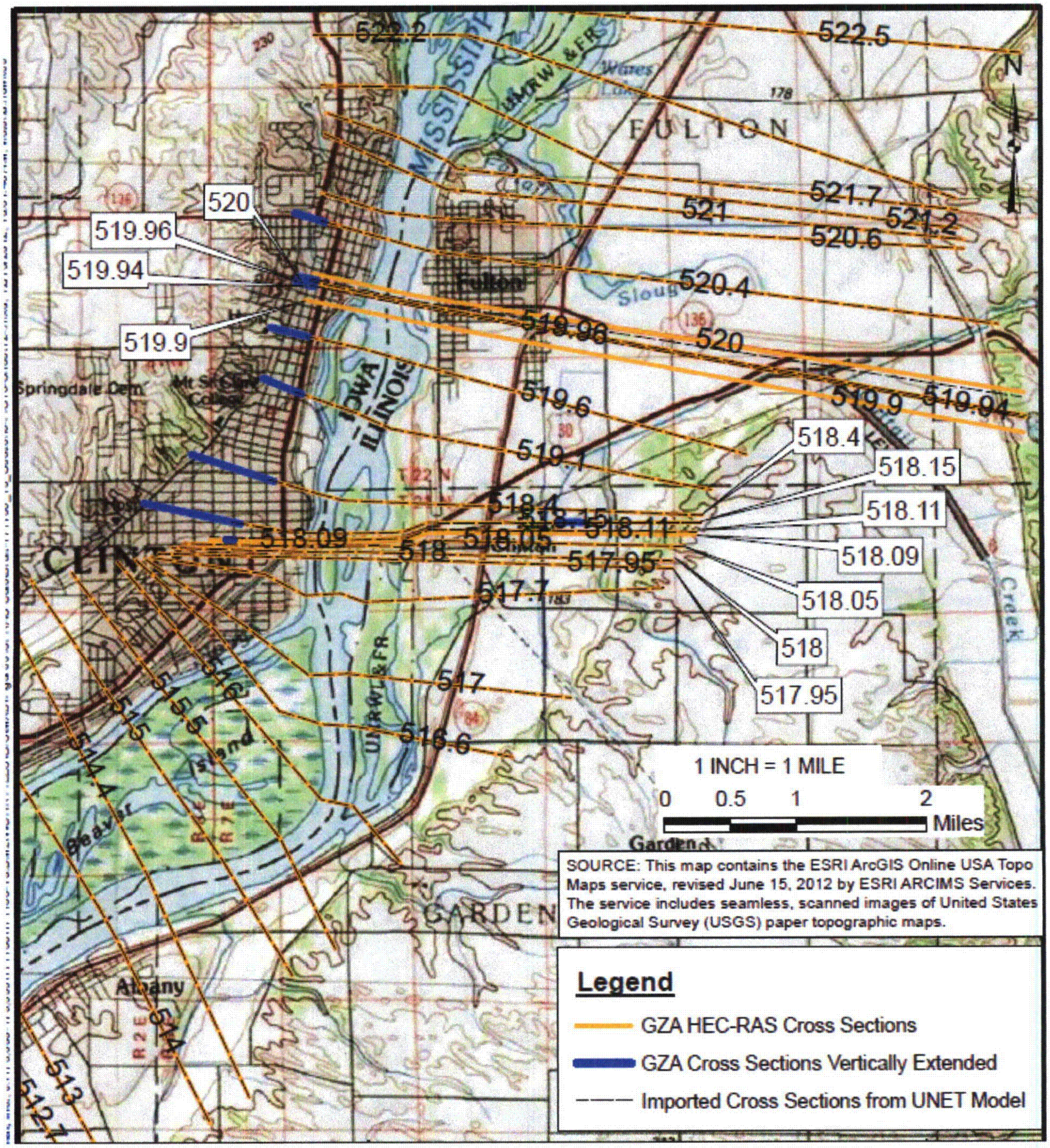



**Figure 16.1: HEC-RAS Cross Sections Locations on Top Of Topographic Map (Cross Section 522.5 to 512.7)**

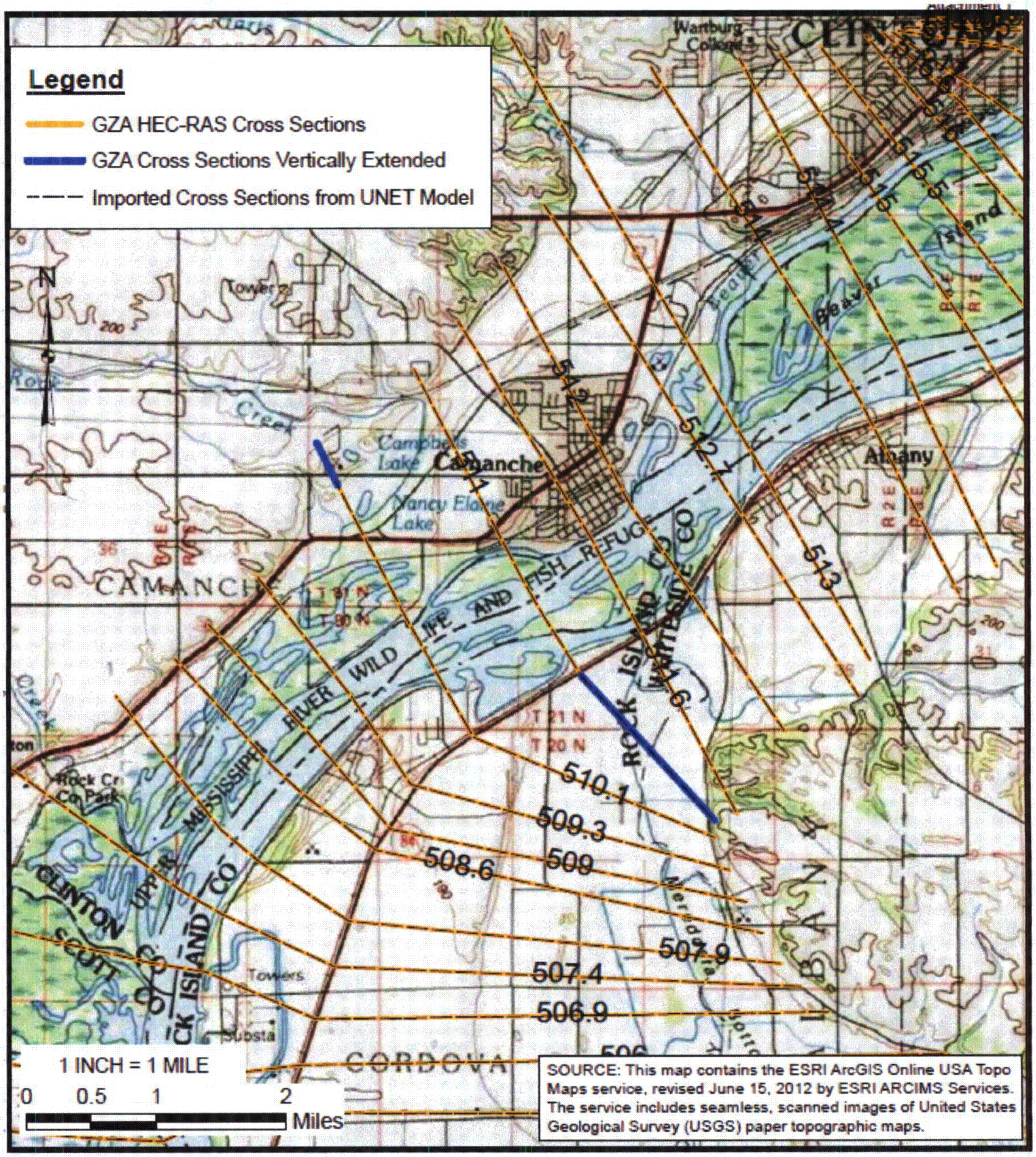



**Figure 16.2: HEC-RAS Cross Sections Locations on Top Of Topographic Map (Cross Section 512.7 to 506)**

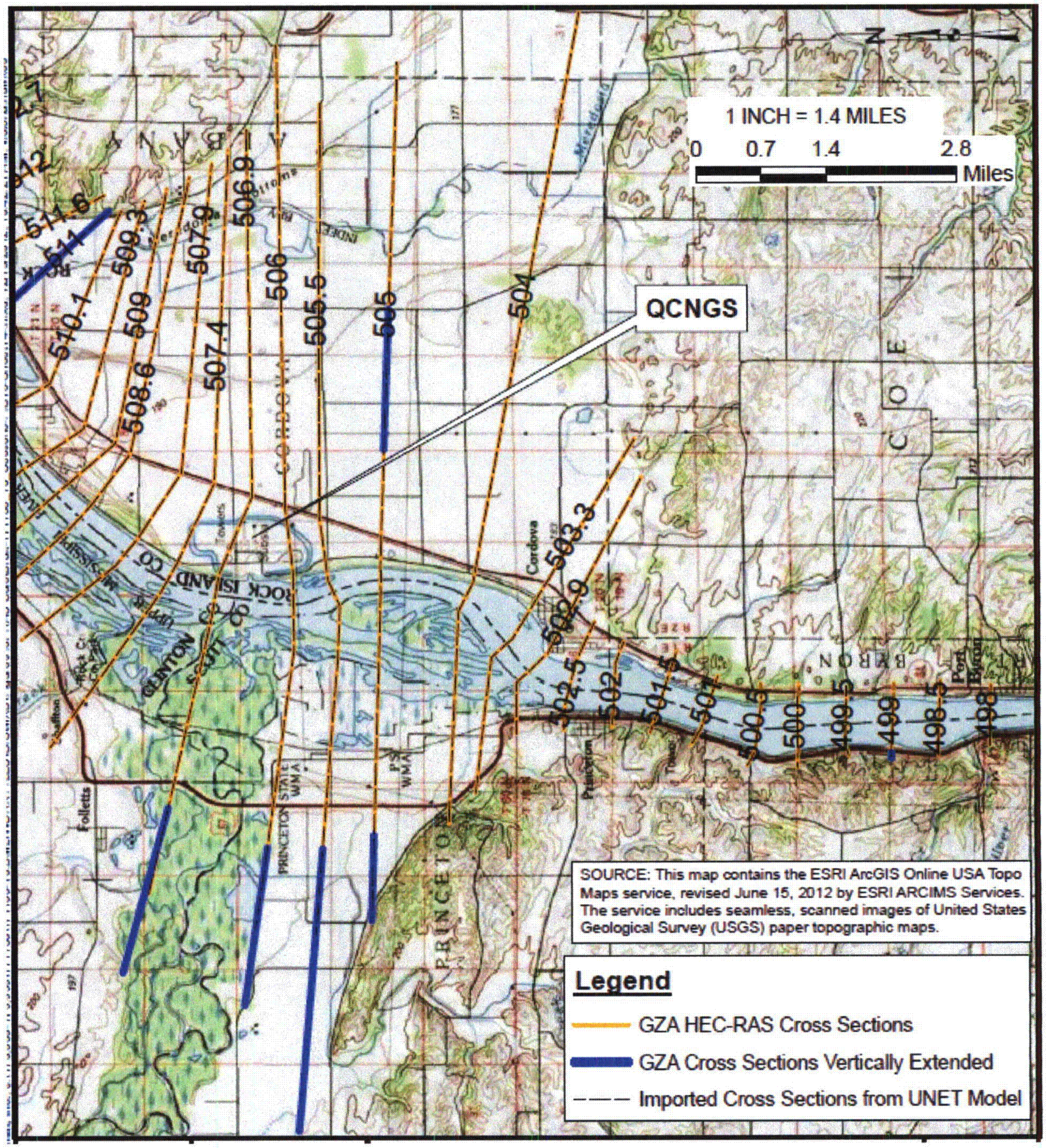



**Figure 16.3: HEC-RAS Cross Sections Locations on Top Of Topographic Map (Cross Section 512 to 498)**

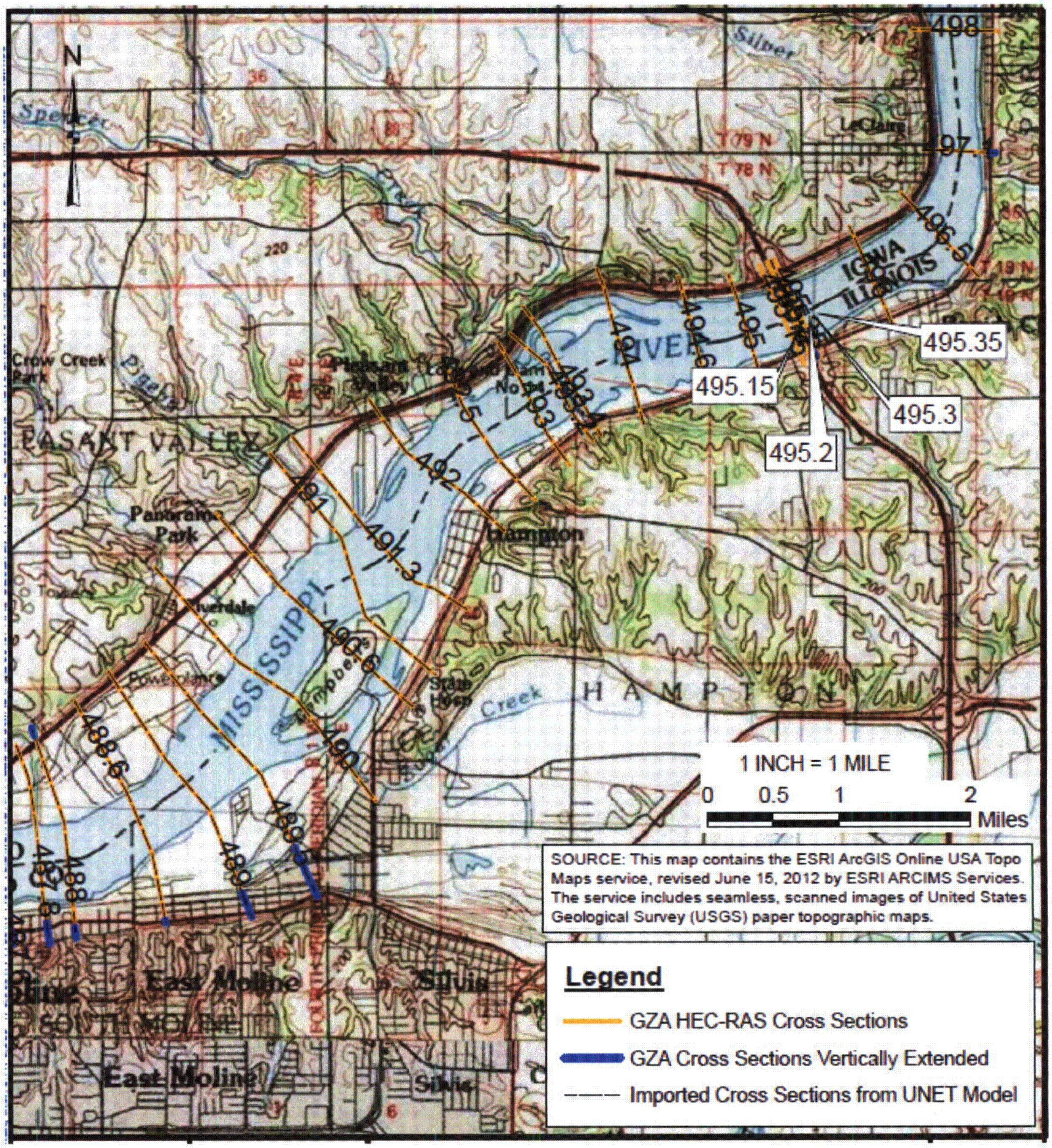



**Figure 16.4: HEC-RAS Cross Sections Locations on Top Of Topographic Map (Cross Section 498 to 488)**

References:

1. Open-Channel Hydraulics, Ven Te Chow, Reprint of the 1959 Edition, McGraw Hill Book Company.

**RAI 17: Dam Failure – Supporting Analysis and Electronic Files**

Background: The discussion of the upstream dam failure flood analysis in FHRR Section 3.2 does not adequately describe the key assumptions, methods, and results. The NRC staff audit found that this information is documented, in part, in Calculation Package QDC-0085-S-2032, "Upstream Dam Failure Flood Evaluations at QCNGS."

Request: Provide the following portions of QDC-0085-S-2032:

- Main text (Pages 1-49)
- Attachment 1, Figures
- Attachment 3, Dams Information
- Attachment 4, Datum Conversions
- Attachment 5, Reservoir Storage Information
- Attachment 7, Muskingum-Cunge Parameters

Also provide electronic editable files for the following:

- QDC-0085-S-2032, Attachment 2, Major Dams in Watershed
- QDC-0085-S-2032, Attachment 10A, NID Subwatershed Dams
- HEC-RAS hydrologic model and HEC-RAS hydraulic model input and output files used for surface flow modeling of the individual and cascading dam failure events discussed in FHHR Enclosure 2, Section 3.2
- HEC-RAS hydrologic model input and output files used for the screening analysis for failure of all dams in the upstream watershed (i.e., Approach 2) discussed in FHHR Section 3.2

**Response:**

The requested excerpts from Calculation QDC-0085-S-2032 and electronic editable files are included in Enclosure 3 DVD #2A under folder labeled "RAI 17". The computer model names corresponding to the critical scenarios are specified in Table 17.1.

**Table 17.1 – Computer Models Corresponding to Dam Failure Analysis**

| Computer Software | Computer Model Name |
|---|---|
| | **Approach 1 - Hydrologic** |
| HEC-HMS | HEC-HMS Approach 1\QCNGS_DamFailures |
| HEC-RAS | HEC-RAS Approach 1\DamFailureModel-Delivered_5-27-14 |
| *Project* | QCNGS-Dam Failure |
| *Plan* | Hydrologic (PMF) Dam Failure (.p08) |
| *Geometry file* | Final QCNGS Geometry (.g14) |
| *Unsteady Flow File* | Hydrologic (PMF) Dam Failure (.u78) |
| | **Approach 1 - Seismic** |
| HEC-HMS | HEC-HMS Approach 1\QCNGS_DamFailures |
| HEC-RAS | HEC-RAS Approach 1\DamFailureModel-Delivered_5-27-14 |
| *Project* | QCNGS-Dam Failure |
| *Plan* | Seismic Day-Dambreak Dam 13 (.p22) |
| *Geometry file* | Final QCNGS Geometry (.g14) |
| *Unsteady Flow File* | Seismic Dam Failure (.u79) |
| | **Approach 1 - Sunny Day** |
| HEC-HMS | HEC-HMS Approach 1\QCNGS_DamFailures |
| HEC-RAS | HEC-RAS Approach 1\DamFailureModel-Delivered_5-27-14 |
| *Project* | QCNGS-Dam Failure |
| *Plan* | Sunny Day-Dam Failure (.p17) |
| *Geometry file* | Final QCNGS Geometry (.g14) |
| *Unsteady Flow File* | Sunny Day Dam Failure (.u77) |

**Table 17.1 – Computer Models Corresponding to Dam Failure Analysis (Continued)**

| Computer Software | Computer Model Name |
|---|---|
| **Approach 2 - Hydrologic** | |
| HEC-HMS | HEC-HMS Approach 2\Hypothetical_Dam_PMF |
| HEC-RAS | HEC-RAS Approach 2 |
| *Project* | QCNGS Dam Failure Combined |
| *Plan* | PMF Dambreak Dam Combined (.p14) |
| *Geometry file* | Final QCNGS Geometry (.g14) |
| *Unsteady Flow File* | PMF-Cool Season-McGregor Centroid (.u82) |
| **Approach 2 - Seismic** | |
| HEC-HMS | HEC-HMS Approach 2\Hypothetical_Dam_Seismic |
| HEC-RAS | HEC-RAS Approach 2 |
| *Project* | QCNGS Dam Failure Combined |
| *Plan* | Seismic Dam Failure (.p15) |
| *Geometry file* | Final QCNGS Geometry (.g14) |
| *Unsteady Flow File* | Seismic Dam Failure(.u83) |

**RAI 18: Dam Failure – Large Dam Criteria**

Background: Many dam safety agencies consider dams over 50 ft in height as "large dams." However, the dam failure flood analysis reported in FHRR Section 3.2 limited consideration to dams over 60 feet in height (labeled "significant" dams). The limitation to dams over 60 feet in height may potentially exclude some dams that could have a significant impact on estimated flood levels due to dam failure.

Request: Provide justification for limiting consideration to dams over 60 feet in height in the dam failure flood analysis.

**Response:**

There are approximately 236 major dams in the watershed contributory to Quad Cities Nuclear Power Station (QCNPS), based on the criteria used in National Atlas database. National Atlas criteria for major dams is - "*Major dams include dams 50 feet or more in height, dams with a normal storage capacity of 5,000 acre-feet or more, and dams with a maximum storage capacity of 25,000 acre-feet or more.*"

The criterion for major dams described in the National Atlas was used in selection of dams within 100 miles. All dams within a 100 mile radius of QCNPS that meet the major dam criteria were included in the dam failure analysis. Only two dams within the 100 miles radius have a

height between 50 feet and 60 feet with normal storage capacity of 163 acre-feet and 670 acre-feet, and with maximum storage capacity of 325 acre-feet and 2,130 acre–feet respectively. The normal and maximum storage capacities for these two dams are well below the major dam criteria described in the National Atlas.

Therefore, there are no dams within the 100 mile radius, between 50 feet to 60 feet height range, which meets the major dam criteria described in the National Atlas for normal and maximum storage. The minimum dam height in the QCNPS watershed that meets the National Atlas criteria for major dams is 60 feet, therefore, 60 feet was listed as the limiting criteria for the QCNPS watershed in the FHRR.

In summary, the major dams criteria described in National Atlas was considered in selecting individual dams for the dam failure analysis.

Based on the database in National Inventory of Dams and the National Atlas, most dams in the watershed are relatively low-head or low-storage due to the limited topographic relief in the watershed.

Dams located more than 100 miles upstream were determined to not significantly contribute to peak flooding at QCNPS. This is because dam failure flood waves attenuate as they travel downstream, and the flood-carrying capacity of the Mississippi River and its 88,000 square-miles drainage area at QCNPS is able to accommodate significant flow below site grade elevation (Calculation No. QDC-0085-S-1992).

Eau Galle Reservoir is the largest dam in the QCNPS watershed lies within 250 miles of QCNPS. Based on a combination of height and storage, Eau Galle Reservoir Dam was incorporated into the dam failure analysis. The 127-foot high Eau Galle Reservoir Dam is the lone dam in the watershed with a height of over 100 feet.

Additionally, Lock & Dams 11, 12, and 13 were included in the dam failure analysis because they are on the Mississippi River within 100 miles of QCNPS. The locks and dams are low-head structures that might be submerged during flood conditions, which would result in relatively minor dam breach flood flows. A domino failure of the three lock dams plus the dam breach flows (plus flood flows, as applicable) from other upstream dams on tributaries to the Mississippi River was used in the dam failure analysis.

**RAI 19: Dam Failure – Failure of All Upstream Dams**

Background: FHHR Section 3.2 describes a screening method that considers all upstream dams by lumping them into several hypothetical dams (referred to as Approach 2). However, the FHRR does not adequately describe the modeling decisions made when lumping upstream dams into hypothetical dams.

Request: Describe how dams were lumped, including justification of any adjustments made to storage volumes during the lumping process.

**Response:**

This screening analysis was developed before the publication of the Nuclear Regulatory Commission's (NRC's) Guidance for Assessment of Flooding Hazard Due to Dam Failure, JLD-ISG-2013-01, Revision 0; dated July 29, 2013 was published. However, this analysis incorporates conservative concepts discussed in JLD-ISG-2013-01.

Two approaches were considered to evaluate flooding from upstream dam failure (Calculation QDC-0085-S-2032). Approach 1 considered failure of a subset of the upstream dams to develop a conservative but representative upstream dam failure scenario based on ANSI/ANS 2.8 guidance, which states that some dams can be eliminated from dam failure analysis based on "low head differential, small volume, distance from plant site, and major intervening natural or reservoir detention capacity." Smaller and more remote dams were excluded in Approach 1 per the ANSI/ANS 2.8 guidance. Approach 2 was applied for sensitivity purpose only and introduced additional conservatism by evaluating failure of all 1,558 upstream dams within the watershed, represented in the hydrologic model as hypothetical dams. The NRC Dam Failure Interim Staff Guidance (July, 2013) was released after Quad Cities Flood Hazard Reevaluation report was submitted and, therefore, was not applicable to the reevaluation. The approach was developed based on ongoing discussions, at the time the reevaluation was being conducted, of dam failure methodology between the NRC and the Nuclear Energy Institute (NEI). The information provided in response to the Request for Information is based on results from Dam Failure Approach 1. Approach 2 was performed only as a sensitivity study and final results are based on Approach 1.

A screening level conservative dam failure analysis was performed for the dams listed in the National Inventory of Dams (NID) database within the contributory watershed to Quad Cities Nuclear Power Station (QCNPS). Representative hypothetical dams were created in each of the ten sub-watersheds. Dams that were modeled individually as part of Approach 1, were not included in the hypothetical reservoirs/dams. The Approach 1 dams were failed individually in Approach 2.

The geometry and characteristics of the hypothetical dam structures were based on the NID database, developed and maintained by the U.S. Army Corps of Engineers (USACE). The dams were then grouped by sub-watershed. A single hypothetical reservoir was created for each sub-watershed and inserted into the HEC-HMS rainfall-runoff model at the outlet of each respective sub-watershed.

Each single hypothetical reservoir contains 50 percent of the sum of the NID storage for all the NID dams in a given sub-watershed (excluding the individually modeled dams discussed in Approach 1). The use of 50 percent of the NID storage was selected to account for floodplain storage, flood control dams within the watershed and the fact that the probable maximum precipitation (PMP) does not cover the entirety of the large QCNPS watershed at any given time (due to meteorological limitations of the size of the storm). Typically, flood control dams are operated at a minimum below the maximum storage capacity or empty: in this case approximately 20 percent of all the dams located within the QCNPS watershed are flood control structures. It is also noted that a portion of these dams are likely to be designed to withstand flooding such as that experienced during the probable maximum flood (PMF) at QCNPS, particularly since the storm that produces the PMF would not typically produce a dam-specific PMF. However, this was conservatively not considered directly in this evaluation.

The storage represents each reservoir's maximum pool volume or normal pool volume, in the absence of maximum pool information. The hypothetical reservoir was represented in HEC-HMS using an elevation versus volume data table. The table included two points: a zero point at the toe of the dam and 50 percent of the total NID storage volume (in acre-feet) at the top of the dam.

The top of dam elevation was assigned based on the NID storage-weighted dam height of the dams in a given sub-watershed. The weighted dam height was calculated as follows:

$$Hypothetical\ Dam\ Height = \frac{Sum\ (NID\ Storage\ \times Dam\ Height)}{(Sum\ of\ Total\ Storage)}$$

The toe (i.e., bottom) elevation of the hypothetical dam was set to 0 feet. The top of dam elevation was set to be equal to toe of dam elevation plus the hypothetical dam height.

The dam breach width and side slope input parameters used for the sub-watershed hypothetical dams were based on published guidance and engineering judgment to represent the disparate locations of the dams within a given sub-watershed. The average breach width was selected as 2.5 times the hypothetical dam height for each dam and the time to failure selected as equal to the time of concentration for the corresponding sub-watershed. This represents the travel time and a portion of the attenuation of the dam break flood wave as it travels from its actual location to the outlet of the sub-watershed where the hypothetical dam in HEC-HMS is located. The average breach width was selected to represent the average breach widths of concrete and earthen embankment dams included in the hypothetical dam.

**RAI 20: Ice Jam Flooding – Supporting Information**

Background: The ice jam flooding analysis described in FHRR Section 3.6 does not adequately describe key assumptions, methods, and results. The NRC staff audit found that this information is documented, in part, in Calculation Package QDC-0085-S-2033, "Ice-Induced Flooding Evaluation at QCNGS."

Request: Provide the following portions of QDC-0085-S-2033:

- Main text (Pages 1-12)
- Attachment 1, Figures
- Attachment 3, Stage Flow Rating Curve, including the source for the curve

Also provide electronic editable files for the following:

- USACE Ice Jam Query Results (QDC-0085-S-2033, Attachment 2)
- HEC-RAS hydraulic model input and output files for the calculation of water surface elevations for the historic, upstream, and downstream ice jams discussed in FHRR Section 3.6

**Response:**

The requested excerpts from Calculation QDC-0085-S-2033 and electronic editable files are included in Enclosure 3 DVD #2A under folder labeled "RAI 20". The computer model names corresponding to the critical scenarios are specified in Table 20.1. The stage flow rating curve from Attachment 3 of Calculation QDC-0085-S-2033 is provided as Figure 20.1. The stage flow rating curve was developed using the USGS stream gage data and the HEC-RAS model developed as part of the Calculation QDC-0085-S-1991. Information on the rating curve in Figure 20.1 below beyond the theoretical maximum stillwater flood elevation of 600.9 MSL 1912 is not valid and should not be used. Meteorological and hydrological data does not support a flood of greater magnitude. Note also that the recurrence interval estimates shown on Figure 20.1 were developed in Calculation QDC-0085-S-1992 based on a simplified extrapolation of a Log-Pearson Type III probability distribution function. The methods used in this calculation follow guidance in USGS Bulletin 17B (Reference 1) but did not involve a rigorous and comprehensive probabilistic flood hazard assessment.

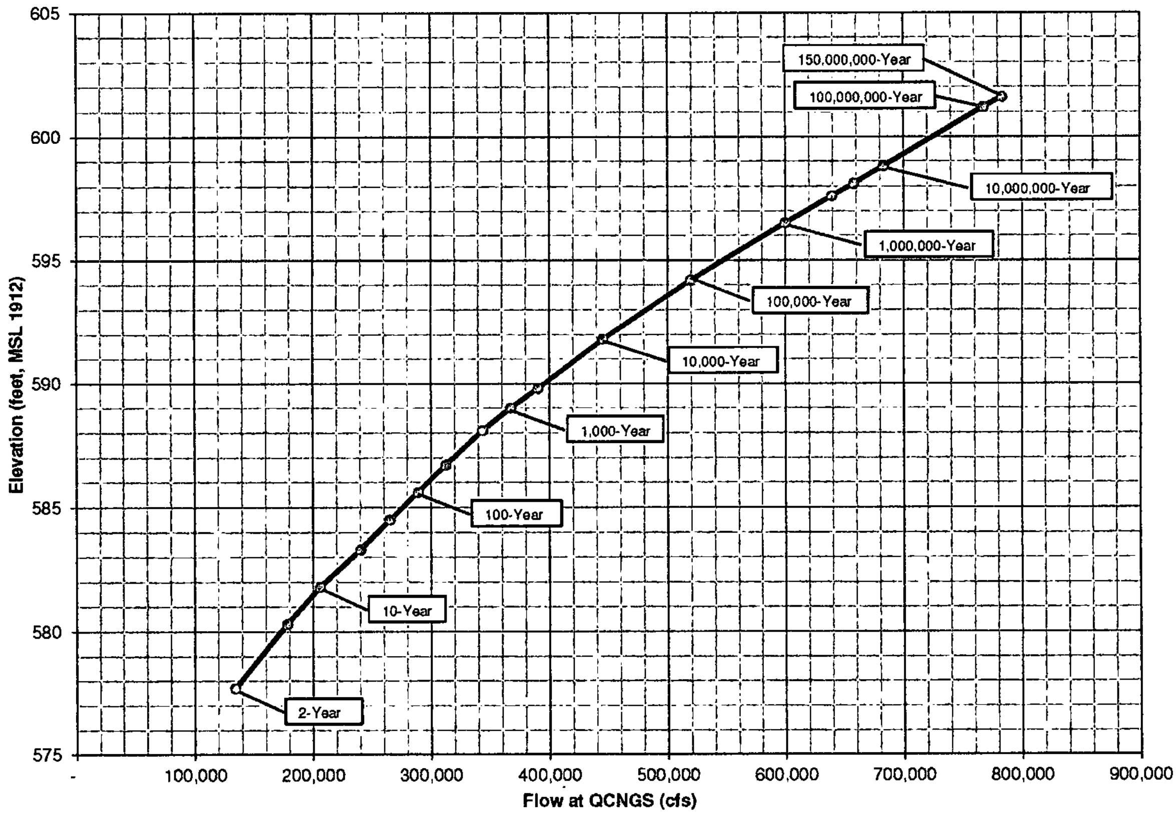



**Figure 20.1 – Stage Flow Rating Curve for QCNPS**

**Table 20.1 – Computer Models Corresponding to Upstream Ice Jam Failure Analysis**

| Computer Software | Computer Model Name |
|---|---|
| **Ice Jam Failure** | |
| HEC-RAS | RAI Response\RAI 20\HEC-RAS |
| *Project* | QDCNGS_Uppstream-Ice_Jam |
| *Plan* | Constant37000 (.p09) |
| *Geometry file* | Final QCNGS Geometry-UpstreamICEJAM (.g02) |
| *Unsteady Flow File* | Constant37000 (.u05) |
| *Output* | QDCNGS_Upstream-Ice.dss |

References:

1. U.S. Department of the Interior Geological Survey, "Guidelines for Determining Flood Flow Frequency Bulletin #17B", September 1981

**RAI 21: Ice Jam Flooding – Ice Jam Locations**

Background: NUREG/CR-7046, "Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America," November 2011 (ADAMS Accession No. ML11321A195), recommends that the size and location of the jam or dam and its breach parameters should be postulated conservatively to maximize the flood caused by release of impounded water. In part, this should include an examination of locations that may be susceptible to ice jam formation. The most common location for ice jam formation is a reach where the river slope decreases significantly. Other common locations include river bends and areas of obstructions, such as a bridge or dam piers. Confluences of tributary streams with larger rivers or confluences of rivers with lakes or reservoirs are also prone to ice jam formation. However, FHRR Section 3.6 identifies the first upstream and downstream bridge with no rational given for choosing these locations.

Request: Provide the rational for choosing to locate the ice jams at the first upstream and downstream bridges giving consideration to the common ice jam formation locations discussed above.

**Response:**

The Mississippi River near Quad Cities Nuclear Power Station (QCNPS) is regulated for navigational purposes and is not prone to significant hydraulic changes. QCNPS is located adjacent to Pool 14 of the Mississippi River, at river mile 506.9. Pool 14 spans 29.2 miles from upstream Lock & Dam 13 located at river mile 522.5 to Lock & Dam 14 located at river mile 493.3. From river mile 522.5 to 497 the slope of the Mississippi River is 0.13 feet per mile and from river mile 497 to 493.3 the slope is 1.5 feet per mile (Reference 1). Therefore, there is no significant decrease in slope within Pool 14.

There are two bends in the river within Pool 14, at approximately river mile 516 and 497:

a. The upstream bend (river mile 516) is located 2.1 miles downstream from the US 30 bridge (the upstream bridge where the ice jam was assumed to form). The results from the upstream ice jam failure at the US 30 bridge resulted in a flood elevation of 573.7 feet at QCNPS, 21.3 feet below site grade. The formation of the historic ice jam 2.1 miles further downstream would not result in an appreciable increase in the flood elevation at QCNPS. In addition, the bridge would more likely be the location for the build-up of ice rather than the bend in the natural river section due to obstructions from the bridge in the river.

b. The normal pool at the downstream bend (river mile 497) is equal to the normal pool at the US 80 bridge (the downstream bridge where the ice jam was assumed to form at river mile 495.4). The peak water surface elevation was calculated as the normal pool at QCNPS combined with the depth of the backwater resulting from the downstream ice jam. The depth of the backwater from a downstream ice jam was equal to the height of the ice jam. Therefore, the depth of backwater and the resulting water surface elevation would be equal regardless of the location of the downstream ice jam (either at the downstream bend or at the US 80 bridge).

The confluence of the Wapsipinicon River is located at river mile 506.9, on the opposite bank from QCNPS. The normal pool at river mile 506.9 is equal to the normal pool at the US 80 Bridge (the downstream bridge where the ice jam was assumed to form at river mile 495.4). The peak water surface elevation was calculated as the normal pool at QCNPS combined with the depth of the backwater resulting from the downstream ice jam. The depth of the backwater from a downstream ice jam was equal to the height of the ice jam. Therefore, the depth of backwater and the resulting water surface elevation would be equal regardless of the location of the downstream ice jam (either at the confluence of the Wapsipinicon River or at the US 80 bridge).

References:

1. Lock and Dam No. 14 Master Water Control Manual – Mississippi River Nine-Foot Channel Navigation Project-Appendix 14, U.S. Army Corps of Engineers, October 2002.

**RAI 22: Combined Effects – Supporting Information**

Background: The discussion of the combined effect flood analysis in FHRR Section 3.8 does not adequately describe the key assumptions, methods, and results. The NRC staff audit found that this information is documented, in part, in Calculation Package QDC-0085-S-2034, "Combined Events Flood Assessment at QCNGS."

Request: Provide the following portions of QDC-0085-S-2034:

- Main text (Pages 1-28)
- Attachment 1, Figures
- Attachment 3, 2-Minute Wind Speed Calculation and Formulas

**Response:**

The requested excerpts from Calculation QDC-0085-S-2034 are included Enclosure 3 DVD #2A under folder labeled "RAI 22".

## RAI 23: Combined Effects – Waves

Background: The discussion of the combined effect flood analysis in FHRR Section 3.8 states that the licensee used engineering judgment and information on topography and bathymetry of the Mississippi River bottom to determine that waves will not break at or near the QCNPS site, thus eliminating the need to calculate the wave set up.

Request: Provide the topography and bathymetry of the Mississippi River in the vicinity of QCNPS site associated with this conclusion in FHRR Section 3.8.

**Response:**

Topographic and bathymetric input data for the wind-wave calculation were derived from the Quad Cities Nuclear Power Station (QCNPS) probable maximum flood (PMF) HEC-RAS model. Two cross sections from the PMF HEC-RAS model intersect QCNPS: cross sections at river mile 506.9 and 506.0. The HEC-RAS model was initiated by importing the geometry file from the UNET hydraulic model, developed as part of the Upper Mississippi River System Flow Frequency Study by the U.S. Army Corps of Engineers (USACE). The UNET model was obtained from the Rock Island District of the USACE. The cross section information from the UNET model was converted to HEC-RAS format. The imported UNET cross section elevation data was validated through visual comparison of elevations with USGS quadrangle topographic mapping data (Reference 1) and USACE hydrographic data (Reference 2).

The requested topography and bathymetry files along with UNET input files are included in Enclosure 3 DVD #2A under folder labeled "RAI 23".

**References**

1. Quadrangle Topographic Maps 7.5 Minute Series, U.S. Geological Survey. The map images were downloaded from three sources:
   - Illinois Natural Resources Geospatial Data Clearinghouse. (http://crystal.isgs.uiuc.edu/nsdihome/webdocs/drgs/drgorder24bymap.html).
   - Iowa Natural Resources Geographic Information Systems Library. (http://www.igsb.uiowa.edu/nrgislibx/).
   - Wisconsin Department of Natural Resources Geographic Information Systems. (http://dnr.wi.gov/maps/gis/datadrg.html#data)
2. Hydrographic Surveys of the Mississippi River, USACE Rock Island District. http://www2.mvr.usace.army.mil/odrsurvey/default.cfm

**RAI 24: Combined Effects – Clarification of Scenarios**

Background: In the combined effects flood analysis (FHRR Section 8) the licensee briefly describes three alternatives with respect to floods caused by precipitation events and two alternatives with respect to floods caused by seismic dam failure and states that the alternatives are "described in detail earlier". However, it is not clear where the alternatives are described in earlier sections of the document. Further, the discussion in this section did not provide a clear rational for why the described alternatives were selected and why others were excluded.

Request: Clarify how the alternatives discussed in this section relate to scenarios, alternatives and approaches described in earlier sections of the document. Provide a clear and detailed description of how the alternatives used in the combined event analysis were selected.

**Response:**

Quad Cities Nuclear Power Station (QCNPS) is located on the eastern bank of the Mississippi River approximately 506.8 miles upstream of the confluence of the Ohio River with the Mississippi River. Topographic relief at the site is low and relatively flat, with a mean station elevation of about 595 feet, mean sea level (MSL) 1912. The site is located approximately equidistant between Lock & Dam Nos. 13 and 14, which are owned and operated by the U.S. Army Corps of Engineers (USACE).

NUREG/CR-7046 *Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America* (NUREG/CR-7046) recommends various scenarios (Appendices H.1 through H.5) for analyzing combined effect floods.

The flood hazard at QCNPS is due to the Mississippi River, therefore all alternatives from the combined events listed in NUREG/CR-7046 Appendix H.1 and H.2 are considered for QCNPS. The QCNPS is not located along an open or semi-enclosed body or enclosed body of water. Therefore, the combinations listed in NUREG/CR-7046 Appendices H.3 and H.4 are not applicable to QCNPS. Analysis of flooding along the shores of an open, semi-enclosed, or enclosed body of water was not performed. Similarly, QCNPS is not located along a coast. Therefore, the combinations listed in NUREG/CR-7046 Appendix H.5 are not applicable to QCNPS. Analysis of flooding caused by tsunamis was not performed as tsunami flooding is not an applicable hazard.

Section 1.c of the Flood Hazard Reevaluation Report (FHRR) provides the methodology, approach and results from the three alternatives (All-Season PMF, Probable Maximum Snowpack and 100-Year Cool-Season Rainfall, and 100-Year Snowpack and Cool-Season PMP) listed in NUREG/CR-7046 Appendix H.1, without hydrologic dam failure and added effects of wind-wave. As discussed in Section 1.d of the FHRR, Alternative 3 of Combination H.1 provides the controlling PMF still water surface elevation. Alternatives listed in Combination H.1 are evaluated in Calculations QDC-0085-S-1990 and QDC-0085-S-1991, without the effects of dam failure and wind-wave activity.

Hydrologic dam failure in combination with Alternative 3 of Combination H.1 provides the controlling still water surface elevation of all dam failure scenarios. Section 2 of the FHRR provides the methodology, approach and results from the hydrologic dam failure scenario. Dam

failure scenarios are evaluated in Calculation QDC-0085-S-2032, without the effects of wind-wave activity.

Section 7 of the FHRR provides the methodology, approach and results for wind-wave activity coincident with the controlling scenario, Alternative 3 of Combination H.1 with hydrologic dam failure. Wind–wave effects are evaluated in Calculation QDC-0085-S-2034.

The seismic dam failure scenarios are evaluated due to requirements in the Combined-Effect Flood evaluation discussed in NUREG/CR-7046, Appendix H.2 with the potential for flooding above site grade and shorter response time than may be available under PMF conditions. The analysis indicates that the seismically-induced dam failure scenario does not produce a flood that reaches plant grade.

The Sunny Day scenario evaluated in this dam failure analysis is conservatively assumed to correspond to "Alternative 1" of the NUREG/CR-7046, Appendix H.1 - Floods Caused by Precipitation Events. Section 2 of the FHRR provides the methodology, approach and results from these dam failure scenarios. Dam failure scenarios are evaluated in Calculation QDC-0085-S-2032, without the effects wind-wave activity.

**Enclosure 2**

**Quad Cities Nuclear Power Station, Units 1 and 2**

**DVD #1 of RS-14-173 for RAI Response No. 4**

**Regarding Fukushima Lessons Learned - Flood Hazard Reevaluation Report**

The contents of Enclosure 2 are
*Security-Related Information – Withhold Under 10 CFR 2.390*

## Enclosure 3

**Quad Cities Nuclear Power Station, Units 1 and 2**

**DVD #2A of RS-14-173 for RAI Response Nos. 11, 12, 13, 15, 17, 20, 22, and 23**
**DVD #2B of RS-14-173 for RAI Response No. 15**
**DVD #2C of RS-14-173 for RAI Response No. 15**
**DVD #2D of RS-14-173 for RAI Response No. 15**
**DVD #2E of RS-14-173 for RAI Response No. 15**
**DVD #2F of RS-14-173 for RAI Response No. 15**

**Regarding Fukushima Lessons Learned - Flood Hazard Reevaluation Report**