Appendix B

Hydrologic and Hydraulic Studies Report

This report summarizes the Hydrologic and Hydraulic Studies completed to analyze and perform preliminary design of alternatives for the reconstruction of Lake Delhi Dam on the Maquoketa River in eastern Iowa. Lake Delhi Dam was breached and failed during a flood on July 24, 2010. The dam in its pre-failure condition did not have sufficient hydraulic capacity to meet current Iowa dam safety criteria. Design of the reconstruction will include significantly increasing Lake Delhi Dam's hydraulic capacity for passing flood flows.

For the alternatives analysis several concepts were developed for reconstructing the dam's spillway(s). Three concepts were taken to preliminary design and evaluated for potential design and construction. Other hydraulic considerations included minimum/low flow passage, lake draining capacity, and cofferdam/bypass during construction. Steps to complete the hydrologic and hydraulic studies for the alternatives analysis included.

- Characterizing Maquoketa River Flows at Lake Delhi Dam
- Developing a hydrologic model of Lake Delhi Dam watershed.
- Developing a hydraulic model of Maquoketa River upstream and downstream of Lake Delhi Dam.
- Performing hazard classification and design flood analysis for Lake Delhi Dam.
- Developing Lake Delhi Dam spillway concepts.
- Addressing other hydraulic issues.

B.1 Maquoketa River Flows

The Maquoketa River is approximately 150 miles long and flows into the Mississippi River. Lake Delhi Dam is located approximately 40 miles downstream from the river's headwaters so is in the upper portion of the river's watershed. The U.S. Geological Survey (USGS) has maintained two gages near Manchester, Iowa at Highway 20 which is approximately 28 miles downstream from the river's headwaters. Gage 05417000 was discontinued in 1973 but has a record of daily flow values from 04/25/33 to 09/30/73. Gage 05416900 is currently in service and has daily flow values from 4/26/00 to the present.

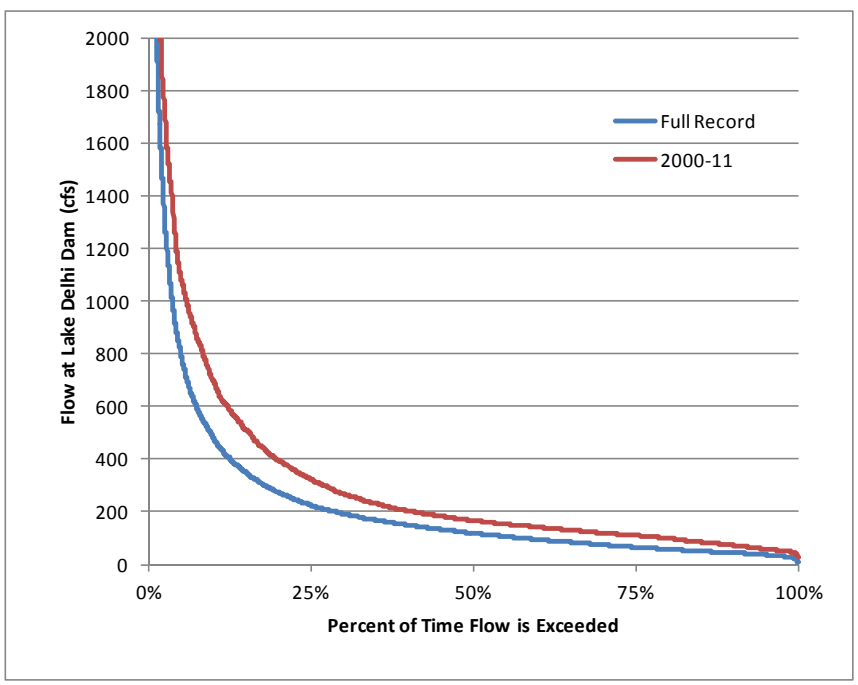
The tributary drainage areas for the Maquoketa River at Highway 20 and at Lake Delhi Dam are 300 square miles and 349 square miles respectively. This means that river flows recorded at the USGS gage are typically going to be smaller than river flows at the dam. *Techniques for Estimating Flood-Frequency Discharges for Streams in Iowa* (USGS, 2001) provides regional empirical equations for translating flows between points on Iowa rivers that have proportional areas. The multiplier for Highway 20 USGS gage flows to Lake Delhi Dam flows was computed to be 1.07. In reality, the difference in flows between the two locations will vary depending on rainfall, groundwater, and snowmelt conditions, but 1.07 provides a reasonable estimate.

Following the 2010 flood, USGS performed a frequency analysis of peak flows at the Highway 20 gages and established new return period discharge estimates. Table B-1 displays the results with Lake Delhi Dam return period flows estimated using the 1.07 multiplier.

Table B-1 Return Period Flows

Return Period (yrs)	Annual Exceedance Probability	USGS Gage Flow (cfs)	Lake Delhi Dam Flow (cfs)
1	0.95	1,393	1,491
2	0.5	4,506	4,821
5	0.2	8,636	9,241
10	0.1	12,300	13,161
25	0.04	18,130	19,399
50	0.02	23,420	25,059
100	0.01	29,610	31,683
200	0.005	36,820	39,397
500	0.002	48,150	51,521

The frequency analysis provides a characterization of peak flood flows for the Maquoketa River. These and larger flow magnitudes were used in designing the hydraulic capacity of the reconstructed Lake Delhi Dam's spillway alternatives. A flow duration analysis provides a characterization of the range of flows that are likely to occur at the dam. For the flow duration analysis the entire 51 year period of daily flow record is (1933-1943, 2000-2011) used to plot a graph showing the percent of time flow a given flow is exceeded over the period of record. The plot is shown in Figure B-1.Similar to the frequency analysis USGS gage flows were adjusted by the 1.07 multiplier.



Flow Duration Curve at Lake Delhi Dam
Figure B-1

The 50% value on the X-axis represents the average flow at the dam. In addition to the full gage record, a depiction of flows over the last 11 years was developed. As shown on the graph, Maquoketa river flows over the last 11 years have been higher relative to the gage record. The average flow for the full gage record is 118 cubic feet per second (cfs) and for the last 11 years the average flow was 167 cfs.

B.2 Hydrologic Model

The hydrologic model was used to develop a series of design flood hydrographs (i.e. analysis derived) for the Lake Delhi Dam watershed. The flood hydrographs were used as an input for the hydraulic model

The watershed area was obtained from the Iowa Department of Natural Resources (DNR) which provided an ArcGIS shapefile of the delineation of the dam's tributary watershed. This delineation was checked against USGS topography and hydrologic unit code (HUC) maps and matched very closely.

The infiltration rate was estimated using ArcGIS mapping software. Hydrologic soil group data for the watershed area was obtained from the Natural Resources Conservation Service (NRCS) Soil Survey Geographic Database. The NRCS has established typical infiltration rates for given hydrologic soil groups. As is typical for flood modeling, initial infiltration losses were ignored and only a constant infiltration rate was used.

Time of concentration was estimated by measuring the length of the Maquoketa River from the headwaters to the upstream end of Lake Delhi, estimating an appropriate river velocity and computing the travel time. A river velocity of 3feet per second was assumed which matched the velocities computed by the hydraulic model of the Maquoketa River closely.

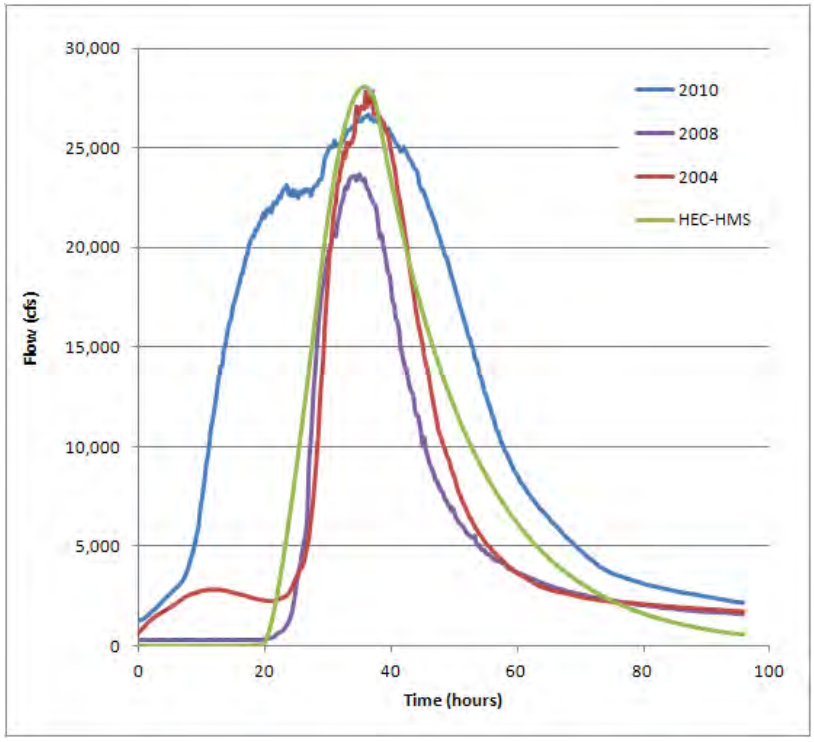
The storage coefficient was established using flood hydrographs recorded at the USGS stream gage (05416900) located at the Highway 20 Bridge near the upstream end of Lake Delhi. Flow data for the 2004, 2008, and 2010 floods were obtained from the USGS gage and plotted. The storage coefficient was computed from the slope of the descending limb of the flood hydrograph. Each recorded flood had a slightly different storage coefficient so a representative coefficient was estimated.

The rainfall events modeled in HEC-HMS were a 100-year storm and a Probable Maximum Precipitation (PMP). The 100-year/24-hour rainfall for the Lake Delhi area was obtained from *Iowa Rainfall Frequencies* (Waite, 1988). The PMP was established using National Oceanic and Atmospheric Administration's (HMR 51 and HMR 52) guidelines. A 72-hour storm duration provided the probable maximum storm, with the bulk of the rainfall falling within 6 hours on the second day.

The watershed parameters and rainfall events were input into HEC-HMS and the flood hydrographs were computed. The peak HEC-HMS derived 100-year flow was checked against the USGS gage 2004, 2008, and 2010 hydrographs as well as the 100-year flow established at the gage and the flows matched closely. The HEC-HMS derived 100-year flow used the 100-year rainfall from the DNR recommended *Iowa Precipitation Frequencies* (Waite, 1988). Similar to the flow frequency analysis which has increased return period flood flows after incorporating recent floods, the 100-year rainfall value would likely increase with an updated rainfall frequency

analysis. So the HEC-HMS 100-year hydrograph could more closely match the updated the 100-year peak flow from the USGS flood frequency analysis with an updated 100-year rainfall.

Figure B-2 displays the hydrograph comparison. USGS gage flow hydrographs were adjusted by the 1.07 multiplier. Watershed parameters, rainfall depths, and peak flood flows are provided in Table B-2.



Flood and HEC-HMS Hydrograph Comparison
Figure B-2

Table B-2 Lake Delhi Dam Watershed Parameters

Parameter	Value
Drainage Area (mi^2)	349
Infiltration (in/hr)	0.25
Time of Concentration (hrs)	18
Storage Coefficient (hrs)	15
PMF Rainfall Total (in)	25.8
PMF Peak Flow (cfs)	143,900
100-Year Rainfall Total (in)	6.4
100-Year Peak Flow (cfs)	28,100

B.3 Hydraulic Model

The starting point for the hydraulic modeling was the HEC-RAS model of the Maquoketa River developed by the DNR to evaluate the 2010 breach of Lake Delhi Dam. The upstream end of the river model is at the Highway 20 Bridge and the model extends approximately 23 miles to just downstream of Hopkinton.

HEC-RAS software was developed by the U.S. Army Corps of Engineers and uses a series of river cross-sections and structures to model a constant flow (steady) or hydrograph (unsteady) through the river reach. All hydraulic modeling performed for the alternatives analysis used unsteady flow modeling which allowed the flood and dam breach hydrographs to be routed through the reservoir, dam, and river system.

The HEC-RAS model of the Maquoketa River as well as supporting background data was provided to Stanley Consultants by the DNR. The DNR HEC-RAS model was created using HEC-GeoRAS, which is an ArcGIS interface with HEC-RAS that allows the model elements to be geographically based and provides improved flood mapping capabilities over the stand-alone HEC-RAS software. The following adjustments were made to the DNR HEC-RAS model:

- River channel topography was updated with post-breach LiDAR data obtained in fall 2010.
- HEC-RAS river cross-sections were extended up to elevations where the design floods were contained within the cross-section.
- Bridge structures were added downstream of the dam (Quarter Road, 295th Street and Hopkinton) using construction drawings provided by Delaware County.
- One inflow hydrograph was used at the upstream end of the model (DNR model used two).
- The dam was modified to reflect the proposed condition (working gates, principal/auxiliary spillway).

The DNR HEC-RAS model was correlated to the 2010 flood using both the approximate time and elevation of high water marks (peak flood levels) at the Lake Delhi Dam and several bridges on the Maquoketa River. The DNR HEC-RAS model provided a good replication of the 2010 flood event. The 2010 flood hydrograph was run through the adjusted model (with the existing dam) and the resulting flood profile did not change from the original DNR model so the adjusted model is also thought to provide a good representation of the Maquoketa River.

B.4 Hazard Classification

Hydrologic and Hydraulic analysis and design standards for dams in Iowa are specified in *Technical Bulletin 16 - Design Criteria and Guidelines for Iowa Dams* (DNR, 1990). The standards are defined according to the dam's hazard classification. The state of Iowa has three hazard classifications for dams; Low, Moderate, and High Hazard.

If hydropower is ever redeveloped at Lake Delhi Dam, the reconstructed dam will have to meet FERC criteria. FERC also has three hazard classifications; Low, Significant, and High Hazard. The FERC and DNR hazard classification definitions are very similar so the classification

determined by DNR criteria should correspond to a FERC hazard classification. Table B-3 provides the agency hazard classification definitions.

Table B-3 Hazard Classification Definitions

Hazard Class	DNR Definition	FERC Definition
Low	Structures located in areas where damages from a failure would be limited to loss of the dam, loss of livestock, damages to farm outbuildings, agricultural lands, and lesser-used roads, and where loss of human life is considered unlikely.	Structures located in rural or agricultural areas where failure may damage farm buildings, limited agricultural land, or township and country roads. Low hazard potential dams have a small storage capacity, the release of which would be confined to the river channel in the event of a failure and therefore would represent no danger to human life.
Moderate/ Significant	Structures located in areas where failure may damage isolated homes or cabins, industrial or commercial buildings, moderately traveled roads or railroads, interrupt major utility services, but without substantial risk of loss of human life.	Structures located in predominately rural or agricultural areas where failure may damage isolated homes, secondary highways or minor railroads; cause interruption of use or service of relatively important public utilities; or cause some incremental flooding of structures with possible danger to human life.
High	Structures located in areas where failure may create a serious threat of loss of human life or result in serious damage to residential, industrial or commercial areas, important public utilities, public buildings, or major transportation facilities.	Structures located where failure may cause serious damage to homes, agricultural, industrial and commercial facilities, important public utilities, main highways, or railroads, and there would be danger to human life.

The hazard classification of Lake Delhi Dam controls several design parameters including the freeboard design flood. For detailed design to proceed, a hazard classification is needed to establish the applicable dam safety and design criteria.

Previous inspections and analyses have identified Lake Delhi Dam as a low, moderate, and high hazard structure but there has not been a detailed analysis of potential downstream hazard to substantiate the hazard classification. The hazard classification analysis performed for this study provides a more thorough evaluation of risk associated with theoretical dam failure through inundation mapping of a series of flood events with and without dam failure.

The hazard classification analysis consisted of the following steps:

- HEC-HMS generated flood hydrographs (events) for the Lake Delhi Dam were input into the HEC-RAS model of the Maquoketa River.
- The Lake Delhi Dam in the HEC-RAS model was adjusted to reflect the reconstructed condition.
- Dam breach (failure) parameters were developed for Lake Delhi Dam.
- The HEC-RAS model was run under no breach and breach scenarios for a series of HEC-HMS generated flood events.
- Flood profiles (maximum water surface) from the HEC-RAS model were compared and evaluated for the various flood/breach scenarios.
- Flood profiles were exported to ArcGIS and inundation maps created.
- Impacted structures and roadways were tabulated and compared for the no breach and breach scenarios.
- A hazard classification for Lake Delhi Dam was recommended.

HEC-HMS Hydrographs

HEC-HMS derived flood hydrographs were input into the HEC-RAS model. Initially, the full PMF, ½ PMF, and 100-year flood were modeled in HEC-RAS with and without a dam breach. Subsequent flood events modeled included the $^{1}/_{3}$, $^{2}/_{3}$ and ¾ PMF as well as a "sunny day" event which is a dam failure that occurs during normal flow conditions.

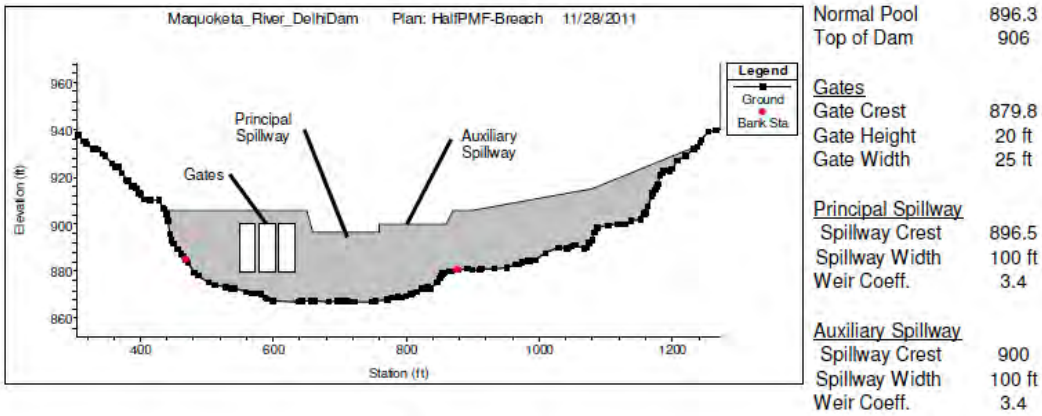
Reconstructed Dam

Discharge from Lake Delhi Dam in the pre-breach condition was provided by two wicket gates and three 25-foot wide by 17-foot high lift gates. When the hydropower facility was in operation, the wicket gates were used to control flow through the turbines. Hydropower generation at the dam was deactivated in 1973, but the wicket gates continued to be used for passing normal flows and maintaining pool elevation. The hydraulic capacity of the two wicket gates was estimated to be roughly 600 cubic feet per second (cfs). The lift gates were used for passing flows that exceeded the capacity of the wicket gates. The gates were difficult to open and close and one of the gate guides cracked during attempted operation during the 2010 flood, contributing to failure of the dam.

The reconstructed dam will have working gates and a new principal/auxiliary spillway to increase hydraulic capacity. The future of hydropower generation and use of the wicket gates is uncertain so wicket gate discharge was excluded from the hydraulic analysis. Design of the Lake Delhi Dam reconstruction is in the preliminary stage, so the "reconstructed" dam in the HEC-RAS model represents an approximation.

Gates will be replaced as part of the reconstruction so they were assumed to be fully operable in the HEC-RAS model. Prior to the breach fabrication plans had been developed for providing a new gate lifting mechanism that would have allowed the gates to be lifted 20-feet above the spillway crest so the gate openings in HEC-RAS were adjusted to reflect a 25-foot wide by 20-foot high opening.

A principal/auxiliary spillway will be added to the dam as part of the reconstruction. The exact dimensions of the spillway have not been established yet. What is known is that the spillway will fit within the roughly 230 foot long southern embankment, the principal spillway crest would likely be at normal pool, and the auxiliary spillway crest would be a few feet above normal pool. To reflect the reconstructed condition a 100-foot long principal spillway with a crest elevation of 896.5 ft-msl and a 100-foot long auxiliary spillway with a crest elevation of 900 ft-msl were added to the HEC-RAS model. The HEC-RAS spillway concept is shown in Figure B-3.



HEC-RAS Lake Delhi Dam Spillway Concept
Figure B-3

Hazard classification is focused more on the downstream impact of the dam than the specifics of the spillway so using a principal/auxiliary spillway approximation is reasonable for this analysis. The objective is to represent the influence of the spillways on the dam failure and downstream flooding. In this case, the principal and auxiliary spillways increase the amount of flow in the downstream channel for a given Lake Delhi pool elevation and they influence the geometry of the theoretical breach because the spillways will likely be armored (concrete). The various spillway options currently being considered have a similar embankment shape so the proposed HEC-RAS model should provide an adequate depiction of the failure condition no matter which alternative is chosen. However, the analysis will be updated once the reconstruction design is established, but a significant change in results is not expected.

Dam Breach Parameters

The DNR established dam breach parameters for their original HEC-RAS model based upon the Lake Delhi Dam failure of the southern embankment observed in 2010. For the reconstructed dam analysis, the width of the dam breach was reduced from 250 feet to 175 feet to better reflect the reconstructed condition on the southern embankment. The breach formation time was left at 1.5 hours. The failure was set to initiate at the peak of the flood hydrograph which yields the highest flood elevation (i.e. worst-case condition).

The DNR established breach parameters were checked against breach parameters estimated using three empirical methods. The three empirical methods were:

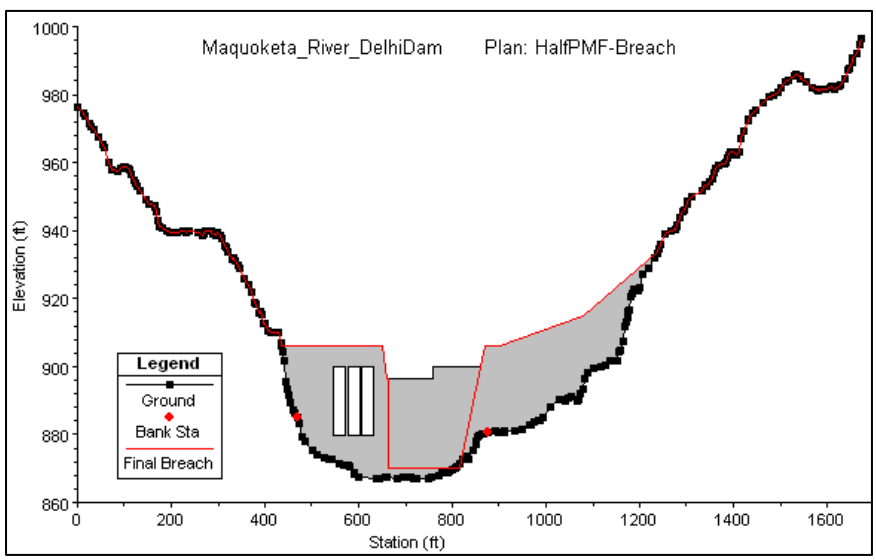
- MacDonald and Langridge-Monopolis – Uses volume of reservoir and height of dam to compute breach parameters.
- FERC – Uses type of dam and height of dam to estimate range of breach parameters.
- Froehlich – Uses type of failure, volume of reservoir and height of dam to compute breach parameters.

The results of the breach parameter computations are shown in Table B-4.

Table B-4 Lake Delhi Breach Parameters

	Method			
Breach Parameter	**M&L-M**	**FERC**	**Froelich**	**Used in HEC-RAS**
Volume Eroded (ft^3)	66781			
Breach Width (ft)	343.4	120	235	175
Side Slopes (h/v)	2	0.5	1	0.1, 1.5
Time to Fail (hrs)	0.9	0.75	1.8	1.5

The empirical computations provided a range of potential breach values. The values used in the HEC-RAS model fell within the range of empirical predictions and were similar to the breach that occurred in 2010 so were considered acceptable. Figure B-4 shows the breach geometry.



HEC-RAS Lake Delhi Dam Breach Limits
Figure B-4

Failure of the existing powerhouse and gated spillways were also considered. The two most likely failure scenarios for these large concrete structures would be tipping/sliding of the structure or undermining of the foundation leading to collapse. Both of these failure scenarios were modeled in HEC-RAS for each structure. The failure time was reduced to 0.5 hours and the geometry of the failure was set to match the extents of the structure being analyzed. Out of the four failure scenarios, tipping/sliding of the gated spillway provided the largest flood wave downstream, but it was not as large as the flood wave created by the failure of the reconstructed southern embankment, so the embankment failure was used as the failure condition for the hazard classification.

HEC-RAS Modeling

Flood events were modeled in HEC-RAS for both failure and non-failure conditions. The dam was assumed to be operating under normal conditions (normal pool of 896.3 ft-msl) prior to the flood. HEC-RAS generates stage (elevation of water surface) and flow hydrographs at each cross-section location in the model. The maximum stages at each cross-section are linked to develop a continuous flood profile of the river segment which is used in creating of inundation (flood) extents along the river channel and surrounding area for a given flood event.

The flood profiles generated by HEC-RAS indicated that the Quarter Road Bridge causes a significant backwater effect for the full and ½ PMF. To analyze the impact of a potential failure of the bridge on flood conditions a HEC-RAS model was created with the Quarter Road Bridge structure removed. Flood scenarios were compared in HEC-RAS with and without the Quarter Road Bridge. Results indicate that Quarter Road Bridge raises flood elevations upstream of the bridge by up to 0.8 feet. Removal of the bridge has minimal impact on the flood elevation or travel time of the floodwave downstream, so this suggests the bridge should not be a significant factor in downstream hazard potential.

Upstream and downstream impacts to flows and water surface elevation were also evaluated. This was analyzed by comparing HEC-RAS models of the pre-breach and reconstructed dam for a series of floods. The HEC-RAS model results show that increasing the dam's hydraulic capacity will reduce upstream pool elevations during a flood event with minimal impact to downstream flood elevations. The analysis will be revisited during detailed design to verify the reconstructed dam will not adversely impact upstream or downstream properties compared to the pre-breach condition.

Inundation Maps

The HEC-RAS flood profiles were exported to ArcGIS using HEC-GeoRAS which uses the profiles to develop inundation extents for each flood/failure event. Inundation maps were created that include geo-referenced aerial imagery so the inundation limits can be viewed relative to downstream buildings and infrastructure. Inundation maps are provided in this Appendix.

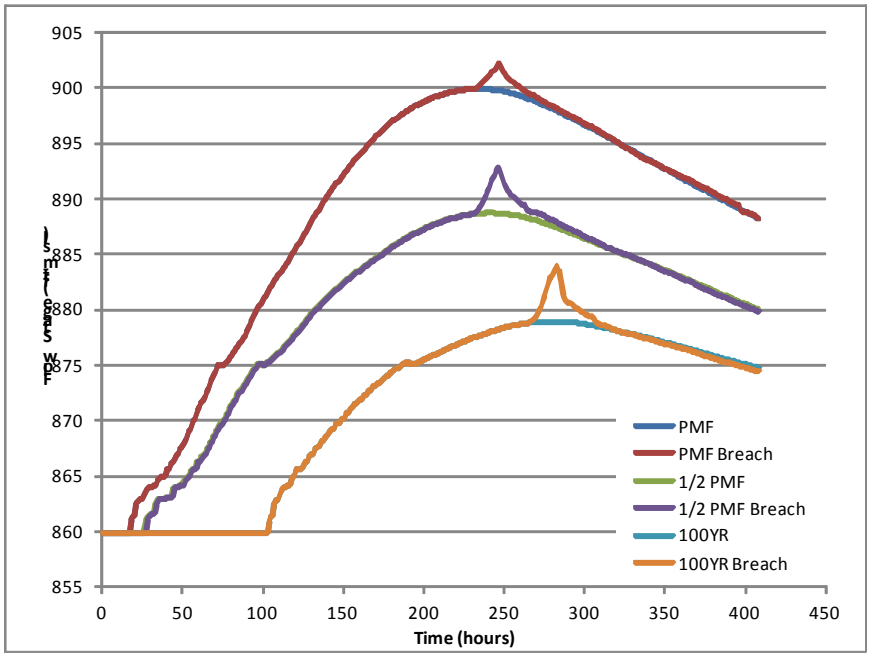
Tabulation of Impacted Structures

The inundation limits were compared for failure and non-failure conditions for the full PMF, ½ PMF, 100-year flood, and sunny day (dam failure during normal flow condition) and buildings and infrastructure inside the inundation limits were tabulated between each set of HEC-RAS cross-sections. The detailed inundation table is provided in this Appendix. A summary of the number of structures impacted is provided in Table B-5.

Table B-5 Impacted Structure Summary

Event	Scenario	Residential	Comm/Ag	Bridges	Roads
PMF	No Breach	104	30	3	12
	Breach	107	30	3	12
Half PMF	No Breach	27	8	3	8
	Breach	29	8	3	8
100-YR	No Breach	3	1	1	5
	Breach	5	2	1	5
Sunny Day	Breach	0	0	0	1

Due to their location and lack of potential warning time, the homes directly downstream of the dam were examined in closer detail for the theoretical failure event. Because of they are located so close to the dam, the nearby downstream properties see the greatest potential increase in flood level due to dam failure. The critical factors for hazard potential are the number of additional properties impacted and the increase in flood level due to breach. The ½ PMF appears to provide the greatest increase in hazard potential from breach due to the fact it raises flood levels by over 4 feet just downstream of the dam. The full PMF inundates more homes but its relative increase in flood level due to breach is 2 feet less than the ½ PMF. The 100-year flood has a greater rise (5 feet) but it impacts fewer buildings and roadways. Exhibits and tables providing an inventory of individual downstream properties impacted are provided in this Appendix. A hydrograph plot comparing the increase in stage (water surface) elevation just downstream of the dam for breach and no breach scenarios is provided in Figure B-5.



HEC-RAS Flood Stage Just Downstream of Lake Delhi Dam
Figure B-5

Hazard Classification

Hazard classification is based on the potential consequence of dam failure. When analyzing the consequences of dam failure during a flood event it is the increase in consequence (i.e. increase in damage and potential loss of life) due to failure that is evaluated. Inundation maps communicate the extent of downstream area that could be impacted by the given flood with and without dam failure.

Results of the HEC-RAS modeling and inundation mapping indicate that dam failure during flood events does not appear to cause a significant increase in the number of structures inundated. The majority of additional structures that are inundated by a failure event are the homes within 1500 feet downstream of the dam. As the Emergency Action Plan is developed for the reconstructed condition it will be important to have well-defined communication and evacuation procedures defined for these residents.

Hazard classification was reviewed for both the DNR and Federal Energy Regulatory Commission (FERC) definitions. Lake Delhi Dam appears to fit the Moderate (DNR), Significant (FERC) Hazard Classification. The reasoning is as follows:

- HEC-RAS modeling and inundation mapping show that a potential failure during a flood would only cause a small increase in the number of structures impacted.
- A potential sunny day failure conditions stays within the limits of the 100-year floodplain (typically non-developed area) so the potential for damage is less than if sunny day failure flooded more habitable or developable lands.

- Much of the area downstream of Lake Delhi Dam is rural and agricultural. Although future development is possible, most development would likely occur closer to the town of Delhi, which is up above the river channel or in Hopkinton which is far enough downstream that the increase in flood elevation due to failure is roughly 1 foot.
- The Maquoketa River downstream of Lake Delhi Dam is widely used for canoeing and fishing activities, however the river does not contain the type of attractions that bring large numbers of people into the river channel area for extended periods of time (i.e. restaurants, resorts, large campgrounds or trailer parks, etc.)
- Therefore, the DNR definition of Moderate hazard where "…failure may damage isolated homes or cabins, industrial or commercial buildings, moderately traveled roads or railroads, interrupt major utility services, but without substantial risk of loss of human life." is appropriate for the reconstructed Lake Delhi Dam.
- The FERC definition of Significant hazard for "Structures located in predominately rural or agricultural areas where failure may damage isolated homes, secondary highways, or minor railroads; cause interruption of use or service of relatively important public utilities; or cause some incremental flooding of structures with possible danger to human life." Also seems the appropriate classification for Lake Delhi Dam.

Design Flood

Per *Technical Bulletin 16 - Design Criteria and Guidelines for Iowa Dams* (DNR, 1990), a moderate hazard classification establishes the freeboard design flood as the ½ PMF. FERC uses an incremental analysis to establish the design flood by determining the largest food where failure causes an increase in downstream hazard. The FERC method was analyzed by adding the ¾ PMF, the $^{2}/_{3}$ PMF, and $^{1}/_{3}$ PMF scenarios to the HEC-RAS model.

FERC recommends using a two-foot increase in flood elevation due to failure as the minimum threshold where hazard potential is increased by dam failure. For all flood events at Lake Delhi Dam, the increase in flood elevation due to failure is greater than two feet just downstream of the dam, but then decreases to less than two feet downstream of the Quarter Road Bridge (located roughly 4 miles downstream of the dam). Besides the immediate homes at the dam, there are no buildings inundated by any of the flood events until the flood is past the Quarter Road Bridge.

The flood that has the greatest overall increase flood elevation due to failure is the 100-year flood. The $^{1}/_{3}$ PMF and ½ PMF have a comparable rise in flood elevation due to failure relative to the 100-year flood. For floods greater than the ½ PMF (Full PMF, ¾ PMF and $^{2}/_{3}$ PMF) the increase in flood elevation due to failure is discernibly less than the floods greater than ½ PMF, $^{1}/_{3}$ PMF and the 100 Year Flood. So given that the greatest increases in flood elevation due to failure are from floods of a lesser magnitude than the ½ PMF, using the DNR designated ½ PMF as the freeboard design flood should also meet FERC criteria.

Recommendation for Final Design

Based on the analysis Stanley Consultants recommends that design of the Lake Delhi Dam reconstruction proceed with a classification as a Moderate Hazard structure and a freeboard design flood of the ½ PMF. This classification will be verified with an updated analysis once reconstruction design has been established.

A detailed summary of the computations performed for the hazard classification is provided with this Appendix.

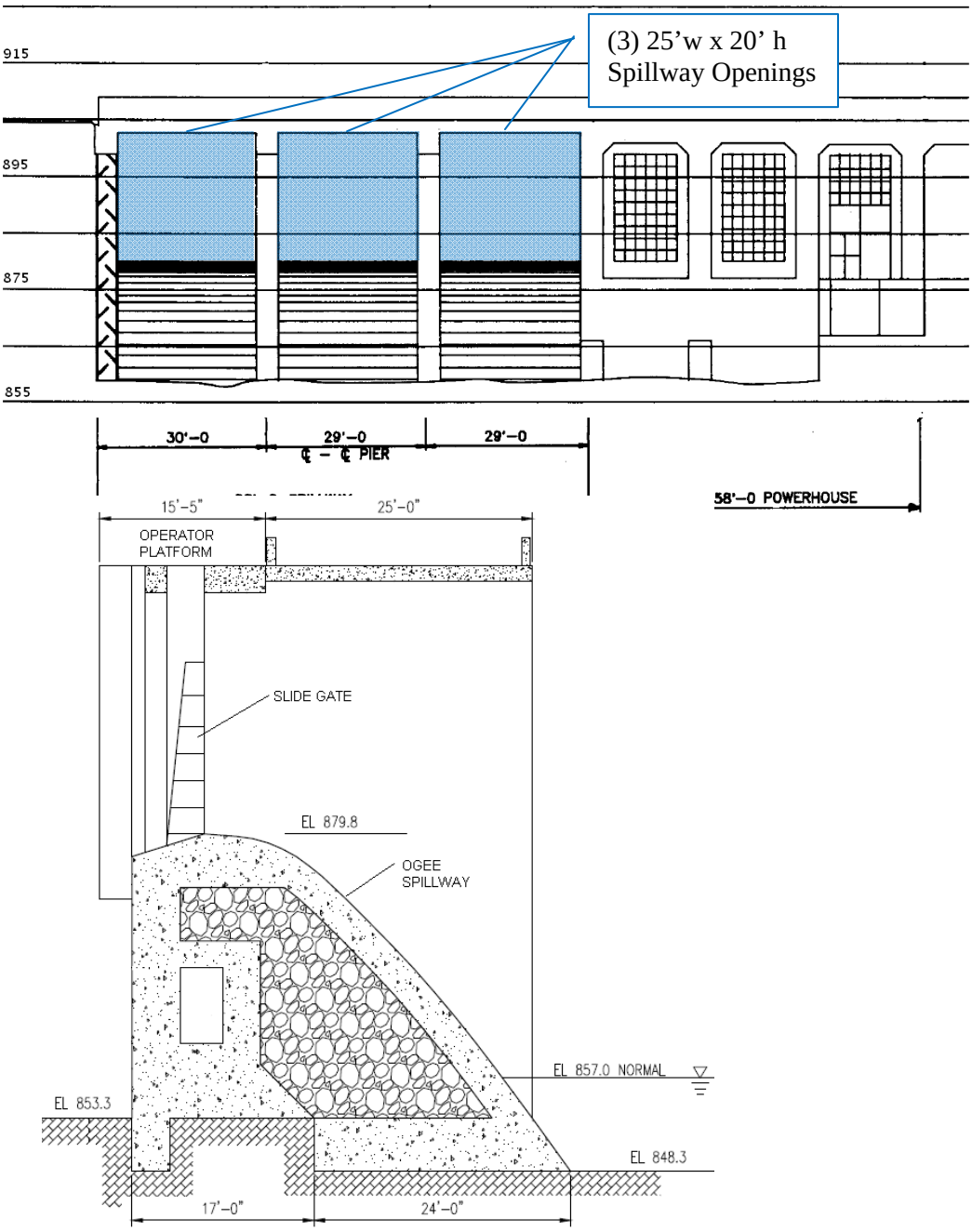
B.5 Spillway Concepts

Using the ½ PMF as the design flood, spillway concepts were developed with the objective of the reconstructed Lake Delhi Dam being able to pass the ½ PMF without overtopping the existing powerhouse/gated spillway structure.

Prior to the breach, river flows exceeding 600 cfs were passed by opening the three spillway gates located adjacent to the powerhouse. If working properly, the gates could raise roughly 17 feet, which provided Lake Delhi Dam with a maximum discharge capacity of approximately 28,000 cfs prior to the dam overtopping. The peak flow of the ½ PMF for the Lake Delhi Dam watershed is close to 72,000 cfs. Using HEC-RAS to route the flood through the reservoir shows that to pass a ½ PMF without overtopping the powerhouse the reconstructed Lake Delhi Dam will need to pass roughly 69,000 cfs through its spillway(s) which is more than double the hydraulic capacity of the pre-breach dam.

Spillway Gates

The existing Lake Delhi Dam spillway is located next to the powerhouse structure. It consists of an ogee spillway with a crest elevation of 879.8 ft-msl. Above the ogee spillway are three 25-foot wide by 20-foot high openings. Flow through the openings was controlled by three lift gates, which were hoisted from an operator platform above the gates. An elevation and section view of the gates are shown in Figure B-6.



Spillway Gates
Figure B-6

Several gate options were considered for the dam reconstruction; however, the pier and bridge configuration above the spillway is not conducive to different gate systems. The options considered are shown in Table B-6.

Table B-6 Spillway Gate System Comparison

Option	Suitable	Explanation
Radial Gates	No	Radial gates are mounted on an arm and are lifted by rotating the arm upwards so have a circular motion. Installing radial gates at the existing spillway would require removing a significant portion of the bridge deck.
Crest Gate	No	Crest gates are mounted to the crest of the spillway and when lowered are flush with the crest. The ogee spillway at Lake Delhi Dam is steep and does not have a wide crest, so installation of crest gates would require removal of large portion of the crest to create a platform for mounting the gates.
Lift Gate	Yes	The existing spillway used lift gates so the configuration is suitable for lift gate installation. The gate guides were damaged and need replacement but that repair would be minor compared to the work required to install other gate systems.

Prior to the 2010 dam failure a project was underway to replace the lift gate hoisting mechanism. The hoisting equipment was received by the dam operator but never installed at the dam so could be installed as part of the reconstruction project. The new hoisting equipment should eliminate previous issues experienced with lifting gates and provides an additional 3 feet of lifting height, so the new gate openings will be 25 feet wide by 20 feet high when the gates are fully lifted.

The new lift gate system will have a hydraulic capacity of roughly 30,000 cfs (40% of design flood) with the gates fully raised and the upstream pool at the top of dam. The wicket gates located on the upstream side of the powerhouse could provide an additional 600 cfs of capacity but their future use is uncertain so they were not included in the spillway analysis.

Potential Spillway Options

The new spillway system at Lake Delhi Dam will need to provide roughly 39,000 cfs of additional hydraulic capacity for the dam to pass the design flood of ½ PMF without overtopping the powerhouse or spillway gate structure. There is roughly 230 feet between the buttress wall at the southern end of the existing powerhouse/spillway structure and the southern riverbank where the new dam will tie into existing ground. With this length, a straight, fixed crest at the normal pool elevation of 899.6 ft-msl could pass approximately 13,500 cfs prior to the powerhouse/spillway structure being overtopped. This is less than half of the hydraulic capacity needed so a more hydraulically effective spillway discharge system will be needed at Lake Delhi Dam. Several spillway systems were reviewed for the alternatives analysis. A summary is provided in Table B-7.

Table B-7 Spillway Option Comparison

Option	Suitable	Explanation
Fuse Plug	No	A fuse plug spillway consists of an earthen embankment overlaying a concrete spillway set several feet below the top of embankment. When the pool reaches the top embankment the earth is eroded away, exposing the concrete spillway. At Lake Delhi Dam, the concrete spillway could not be set low enough to provide sufficient hydraulic capacity.
Additional Lift Gates	No	Additional lift gates would require construction of a new section of a tall concrete ogee spillway structure to essentially extend the existing spillway structure. However, bedrock drops away in this area so in addition to the additional cost of purchasing gates and hoisting equipment, the new concrete ogee spillway and operating platform would be founded on sand which would require expensive stability enhancements to make construction viable.
Pipes Through Embankment	No	In addition to concerns over seepage and maintenance, installing pipes through the dam embankment would not provide sufficient capacity and would require construction of a new intake and operating structure.
Labyrinth Weir	Yes	A labyrinth weir consists of a sharp-crested (vertical wall) in a zigzag pattern that allows a much longer crest length to fit within a shorter length of embankment. The longer crest length significantly increases the hydraulic capacity over a straight weir section. A labyrinth weir is a viable option for meeting hydraulic capacity requirements.
Pneumatic Crest Gates	Yes	Pneumatic crest gates would be installed on top of a new concrete spillway. They would consist of slightly curved, bottom mounted gate panels that could be lowered to be flush with the crest of the new spillway. In their raised position they would be at or just above the normal pool elevation of 896.3 ft-msl, but when lowered could provide an additional 5 to 10 feet of depth for discharging flood magnitude flows. Pneumatic crest gates are also a viable option for meeting hydraulic capacity requirements.

Spillway Alternatives

From the initial review of spillway options, three spillway alternatives were developed for preliminary design and comparison. The three spillway alternatives are:

- **Dual Labyrinth Spillway** – consisting of a lower principal labyrinth weir spillway set at the normal pool to discharge normal flows and a higher auxiliary labyrinth weir spillway set several feet above normal pool to discharge the required flood magnitude flows.
- **Single Labyrinth Spillway** – consisting of a single labyrinth spillway set a normal pool to discharge normal flows but with sufficient hydraulic capacity to also discharge the required flood magnitude flows.
- **Pneumatic Gate Spillway** – consisting of a pneumatic gate system set at normal pool when raised to discharge normal flows and when lowered provides sufficient hydraulic capacity to discharge the required flood magnitude flows

Exhibits showing plans and sections of the spillway alternatives are provided in Appendix F. All spillway alternatives consist of a concrete spillway slab and chute constructed over an earthen embankment with a concrete stilling basin at the end. All spillway alternatives were sized so with the three existing lift gates and the new spillway, the reconstructed dam could pass the ½ PMF without overtopping the powerhouse/spillway structure.

Spillway Hydraulics

In performing preliminary design of the three spillway alternatives, analysis of spillway hydraulics was used to develop stage-discharge curves for each alternative.

Labyrinth weir hydraulics has been studied in detail so it is possible to predict the discharge rating for a given geometry with reasonable accuracy. *Hydraulic Design of Labyrinth Weirs* (Falvey, 2003) was utilized for developing the geometry and estimating the discharge capacity of the labyrinth weir alternatives. Empirical equations have been developed that predict flow for a given weir geometry and depth of flow going over the weir.

The major factors in the hydraulic design of the labyrinth weir are:

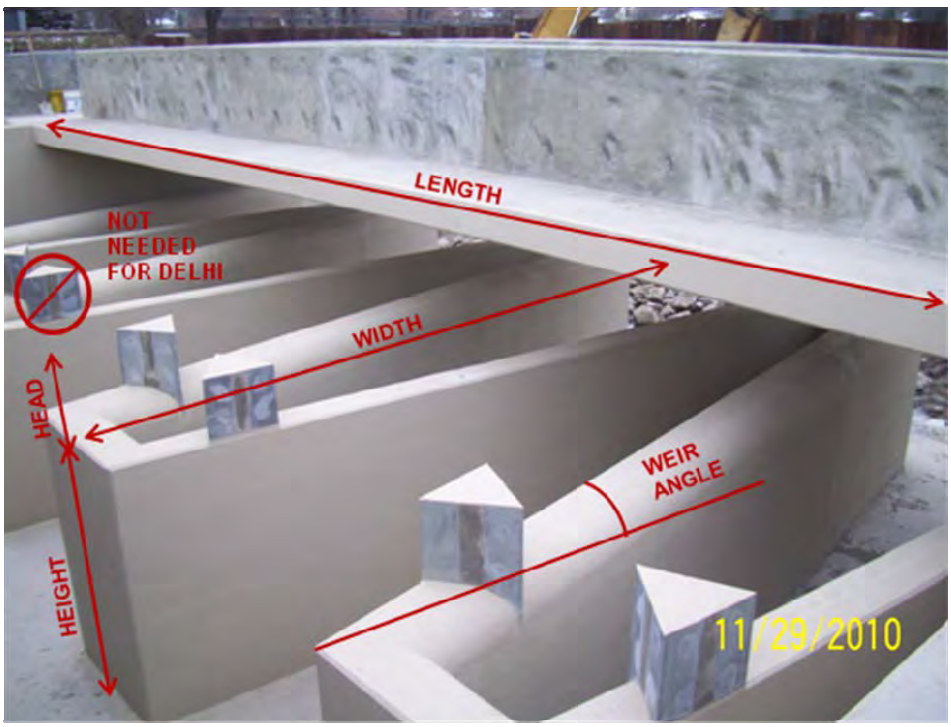
- **Design Head** – the depth of flow going over the labyrinth weir for the design flood. This is the maximum depth of flow the weir would be designed to pass.
- **Weir Height** – the height of the labyrinth weir wall. The higher the design head, the higher the weir height needs to be in order to maintain an effective discharge coefficient from normal flows to the design flood flow.
- **Weir Angle** – the angle of the long-section of the labyrinth weir wall. The overall shape of a labyrinth weir is a series of trapezoids. The weir angle controls how narrow or wide the shape of the trapezoid is. The narrower the shape, the more weir length (i.e. more trapezoids) that can be fit within a given area, but with increasing flow depths the closeness of the weir walls causes interference with flow going over the weir walls so reduces the effectiveness (i.e. reduces discharge coefficient) for higher flows. The wider the shape, the less weir length (i.e. fewer trapezoids) that can be fit within a given area, but there is less interference with increasing flow depths so the

larger weir angle geometries can provide more effective discharge over a larger range of flow depths.

- **Spillway Length** – the length available for the labyrinth weir. The length determines the number of weir cycles (number of trapezoids) that can be fit within the spillway.
- **Weir Cycle Width** – the width available for the labyrinth weir. The width determines how long the angled section of trapezoid can be. The greater the width, the more labyrinth weir that can be fit within the spillway length.

Underlying the labyrinth weir is a concrete spillway slab that provides the weir wall foundation as well as hard surface for flow falling off the weir. The preliminary design exhibits show a flat slab but in reality the labyrinth weir slab will have a slight grade (~2–3 percent) to move flow downstream more effectively.

A photo of a labyrinth weir that was designed by Stanley Consultants and recently constructed as the principal spillway of a dam in central Minnesota is provided in Figure B-7.



Labyrinth Weir
Figure B-7

Pneumatic gates essentially act as a sharp crested weir with an adjustable crest. When flows are low, the crest is kept at or near the normal pool and as flows increased the gate panels are lowered until they are flush with the fixed concrete slab/crest they are mounted to. Typically the gates are operated by a control panel which self-adjusts according to maintenance of a constant normal pool level. For preliminary design, the controlling factor is passage of the design flood, so gates are assumed to be down with the weir crest elevation essentially at the

fixed concrete slab/crest. During detailed design the concrete slab/crest will be shaped to provide an efficient shape for discharging high flows when gates are down.

A photo of a pneumatic gate system that was designed by Stanley Consultants and recently installed as the principal spillway of a dam in north-central Iowa is provided in Figure B-8. Note these gates are air bladder controlled; pneumatic gates can also be mechanically controlled.

Pneumatic Gates
Figure B-8

Energy Dissipation

The spillway chute for all options was assumed to be sloped at 3:1, horizontal:vertical. The top of the chute for all alternatives is between elevations 886 ft-msl and 892 ft-msl. The channel elevation downstream of the dam is at approximately 860, so the chute drops roughly 30 feet vertically over 90 feet horizontally. Flow down the concrete chute will be supercritical (i.e. high velocity) so energy dissipation will be needed at the downstream end, likely in the form of a standard United States Bureau of Reclamation (USBR) standard stilling basin.

The energy dissipation will be most critical at lower magnitude flood flows (i.e. 5-year, 10-year, etc.). At high magnitude flood flows (i.e. 100-year, ½ PMF) the tailwater at the downstream end will be high enough to totally submerge any high velocity scouring flows and preventing them from causing downstream damage. At normal flows, the depth of flow down the chute will be shallow enough that it will not have sufficient power to cause scour of the downstream channel. With low magnitude flood flows the tailwater is low enough that

the high velocity flows are not submerged prior to entering the downstream channel so a hydraulic jump must be created to dissipate the energy (i.e. slow down) the high velocity flows at the bottom of the chute. Similar to labyrinth weirs, stilling basins have been studied in enough detail so they can be designed using flow parameters and do not require building a physical model For the three spillway alternatives a rough design of a USBR Type III basin was laid out. A 40-foot long basin provided sufficient energy dissipation for a range of low magnitude flood flows. The stilling basin will be analyzed further and refined during detailed design.

Note that the auxiliary spillway portion of the Dual Labyrinth Spillway option does not have a stilling basin. This is because the auxiliary spillway would only discharge flows above the 100-year flood so a stilling basin is not needed at the downstream end. A large riprap apron will be sufficient.

Safety is always a concern at a dam. The hydraulics of supercritical flow, gate discharge and energy dissipation can cause rollers, eddies and vortices in the immediate downstream channel area that can be dangerous for recreational users of the downstream waterway. Appropriate warning signage and access control will be needed downstream of the dam. The overall safety of the downstream area will be a major factor in the detailed design of energy dissipation at the dam.

Cost and Structural Considerations

Several factors were taken into consideration in the hydraulic design of the spillway alternatives. The ultimate controlling factor is passage of the ½ PMF design flood, but items impacting cost, structural stability and constructability were also evaluated.

The geometry of the labyrinth weir and pneumatic gate spillways were not just controlled by hydraulics but structural issues as well. Labyrinth weir and gate heights were kept between 8 and 10 feet. A higher weir/gate height could provide more effective discharge, however when the wall or gate starts exceeding 10 feet, the additional structural and foundation requirements to make the overall structure stable start increasing to the point that making the spillway structure longer (i.e. more embankment length) is more cost-effective and constructable than trying to achieve a higher weir/gate.

A similar issue influences the steepness of the spillway chute. The steepness of the chute is controlled by the stability of the underlying earthen embankment. Hydraulically, a steeper chute could be used for the new spillway. However, the soil and stability parameters of the embankment and foundation are not suitable for increasing the steepness of the embankment.

Dual Labyrinth Weir Spillway

Many dams have both a principal and auxiliary spillway. The principal spillway is designed for continuous use in passing normal flows and then the auxiliary spillway is designed for infrequent use in passing high magnitude flood flows. Because the auxiliary spillway is used infrequently, typically cheaper materials that are stable and safe for occasional but not frequent use can be used to construct portions of the spillway. Theoretically, this provides a cost savings in spillway construction. For the Dual Labyrinth Spillway option a principal labyrinth spillway would be used to discharge normal flows, used in tandem with the lift

gates to discharge higher flows, and then the auxiliary spillway would engage at flood magnitude flows.

The Dual Labyrinth Spillway consists of a 120-foot long primary spillway labyrinth weir set at the normal pool elevation of 896.3 ft-msl and a 110-foot long auxiliary spillway labyrinth weir set at an elevation of 900ft-msl. The primary spillway has two labyrinth cycles with a weir wall height of 10 feet, a width of 60 feet and a weir angle of 25 degrees. The auxiliary spillway has four labyrinth cycles with a weir wall height of 8 feet, a width of 40 feet and a weir angle of 15 degrees.

The primary spillway discharges to a concrete chute with a concrete stilling basin at the toe. Training walls were kept straight for the preliminary design but could potentially converge slightly to save a small amount of concrete. The primary spillway weir was kept at a wide angle because of the large depth of flow (head) during the ½ PMF design flood.

DNR design criteria require that at minimum the principal spillway be able to discharge the 50-yr flood (~24,000 cfs) without engaging the auxiliary spillway. Combined with the spillway lift gates, the primary labyrinth weir spillway can discharge roughly the 100-yr flood (~30,000 cfs). This would mean that the size of the principal labyrinth weir spillway could potentially be reduced so the combined gates and principal discharge the 50-yr flood and then the auxiliary spillway engages at flows exceeding the 50-yr flood. However it was determined during design the because the auxiliary spillway crest sits at a higher elevation than the principal spillway crest the auxiliary spillway would have to be upsized more than the principal could be downsized because the principal spillway can discharge more flow due to its lower crest. So the ½ PMF is controlling the design of both the principal spillway and auxiliary spillway.

The auxiliary spillway discharges to either a roller compacted concrete or articulated concrete block chute. These are cheaper surfacing than a concrete chute but are not meant to have continuous or frequent discharge over them. This is an additional reason for keeping a larger principal spillway because it would reduce the potential frequency of use. In the past three years a 50-year auxiliary spillway would have been used three times with the 2004, 2008, and 2010 floods whereas a 100-year auxiliary spillway would likely not have been used. The auxiliary labyrinth weir has a smaller weir angle because the design head is less than the principal spillway. The width of the auxiliary labyrinth weir is also shorter because its slab is at a higher elevation on the embankment so there is less width available.

Concrete training walls will be provided between the principal and auxiliary spillways and on the southern edge of the auxiliary spillway to keep flow contained within the spillway chute.

Single Labyrinth Weir Spillway

With the ½ PMF being the controlling flood, the lower the weir crest elevation, the more flow that can be discharged prior to the upstream pool reaching the top of dam elevation of 906 ft-msl. Using a single labyrinth weir set at the normal pool elevation of 896.3 ft-msl allows a greater length of weir to be at the normal pool elevation so saves on the overall length of spillway required to discharge the ½ PMF.

The Single Labyrinth Spillway consists of a 180-foot long labyrinth weir set at the normal pool elevation of 896.3 ft-msl. The primary spillway has five labyrinth cycles with a weir wall height of 10 feet, a width of 45 feet and a weir angle of 18 degrees. The entire spillway uses a concrete chute and stilling basin.

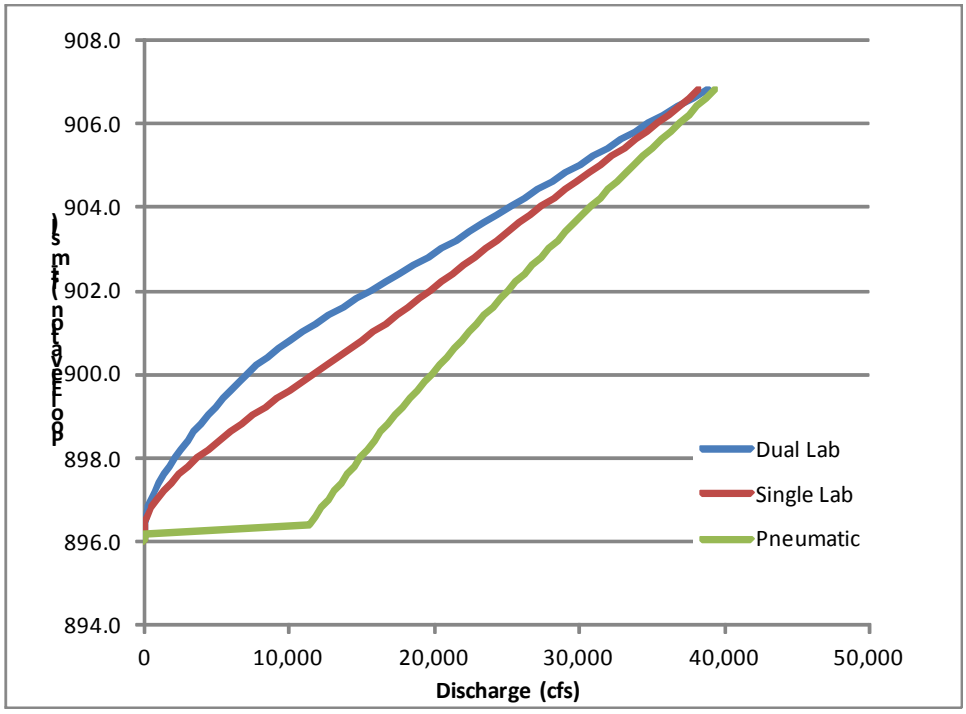
For preliminary design the spillway crest was set at a single elevation. For normal operating conditions a better discharge scenario will likely be to provide a weir segment or series of notches a few inches lower than the rest of the weir crest. This will allow the discharge to be more concentrated rather than a thin film of water going over the entire crest and will help maintain the pool at a more constant elevation. This will be analyzed further and refined in final design. This adjustment will not impact the overall hydraulic capacity of the weir for passing flood flows.

Pneumatic Gate Spillway

Similar to the reasoning for developing the single labyrinth weir option, the pneumatic gates provide ½ PMF discharge capacity by essentially lowering the weir crest below the normal pool elevation during flood flows. Because the pneumatic gates can be lowered they provide an even greater flow depth for discharging floods over the spillway prior to the upstream pool reaching the top of dam.

The range of pneumatic gate settings was set to be from normal pool (896.3 ft-msl) down to 888.3 ft-msl which would be flush with the fixed concrete crest of the spillway. An electronic control system would regulate gate settings for normal flow, maintaining a constant pool elevation of 896.3 ft-msl. The length of the pneumatic gate spillway is 160 feet. Taller gates could reduce the length of spillway but also as the gates get taller the foundation gets larger and the downstream tailwater could impact discharge for floods approaching the ½ PMF magnitude.

Figure B-9 displays Stage-discharge curves were developed for each of the Spillway Alternatives. Combined with the lift gates, all three alternatives can pass the ½ PMF without overtopping the existing powerhouse/spillway which with upstream exterior walls has an overtopping elevation of approximately 906.0 ft-msl.



Spillway Alternative Stage-Discharge Curves
Figure B-9

Comparison of Three Spillway Alternatives

All three spillway alternatives have distinct advantages and disadvantages. Without considering cost or operating/maintenance requirements, the Pneumatic Gates seem to be the best option, they take up the least amount of area, and they provide normal pool control over a wider range of flows. However, pneumatic gates require additional mechanical and electrical systems that are not required for the labyrinth weir spillways. They also require additional operation and maintenance and have a service life of roughly 25 years, which is less than half of the service life of a concrete structure.

The single labyrinth is 30 feet longer than the pneumatic gates but requires no operation. There is a greater sense of security knowing that the principal spillway is not subject to operation and maintenance of equipment. This is not to suggest that a labyrinth spillway will not require maintenance such as debris removal, but over normal day-to-day flows, the fixed labyrinth crest will provide a normal pool within 6 inches of 896.3 ft-msl for river flows up to 500 cfs without operating the lift gates. Reviewing daily flows at the USGS Highway 20 gage between April and December of 2011, lift gates would have been used on approximately 24 days out of the 250-day period. On the flow duration curve for the gage record this translates to roughly 10% of the time (similar to the 4/11-12/11 time period). The amount of time lift gates are used would change year to year depending on rain events that occur but the operation requirement for a single labyrinth weir is significantly less than the pre-breach dam or the pneumatic gate spillway.

Over the same time period with the dual labyrinth weir, lift gates would have been used on approximately 68 days out of the 250-day period. With a shorter principal spillway, the hydraulic capacity for discharging flows within 6 inches of the normal pool is 300 cfs (compared to 500 cfs with the single labyrinth spillway), so the lift gates have to be used more frequently. On the flow duration curve this translates to roughly 25% of the time (similar to the 4/11-12/11 time period). The dual labyrinth weir is also 50 feet longer than the single labyrinth weir, so additional grading will be needed along the south river bank to fit the dual spillways and chutes within the embankment and channel banks. The potential advantage of the dual labyrinth weir over the single labyrinth would be cost of construction where chute and stilling basin concrete (expensive) could be substituted for articulated concrete block or roller compacted concrete (cheaper) for the auxiliary spillway saving money on the overall construction cost. However, if the additional cost of grading and shaping the embankment and channel area for the larger dual labyrinth weir spillway is close to the cost savings of using less concrete, then the single labyrinth weir spillway would be the better option.

Spillway computations are provided in this appendix.

B.6 Other Hydraulic Issues

While the spillway alternatives analysis was the main component of the hydrologic and hydraulic studies, several other hydraulic considerations were evaluated that could impact design and construction. These issues included:

- Minimum/low flow passage
- Lake draining capacity
- Cofferdam/flow bypass during construction.

Minimum/Low Flow Passage

Minimum/low flow passage was a topic of concern with operation of the pre-breach Lake Delhi Dam. During times of normal and low flows, flow downstream of the dam was controlled by wicket gate discharge. Wicket gate settings and pool elevations were recorded but discharge rates were not quantified. During times of low flow there were concerns that insufficient discharge was being provided to the downstream waterway.

An additional concern was dissolved oxygen levels of the discharge. The wicket gates intake elevation is at 881.3, roughly 15 feet below the normal pool elevation where dissolved oxygen levels are typically low. Discharge through the gates was not aerated so waters in immediate downstream channel frequently did not meet dissolved oxygen requirements.

If the wicket gates are restored as the normal means of discharge, an aeration mechanism will be incorporated into the system. If the labyrinth or pneumatic gates are used as the single principal spillway sufficient aeration will be provided by the pool level discharge and flow down the spillway chute.

In addition to the spillway alternatives, installation of valved openings in two of the new lift gates is being considered. During normal operating conditions the valves would be closed. However the valves could be used to:

- Provide additional discharge capacity prior to gates lifting (roughly 150 cfs for two 30-inch valves at normal pool)
- Provide minimum flow passage if the upstream pool drops below the principal spillway crest.
- Provide bypass flow during potential maintenance work or debris removal at the principal spillway without lifting gates.
- Provide the capability to draw down the pool a small amount or maintain a slightly drawn down pool during low flows. The lift gates are good for passing large flows but not for normal bypass flows or drawing down the pool a few inches.

Unlike the wicket gates, the valves will discharge onto the concrete ogee spillway, so even though the valves would likely be 10 feet below the normal pool, discharge would be aerated by the drop over the concrete spillway.

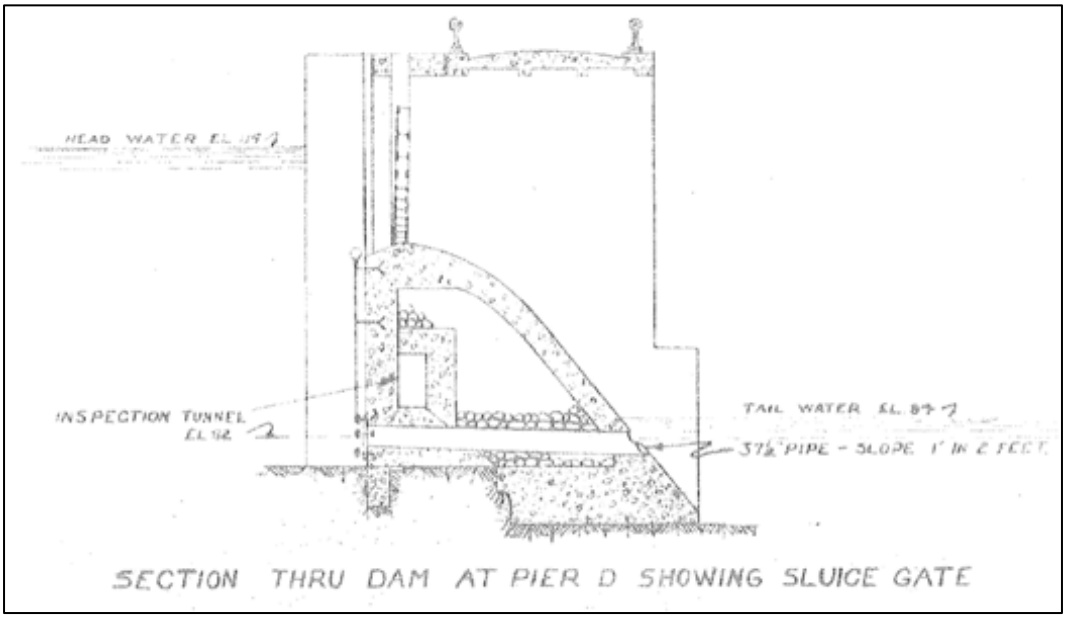
The previous dam operator indicated the 7Q10 flow (lowest 7-day average flow that occurs once every 10 years) for the Maquoketa River at Lake Delhi Dam is roughly 28 cfs. The 30-inch valves would have the capacity to discharge the 7Q10 flow.

As reconstruction design progresses a detailed operating manual will be developed with DNR input and approval that provides operating protocol and discharge rates for the expected range of flow conditions.

Lake Draining Capacity

DNR requires that "A gated low level outlet shall be provided which is capable of draining at least 50 percent of the permanent storage behind the dam within a reasonable length of time." The existing lift gates provide sufficient capacity to drain 50 percent of the volume below the normal pool elevation. In addition, existing plans indicate a set of two 37.5-inch diameter sluice pipes were installed through the northernmost spillway pier approximately 20 feet below the crest of the gated spillway.

If they do exist, the sluice pipe intakes are buried under 20 feet of riprap. This riprap will be removed during the dam reconstruction and the feasibility of restoring the existing sluice pipes will be assessed. The sluice pipes are not necessary to meet DNR design requirements but could be useful during construction and for future maintenance and dredging projects. A copy of the section drawing is shown in Figure B-10.



Potential Existing Sluice Pipes
Figure B-10

Cofferdam/Flow Bypass During Construction

At this stage, the dam reconstruction has been separated into two phases. The first phase would involve restoration of the existing powerhouse/spillway structure and north embankment. The second phase would involve construction of the southern embankment and new spillway.

For the first phase, cofferdams are only needed to prevent high river flows from entering the construction area and for dewatering. Flow bypass is provided by the eroded section of the dam. Relatively short cofferdams would be constructed upstream of the existing spillway and riprap area and downstream of the existing stilling basin.

The size of the second phase cofferdams will be partially controlled by the condition of the existing sluice pipes. If the sluice pipes are restorable, significant bypass capacity can be provided without construction of a tall upstream cofferdam (saves construction time and money). If the sluice pipes are not restorable the cofferdam will have to be constructed several feet above the existing gated spillway crest in order to a means for flow bypass. A taller cofferdam will mean a higher pool during construction (benefit to lake residents) but greater risk and additional cost to the project.

Using the USGS frequency analysis, required cofferdam heights were estimated for given return period flows. Table B-8 provides a summary.

Table B-8 Cofferdam Height Estimates

Return Period (years)	River Flow (cfs)	Phase I Cofferdams				Phase II Cofferdams			
		Upstream		Downstream		Upstream		Downstream	
		Elev.	H (ft)	Elev.	H (ft)	Elev.	H (ft)	Elev.	H (ft)
1	1,400	869.5	0	861.5	4	883	18	861.5	2
2	4,500	872	2	866.1	8	886.5	22	865.9	6
5	8,700	874	4	869.5	12	890.5	26	870	10
10	12,300	876	6	872.2	14	893	28	871.9	12

For the cofferdam analysis, sluice pipes were assumed to be inoperable. The cofferdam heights shown in Table B-8 are strictly estimates based upon flow bypass capacity. Ultimately cofferdams will either be designed in subsequent phases of the project or at the discretion of the contractor based upon his assessment of risk.

B.7 References

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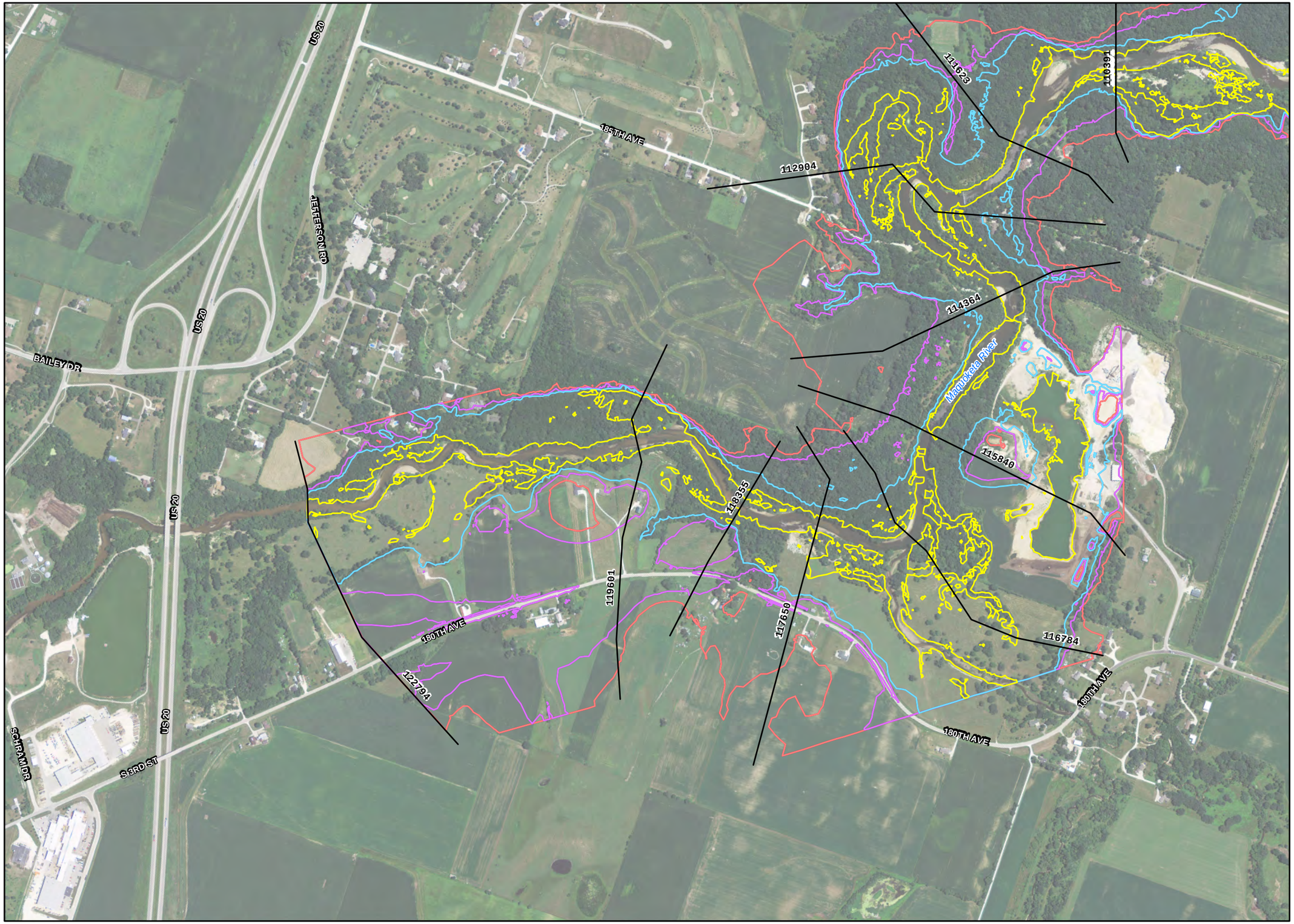


Stanley Consultants INC.

SEE SHEET 2

LEGEND

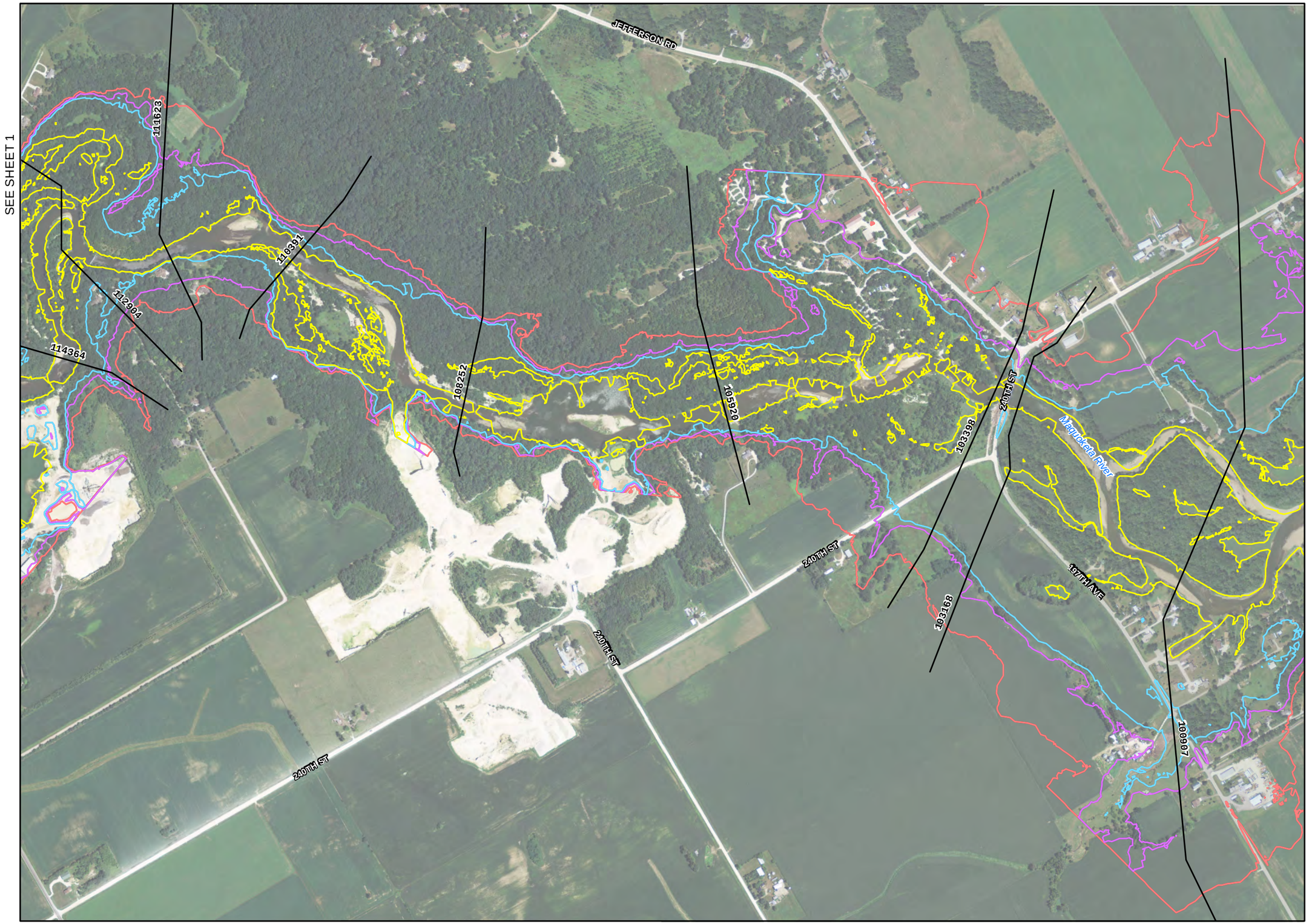
- HEC-RAS X-Sect
- Sunny Day Breach
- 100YR
- 100YR Breach
- Half PMF
- Half PMF Breach
- PMF
- PMF Breach

0 400 800 Feet

LAKE DELHI DAM RECONSTRUCTION

INUNDATION MAPS

SHEET 1



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LEGEND

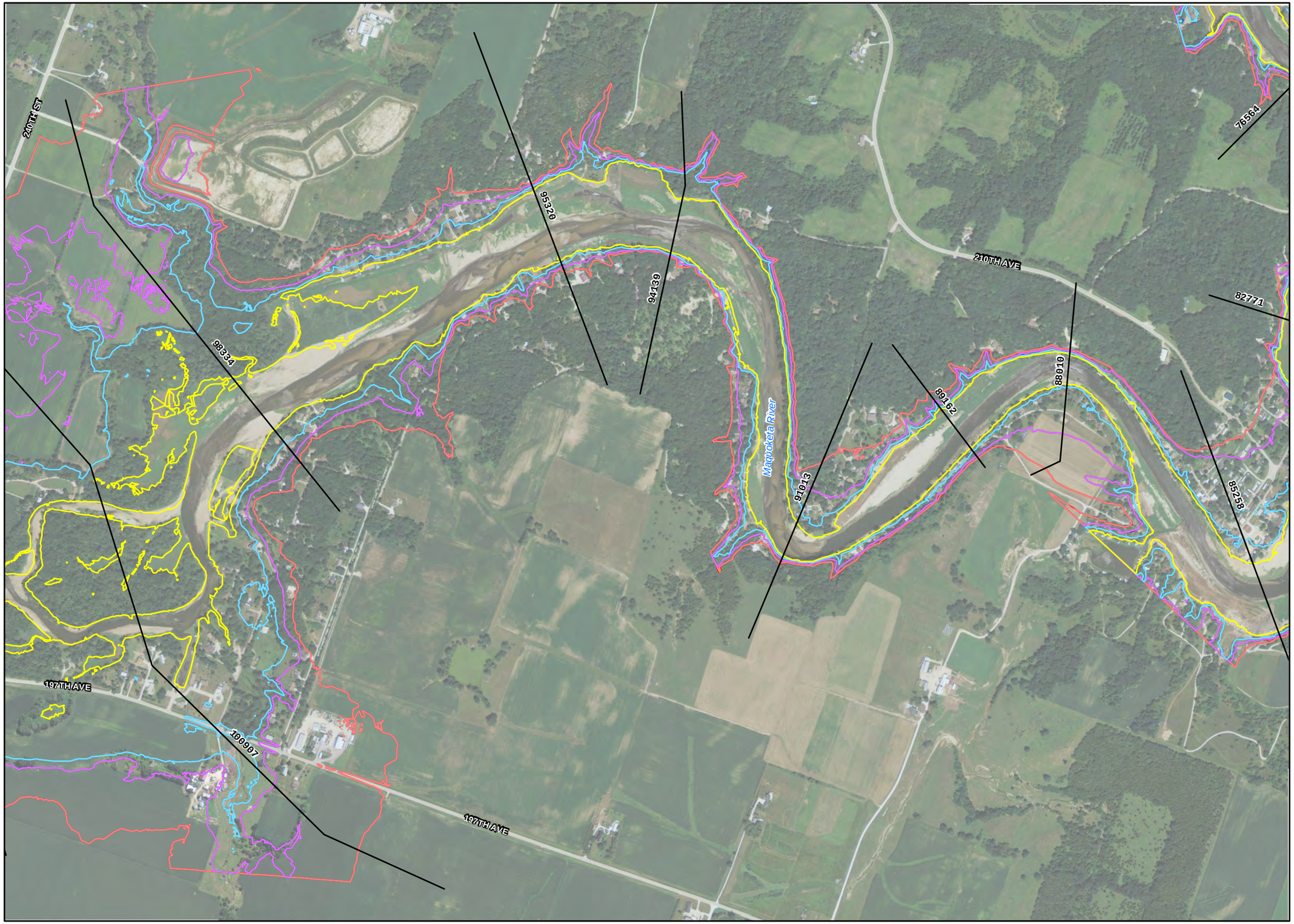
- HEC-RAS X-Sect
- Sunny Day Breach
- 100YR
- 100YR Breach
- Half PMF
- Half PMF Breach
- PMF
- PMF Breach

0 400 800 Feet

LAKE DELHI DAM RECONSTRUCTION

INUNDATION MAPS

SHEET 2



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LEGEND

- HEC-RAS X-Sect
- Sunny Day Breach
- 100YR
- 100YR Breach
- Half PMF
- Half PMF Breach
- PMF
- PMF Breach

0 400 800 Feet

LAKE DELHI DAM RECONSTRUCTION

INUNDATION MAPS

SHEET 3

Stanley Consultants INC.

LEGEND

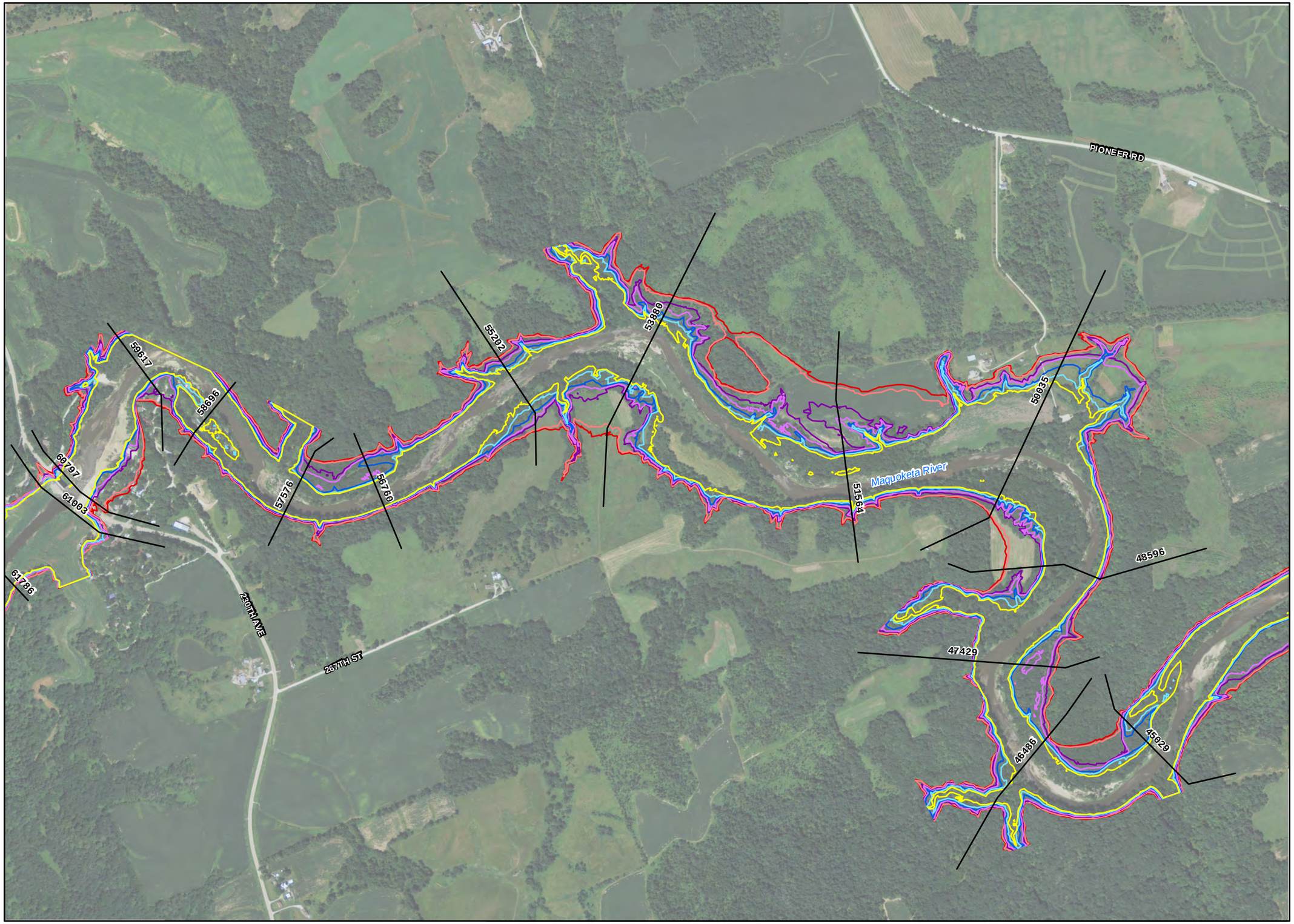
- HEC-RAS X-Sect
- Sunny Day Breach
- 100YR
- 100YR Breach
- Half PMF
- Half PMF Breach
- PMF
- PMF Breach

0 400 800 Feet

LAKE DELHI DAM RECONSTRUCTION

INUNDATION MAPS

SHEET 4



Stanley Consultants INC.

LEGEND

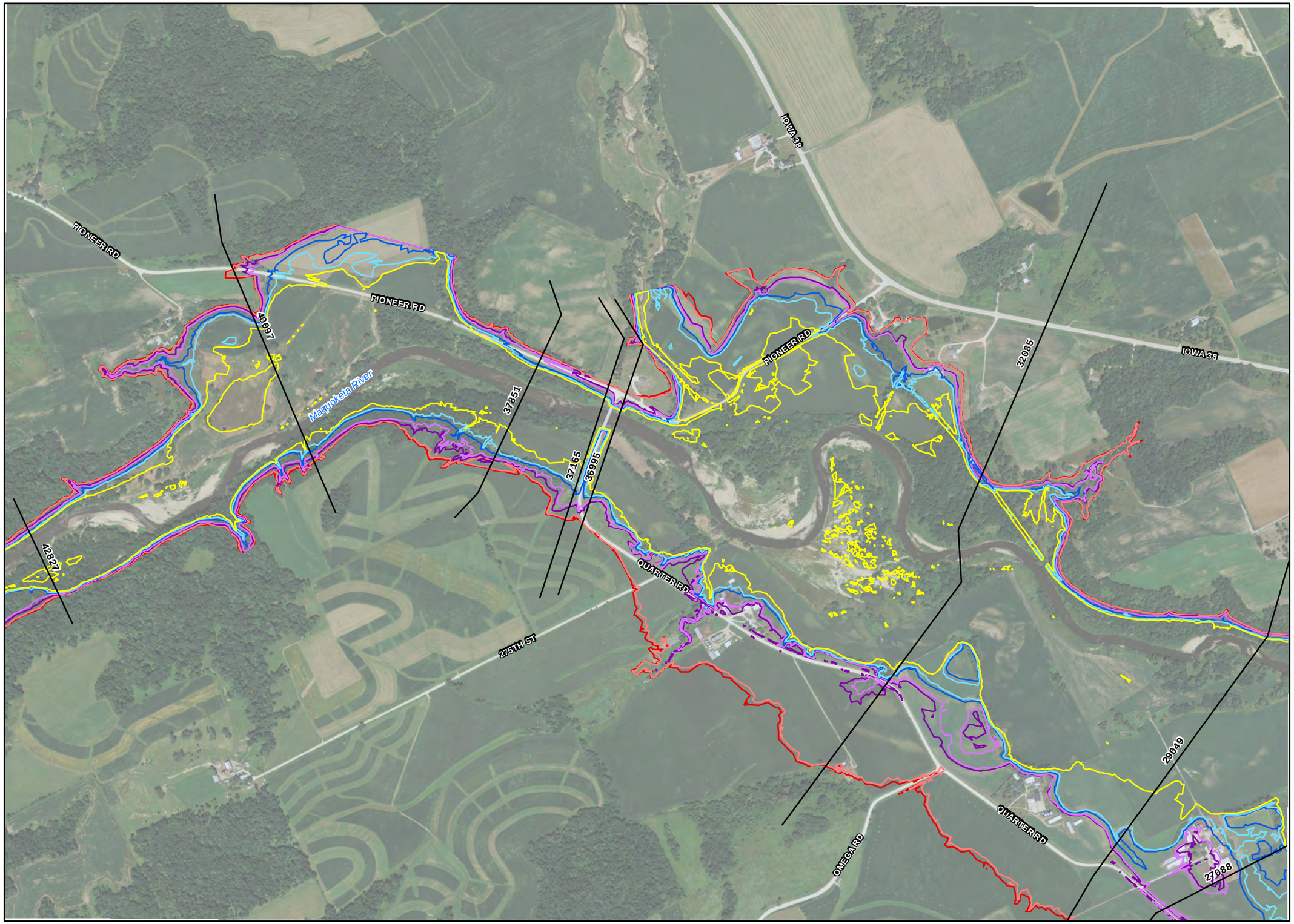
- HEC-RAS X-Sect
- Sunny Day Breach
- 100YR
- 100YR Breach
- Half PMF
- Half PMF Breach
- PMF
- PMF Breach

0 400 800 Feet

LAKE DELHI DAM RECONSTRUCTION

INUNDATION MAPS

SHEET 5



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LEGEND

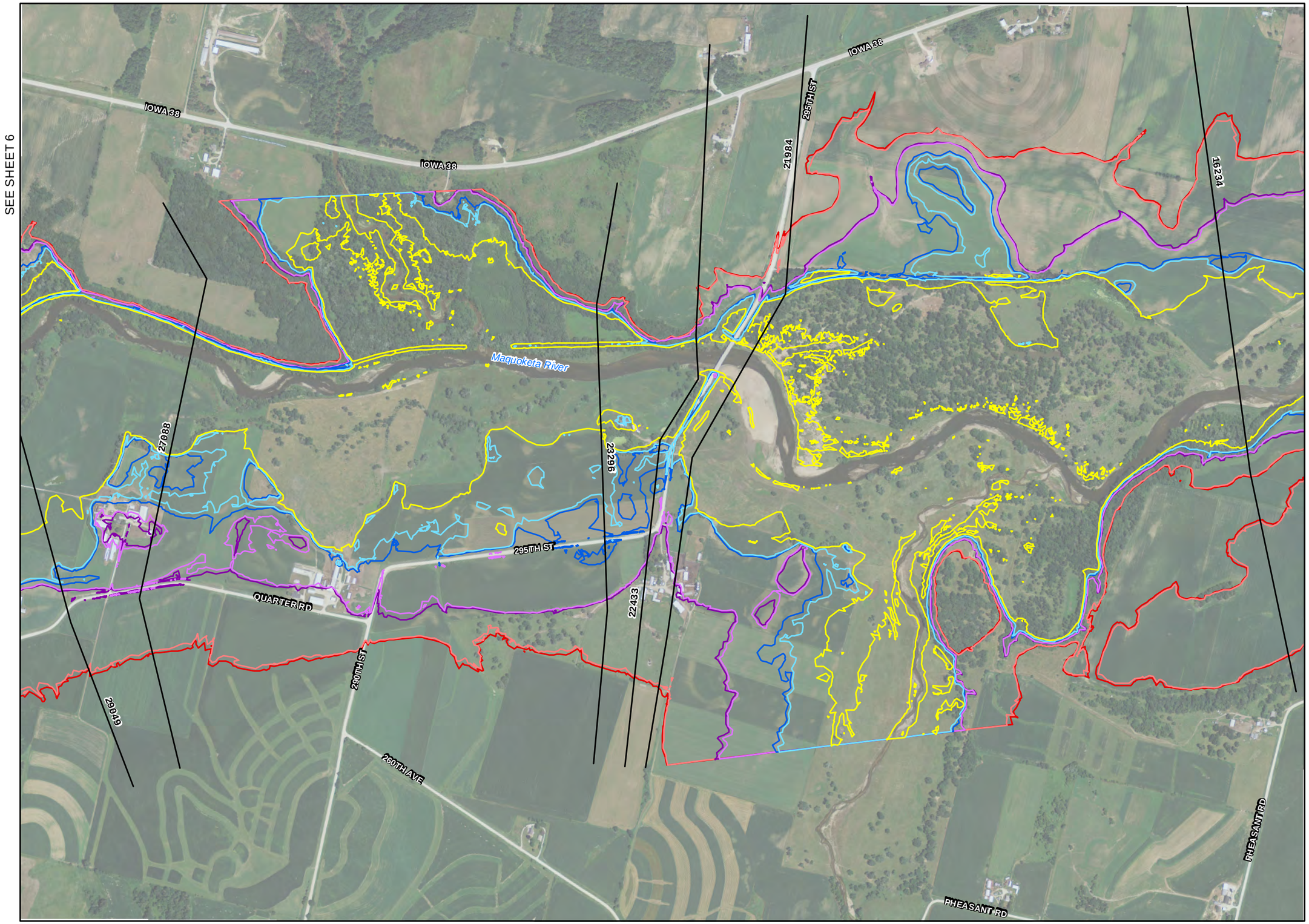
- HEC-RAS X-Sect
- Sunny Day Breach
- 100YR
- 100YR Breach
- Half PMF
- Half PMF Breach
- PMF
- PMF Breach

0 400 800 Feet

LAKE DELHI DAM RECONSTRUCTION

INUNDATION MAPS

SHEET 6



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LEGEND

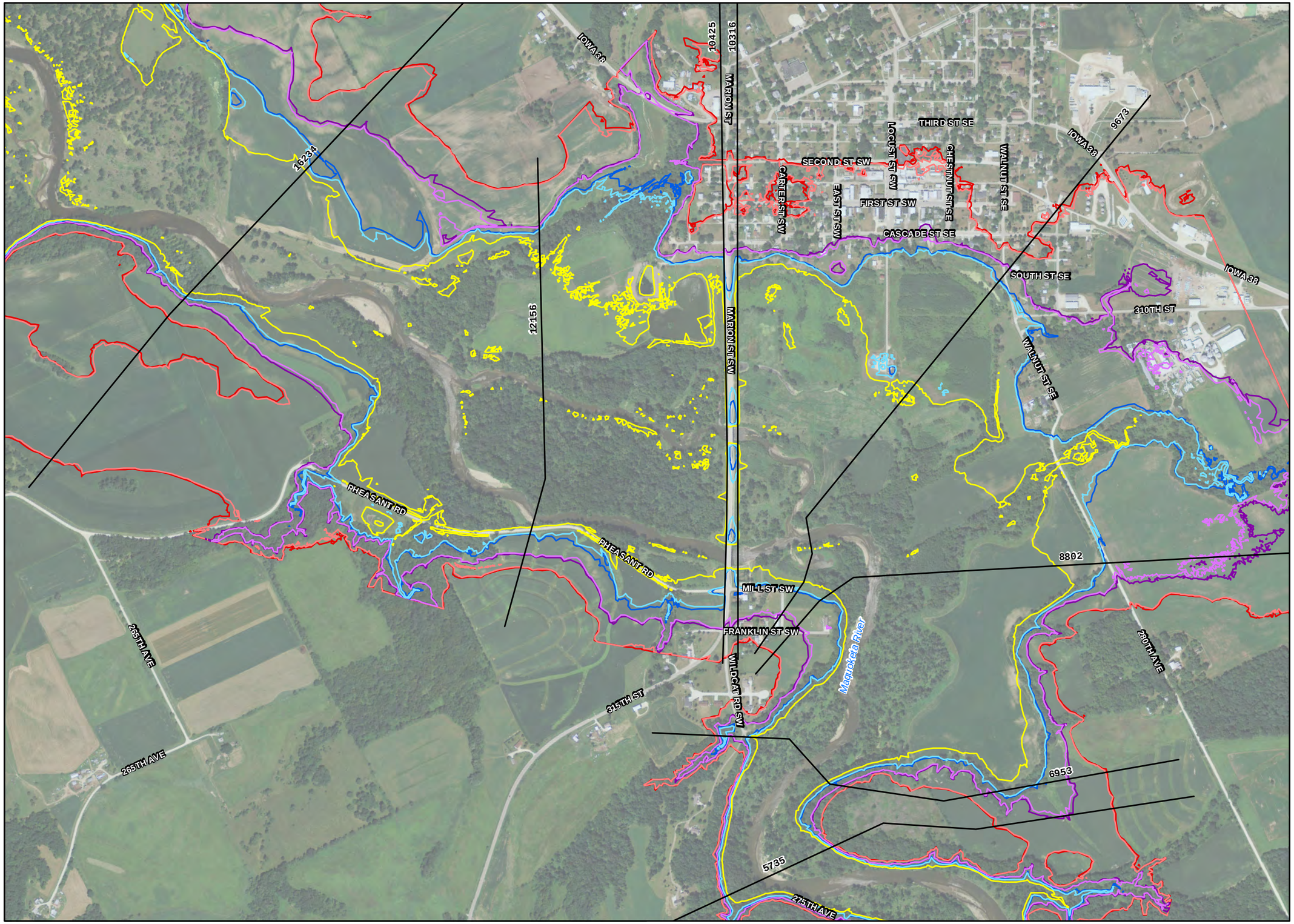
- HEC-RAS X-Sect
- Sunny Day Breach
- 100YR
- 100YR Breach
- Half PMF
- Half PMF Breach
- PMF
- PMF Breach

0 400 800 Feet

LAKE DELHI DAM RECONSTRUCTION

INUNDATION MAPS

SHEET 7



Stanley Consultants INC.

LEGEND

- HEC-RAS X-Sect
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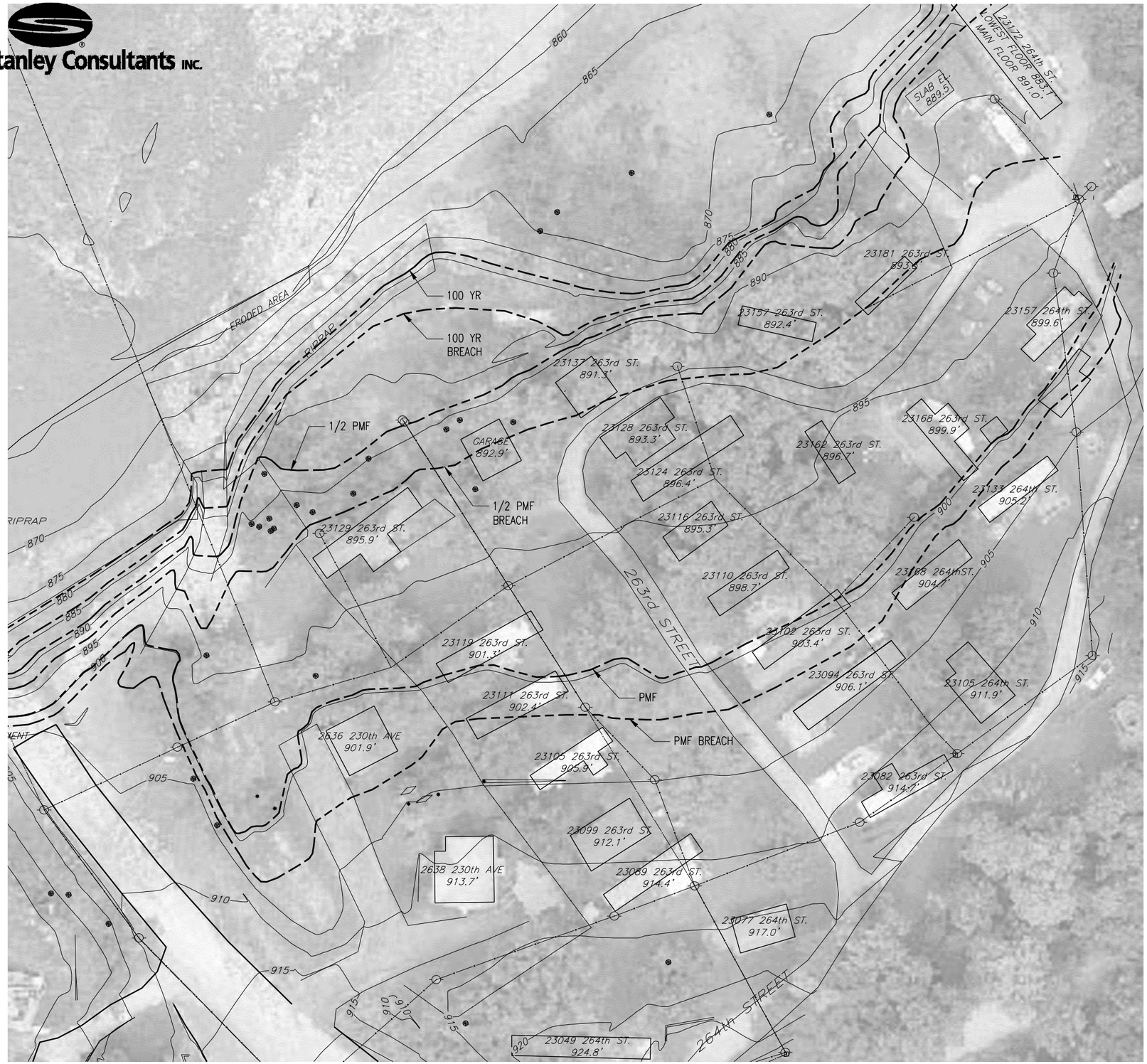
0 400 800 Feet

LAKE DELHI DAM RECONSTRUCTION

INUNDATION MAPS

SHEET 8

Stanley Consultants INC.



PLAN

PROPERTY INUNDATION DEPTH TABLE

Property			PMF Depth		1/2 PMF Depth		100YR Depth	
Address	Street	F.F. Elev.	No Breach	Breach	No Breach	Breach	No Breach	Breach
2636	230th Ave	901.9	0.0	0.2	0.0	0.0	0.0	0.0
2638	230th Ave	913.7	0.0	0.0	0.0	0.0	0.0	0.0
23082	263rd St	914.7	0.0	0.0	0.0	0.0	0.0	0.0
23089	263rd St	914.4	0.0	0.0	0.0	0.0	0.0	0.0
23094	263rd St	906.1	0.0	0.0	0.0	0.0	0.0	0.0
23099	263rd St	912.1	0.0	0.0	0.0	0.0	0.0	0.0
23102	263rd St	903.4	0.0	0.0	0.0	0.0	0.0	0.0
23105	263rd St	905.9	0.0	0.0	0.0	0.0	0.0	0.0
23110	263rd St	898.7	1.0	3.2	0.0	0.0	0.0	0.0
23111	263rd St	902.4	0.0	0.0	0.0	0.0	0.0	0.0
23116	263rd St	895.3	4.4	6.6	0.0	0.0	0.0	0.0
23119	263rd St	901.3	0.0	0.7	0.0	0.0	0.0	0.0
23124	263rd St	896.4	3.3	5.5	0.0	0.0	0.0	0.0
23128	263rd St	893.3	6.4	8.6	0.0	0.0	0.0	0.0
23129	263rd St	895.9	3.9	6.1	0.0	0.0	0.0	0.0
23137	263rd St	891.3	8.4	10.6	0.0	1.2	0.0	0.0
23157	263rd St	892.4	7.2	9.5	0.0	0.0	0.0	0.0
23162	263rd St	896.7	0.0	0.0	0.0	0.0	0.0	0.0
23168	263rd St	904.7	2.9	5.2	0.0	0.0	0.0	0.0
23181	263rd St	893.3	6.3	8.6	0.0	0.0	0.0	0.0
23049	264th St	924.8	0.0	0.0	0.0	0.0	0.0	0.0
23077	264th St	917	0.0	0.0	0.0	0.0	0.0	0.0
23105	264th St	911.9	0.0	0.0	0.0	0.0	0.0	0.0
23168	264th St	904.7	0.0	0.0	0.0	0.0	0.0	0.0
23133	264th St	905.2	0.0	0.0	0.0	0.0	0.0	0.0
23157	264th St	899.6	0.0	2.2	0.0	0.0	0.0	0.0
23172	264th St	883.1	16.4	18.7	5.1	9.2	0.0	0.2

0' 15' 30' 60'

SCALE: 1" = 60' -0"

LAKE DELHI DAM
RECONSTRUCTION ALTERNATIVES
HYDRAULIC ANALYSIS

DOWNSTREAM PROPERTY
INUNDATION EXHIBIT A

CADD D4-R3

Note: Aerial Photo shows Pre-Breach Condition

PROPERTY INUNDATION DEPTH TABLE								
Property			PMF Depth		1/2 PMF Depth		100YR Depth	
Address	Street	F.F. Elev.	No Breach	Breach	No Breach	Breach	No Breach	Breach
26287	231st Ave	889.1	10.0	12.3	0.0	2.7	0.0	0.0
26294	231st Ave	894.4	4.7	7.0	0.0	0.0	0.0	0.0
26299	231st Ave	892.4	6.7	9.0	0.0	0.0	0.0	0.0
26269	232nd Ave	879.8	19.3	21.6	8.0	12.0	0.0	3.0

LAKE DELHI DAM
RECONSTRUCTION ALTERNATIVES
HYDRAULIC ANALYSIS

DOWNSTREAM PROPERTY
INUNDATION EXHIBIT B

LAKE DELHI DAM DOWNSTREAM STRUCTURE AND ROADWAY FLOOD INUNDATION TABLE

Structure	River Sta	Residences							Commerical/Agricultural Bldgs							Roads						
		PMF		Half PMF		100YR		Sunny Day	PMF		Half PMF		100YR		Sunny Day	PMF		Half PMF		100YR		Sunny Day
		No Breach	Breach	No Breach	Breach	No Breach	Breach	Breach	No Breach	Breach	No Breach	Breach	No Breach	Breach	Breach	No Breach	Breach	No Breach	Breach	No Breach	Breach	Breach
Lake Delhi Dam	60900																					
	60797	9	12	1	2		1									Local	Local					
	59617	4	4	1	2		1											Dwy	Local		Dwy	
	58696																					
	57576																					
	56760																					
	55202																					
	53880																					
	51564																					
	50035															Dwy	Dwy	Dwy	Dwy	Dwy	Dwy	Dwy
	48596																					
	47429																					
	46486																					
	45029																					
	42827																					
	40097															Pioneer Rd	Pioneer Rd	Pioneer Rd	Pioneer Rd	Pioneer Rd	Pioneer Rd	Pioneer Rd
	37851															Pioneer Rd	Pioneer Rd	Pioneer Rd	Pioneer Rd	Pioneer Rd	Pioneer Rd	Pioneer Rd
	37165															Pioneer Rd	Pioneer Rd	Pioneer Rd	Pioneer Rd	Pioneer Rd	Pioneer Rd	Pioneer Rd
Quarter Road Bridge	37080															Overtop	Overtop	Overtop	Overtop			
	36995	1	1						4	4	2	2		1		Pionr/Quarter	Pionr/Quarte	Pioneer Rd	Pioneer Rd	Pioneer Rd	Pioneer Rd	Pioneer Rd
	32085	1	1						1	1						Quarter Rd	Quarter Rd					
	29049	1	1								1	1				Quarter Rd	Quarter Rd					
	27088	3	3	2	2											38/295th	38/295th	38/295th	38/295th	Hwy 38	Hwy 38	
	23296	1	1	1	1	1	1		1	1	1	1	1	1								
	22433	1	1						1	1	1	1										
295th Street Bridge	22180															Overtop	Overtop	Overtop	Overtop			
	21984															260/297th	260/297th	260/297th	260/297th	260th	260th	
	16234	1	1						1	1						Pheasant	Pheasant	Pheasant	Pheasant	Pheasant	Pheasant	
	12156	1	1						2	2	1	1				Pheasant	Pheasant	Pheasant	Pheasant	Pheasant	Pheasant	
	10425	12	12	2	2											Local	Local	Pheasant	Pheasant	Pheasant	Pheasant	
Hopkinton Bridge	10370															Overtop	Overtop	Overtop	Overtop	Overtop	Overtop	
	10316	45	45	12	12	1	1		10	10						Local	Local					
	9673	20	20	7	7	1	1		10	10	2	2				Wilson/38	Wilson/38	Wilson	Wilson	Wilson	Wison	
	8802	3	3	1	1											Local	Local	Local	Local			
	6953																					
	5735																					
	2528																					
	702																					
TOTAL		**103**	**106**	**27**	**29**	**3**	**5**	**0**	**30**	**30**	**8**	**8**	**1**	**2**	**0**	**12**	**12**	**8**	**8**	**5**	**5**	**1**

Stanley Consultants INC.

Computed by	A. Judd	Date	9/28/2011	Job No.	23601
Checked by	M. Weber	Date	10/12/2011	Subject	Lake Delhi Dam
Approved by		Date			Hydrology/Hydraulics
					Computation Summary

Description:

Summary of hydrologic and hydraulic analysis performed for
Lake Delhi Dam reconstruction project.

Reference:

(1) Iowa DNR, ***Design Criteria and Guidelines for Iowa Dams***, T.B. 16, 1990.
(2) Ashton-Barnes Engineers, Inc., ***Report of Inspection of Lake Delhi Dam on Maquoketa River***, 1998.
(3) FERC, ***Engineering Guidelines for the Evaluation of Hydropower Projects***, 1993, 2001.
(4) Independent Panel of Engineers, Report on Breach of Delhi Dam, Dec. 2010.
(5) USGS, ***Techniques for Estimating Flood-Frequency Discharges for Streams in Iowa***, WRIR 0-4233, 2001.
(6) NOAA/USACE, ***Probable Maximum Precipitation Estimates/Application***, HMR 51/52, 1978/1982.
(7) Colorado Dam Safety Branch, ***Guidelines for Dam Breach Analysis***, 2010.

Analysis:

Reconstruction of Lake Delhi Dam must meet requirements set forth in Ref 1. In addition, if hydropower generation is to be reinstated at Lake Delhi Dam, the reconstructed dam must meet design standards set forth in Ref 3. The hydrologic and hydraulic analysis was performed in adherence to both Ref 1 and Ref 3. The analysis consisted of the following steps.

	Task	Software	Inputs	Results
1	Establish/Verify Hydrology	ArcGIS	Ref 2/ArcGIS	Drainage Area, Infiltration Rate,Tc
2	Establish/Verify PMP	HMR 52	DA, Location	Hourly PMP Depths
3	Establish/Verify PMF	HEC-HMS	Tasks 1,2	PMF Inflow Hydrograph to Lake Delhi
4	Establish Hydraulic Model	HEC-GeoRAS	LiDAR	HEC-RAS model of Dam and River
5	Design Flood/Hazard Class	HEC-GeoRAS	PMF	Inundation Maps, Hazard Classification

Task 1 **Establish/Verify Hydrology**

As part of their 1997 inspection of Lake Delhi Dam, Ashton-Barnes completed a spillway adequacy analysis that included a development of a Probable Maximum Flood (PMF) and analysis of the dam's spillway capacity.

The analysis is described in Ref 2 and generally follows analysis methodology presented in Ref 1 and Ref 3. The Ref 2 analysis was reviewed and updated to reflect some adjustments to watershed/rainfall parameters.

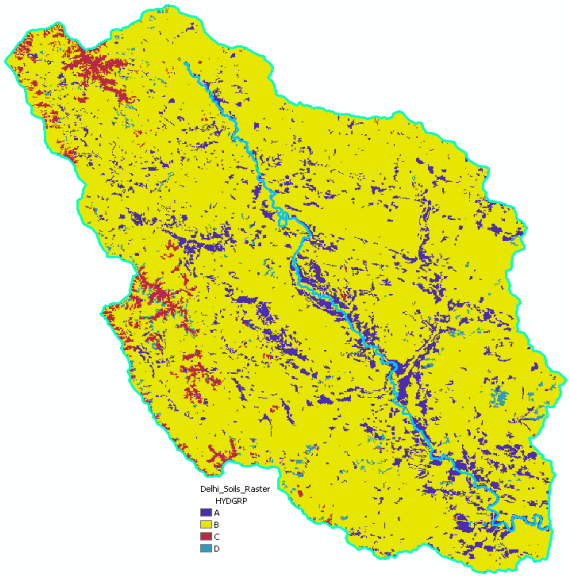
Parameter	Ref 2 Value	2011 Value	2011 Method
DA (mi^2)	347	349.2	ArcGIS - Iowa DNR supplied drainage area checked against HUC shapefiles
Infiltration	CN = 60	0.25 in/hr	See discussion below
Tc (hrs)	18	18	Clarks Unit hydrograph, see discussion below
PMP Total (in)	22.01	25.77	See Task 2

Discussion

Drainage area between the two studies was very close. For infiltration the Ref 2 study categorized the soil hydrologic group as A/B and land use as a 60/40 split between crops/meadow which gave a curve number of 60.

The soil hydrologic group was reviewed in ArcGIS using SSURGO soil data

Group	Area (mi^2)	Percent
A	36.3	10.4%
B	298.1	85.4%
C	10.7	3.1%
D	4.1	1.2%
Total	349.2	



For large watersheds, Ref 3 recommends using an initial and uniform loss rate for infiltration as opposed to the NRCS CN.

Ref 3 - Table 8-8.1

Hydrologic Group	Minimum Infiltration Rates (in./hr)	Soil Description
A	0.30 to 0.45	Deep sand, deep loess, aggregated silts
B	0.15 to 0.30	Shallow loess, sandy loam
C	0.05 to 0.15	Clay loams, shallow loam, soils low in organic content, soils usually high in clay
D	0 to 0.05	Soils that swell significantly when wet, heavy plastic clays, certain saline soils

Because the majority of soil in the watershed is Group B, this range of infiltration rates was used to represent the waterhsed's infiltration.

No initial infiltration amount was assumed (i.e. saturated conditions)

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Computed by	A. Judd	Date	9/28/2011	Job No.	23601
Checked by	M. Weber	Date	10/12/2011	Subject	Lake Delhi Dam
Approved by		Date			Hydrology/Hydraulics
					Computation Summary

Clark's unit hydrograph method was used in Ref 2 for hydrograph development which is the method recommended by Ref 3
Clark's method uses time of concentration and a storage parameter "R"

The Time of Concentration was computed in Ref 2 using the river length and an average travel time

River Length	36 miles	(45 miles River - 9 miles of Lake)	Length was verified using ArcGIS
Travel Time	18 hours	(3 ft/sec over 36 miles)	V_{river} was verified as reasonable using HEC-RAS and is in the range of the 4-5 hour travel time between the USGS gage and Delhi Dam established by Ref 4 (12 mi/5 hour = 3.5 ft/sec)

For R, Ref 2 used a ratio of Tc, 0.833*Tc = 15 hours
Ref 3 recommends computing R using the descending limb of a stream gage hydrograph (if available)

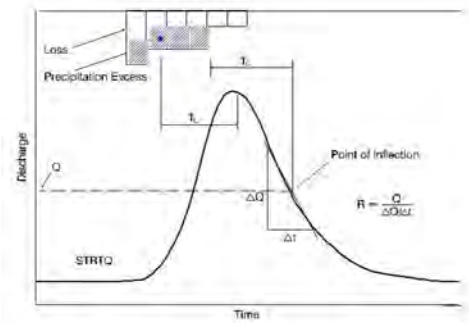

Figure 8-6.2

Stream gage data is available at for USGS Gage 05416900 near Manchester
http://waterdata.usgs.gov/ia/nwis/uv?site_no=05416900

This gage is approximately 12 miles upstream of the Lake Delhi Dam and the drainage areas are

DA_{Delhi} = 349 mi^2
DA_{Gage} = 300 mi^2

Ref 5 recommends estimating flows at Delhi by the following equation

$$Q_{t(aw)} = Q_{t(wg)} (DA_u / DA_g)^x, \quad (6)$$

where $Q_{t(aw)}$ = the area-weighted discharge estimate for an ungaged site on a gaged stream for recurrence interval t;
$Q_{t(wg)}$ = as defined for equation 2;
DA_u, DA_g = as defined for equation 3; and
x = the mean exponent for a hydrologic region; for Region 1, the mean exponent is 0.665; Region 2, 0.446; and Region 3, 0.403.

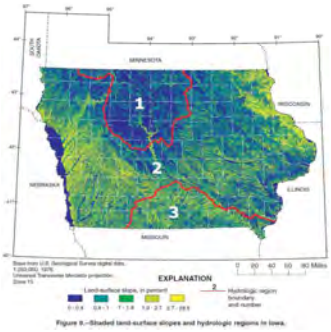

Figure 8.--Shaded land-surface slopes and hydrologic regions in Iowa.

Multiplier for Gage Flows at Delhi is $(349/300)^{0.446}$ = 1.070

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Checked by	M. Weber	Date	10/12/2011	Subject	Lake Delhi Dam
Approved by		Date			Hydrology/Hydraulics
					Computation Summary

15-min gage flows were obtained for three recent floods and multiplied to reflect flows at the Dam.

Clark's R was estimated for the floods using the method presented in Ref 3

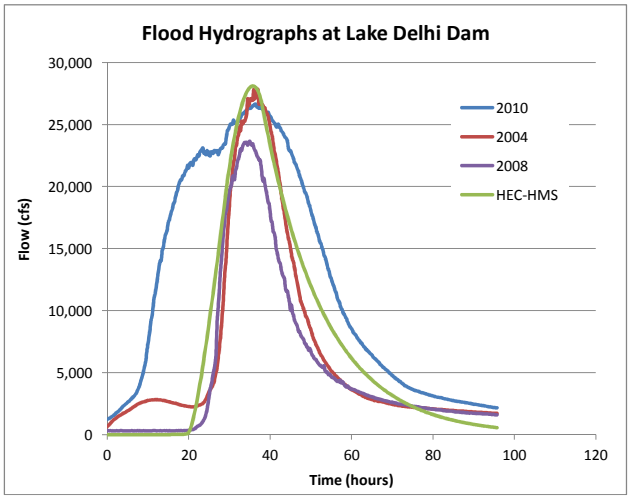
Year	Q (cfs)	ΔQ (cfs)	ΔT (hr)	R (hr)
2010	15100	9900	10	15.3
2008	15200	12960	10	11.7
2004	15200	15550	10	9.8

Both the 2010 and 2004 flood were considered to be close to the 100-year magnitude.

Using HEC-HMS the R and infiltration rate were adjusted to calibrate the 100-year storm (Precip=6.4") and resulting hydrograph to the 2004 hydrograph. The calibrated HEC-HMS hydrograph is shown in the graph.

From calibration the following parameters were developed to represent the Dam's watershed

R =	15 hr
Tc =	18 hr
Infilt =	0.25 in/hr



Task 2 **Establish/Verify PMP**

Probable Maximum Precipitation (PMP) events were developed using methods established by Ref 6.

The PMP is essentially an attempt to quantify the largest rainfall event that could be expected to occur over a given area. It is developed using the area location, area size, and parameters established by Ref 6.

HMR 51 establishes the PMP amount for a given duration and area size for the U.S. east of the 105th Meridian.
HMR 52 establishes a methodology for locating and distributing the PMP event over the given watershed area.

The PMP event is represented by Isohyetals (rainfall intensities) that are rotated and located over the basin to maximize rainfall depth

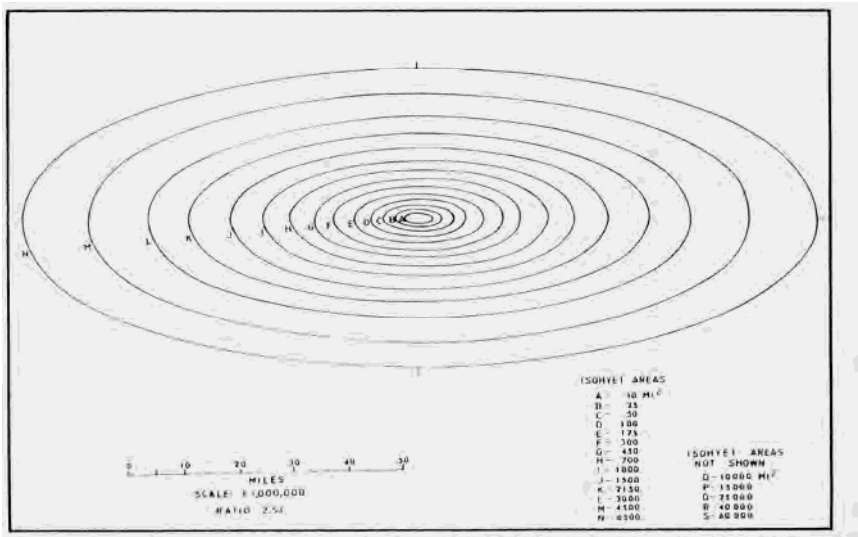


Figure 5.--Standard isohyetal pattern recommended for spatial distribution of PMP east of the 105th meridian (scale 1:1,000,000).

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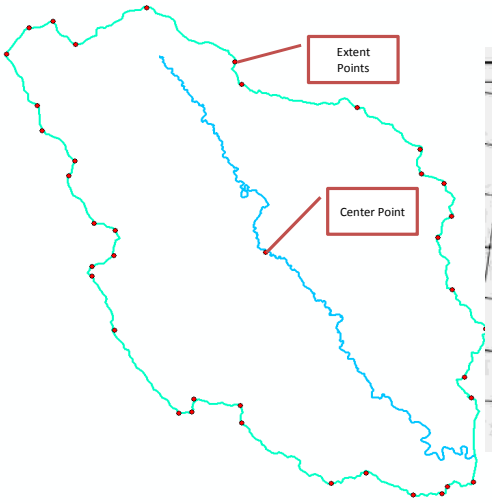
Computed by	A. Judd	Date	9/28/2011	Job No.	23601
Checked by	M. Weber	Date	10/12/2011	Subject	Lake Delhi Dam
Approved by		Date			Hydrology/Hydraulics
					Computation Summary

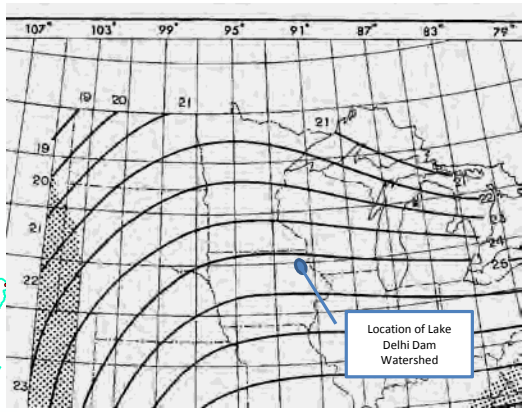
In addition to HMR51/52 manuals, HMR 52 software was developed to automate the development of the PMP for a given area

Inputs to HMR 52 are drainage area extent points, drainage area center, rainfall area/duration depths, and the expected isohyetal orientation which is provided in the HMR 52 manual.

Drainage area extents and center were determined using ArcGIS



Precipitation Depths were obtained from HMR 52 maps



PMP Area/Duration/Depths for Lake Delhi Dam					
Area (mi^2)	6-hour	12-hour	24-hour	48-hour	72-hour
10	25.2	29.5	31	34.4	36.2
200	18.3	21.9	23.8	26.3	28.2
1000	13.5	16.1	18.1	22	22.6
5000	8.3	10.9	12.5	15.2	17.1
10000	6.5	8.7	10.3	13.2	14.9
20000	4.6	6.7	8.2	11.1	12.6

This information is input into HMR 52 software, which then maximizes the rainfall through storm placement and duration

The Output of HMR 52 provides the following precipitation distribution for Delhi Dam's PMP

PROBABLE MAXIMUM STORM FOR DELHI

DAY 1

TIME	PRECIPITATION INCR	PRECIPITATION TOTAL	TIME	PRECIPITATION INCR	PRECIPITATION TOTAL	TIME	PRECIPITATION INCR	PRECIPITATION TOTAL	TIME	PRECIPITATION INCR	PRECIPITATION TOTAL
0100	0.05	0.05	0700	0.06	0.36	1300	0.08	0.75	1900	0.11	1.25
0200	0.05	0.10	0800	0.06	0.42	1400	0.08	0.82	2000	0.11	1.35
0300	0.05	0.15	0900	0.06	0.49	1500	0.08	0.90	2100	0.11	1.46
0400	0.05	0.20	1000	0.06	0.55	1600	0.08	0.98	2200	0.11	1.57
0500	0.05	0.25	1100	0.06	0.61	1700	0.08	1.06	2300	0.11	1.68
0600	0.05	0.30	1200	0.06	0.67	1800	0.08	1.14	2400	0.11	1.79
6-HR TOTAL	0.30			0.37			0.47			0.65	

DAY 2

TIME	PRECIPITATION INCR	PRECIPITATION TOTAL	TIME	PRECIPITATION INCR	PRECIPITATION TOTAL	TIME	PRECIPITATION INCR	PRECIPITATION TOTAL	TIME	PRECIPITATION INCR	PRECIPITATION TOTAL
0100	0.15	1.94	0700	0.34	3.18	1300	1.09	6.80	1900	0.32	22.46
0200	0.16	2.10	0800	0.37	3.56	1400	1.86	8.66	2000	0.29	22.74
0300	0.17	2.27	0900	0.42	3.98	1500	2.85	11.51	2100	0.26	23.00
0400	0.18	2.45	1000	0.49	4.46	1600	6.56	18.07	2200	0.24	23.24
0500	0.19	2.64	1100	0.57	5.03	1700	2.45	20.52	2300	0.22	23.47
0600	0.20	2.84	1200	0.67	5.70	1800	1.63	22.14	2400	0.21	23.68
6-HR TOTAL	1.05			2.86			16.44			1.54	

DAY 3

TIME	PRECIPITATION INCR	PRECIPITATION TOTAL	TIME	PRECIPITATION INCR	PRECIPITATION TOTAL	TIME	PRECIPITATION INCR	PRECIPITATION TOTAL	TIME	PRECIPITATION INCR	PRECIPITATION TOTAL
0100	0.13	23.81	0700	0.09	24.57	1300	0.07	25.09	1900	0.06	25.49
0200	0.13	23.95	0800	0.09	24.66	1400	0.07	25.16	2000	0.06	25.55
0300	0.13	24.08	0900	0.09	24.75	1500	0.07	25.23	2100	0.06	25.60
0400	0.13	24.21	1000	0.09	24.84	1600	0.07	25.30	2200	0.06	25.66
0500	0.13	24.35	1100	0.09	24.93	1700	0.07	25.37	2300	0.06	25.71
0600	0.13	24.48	1200	0.09	25.03	1800	0.07	25.44	2400	0.06	25.77
6-HR TOTAL	0.80			0.54			0.41			0.33	

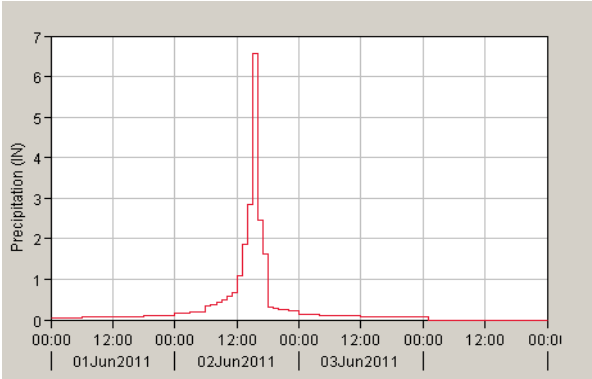
Note that the HMR 52 method uses an area weighted average (for each isohyetal) to determine the average precipitation depth over the entire Delhi Watershed. Given the uncertainty involved with this magnitude of a rainfall event, this averaging is considered acceptable.

Stanley Consultants INC.

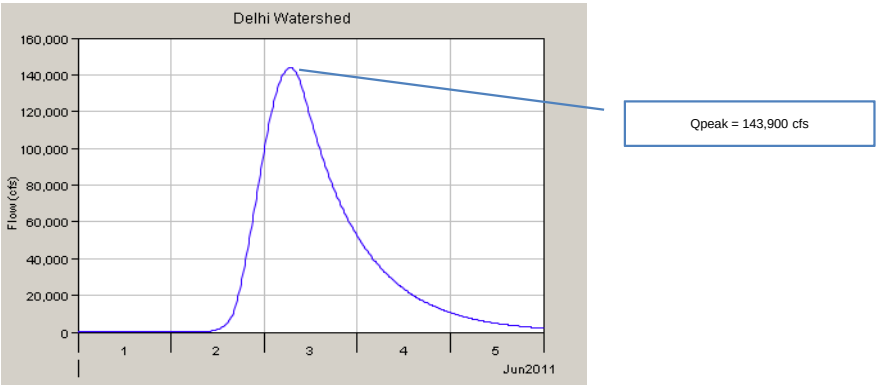
Computed by	A. Judd	Date	9/28/2011	Job No.	23601
Checked by	M. Weber	Date	10/12/2011	Subject	Lake Delhi Dam
Approved by		Date			Hydrology/Hydraulics
					Computation Summary

Task 3 **Establish/Verify PMF**

The PMP distribution (aka hyetograph) was then input into HEC-HMS to route the rainfall



The resulting hydrograph is the Probable Maximum Flood (PMF)



As a point of comparison Ref 2 also performed a PMP/PMF analysis

The Ref 2 PMP was very similar but to develop their PMF, Ashton-Barnes only used Day 2 which lowered the peak of the PMF to 132,800 cfs

Ref 2 PMP

Day 1			
0.32	0.39	0.50	1.90
Day 2			
1.09	2.73	16.63	1.56
Day 3			
0.84	0.58	0.44	0.35

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Task 4 **Establish Hydraulic Model**

A hydraulic model of Lake Delhi Dam and the upstream and downstream waterway was used for the following:

- Routing flood hydrographs through Lake Delhi and downstream of the dam
- Analyzing impact of proposed spillway configurations on upstream and downstream peak flood elevations
- Analyzing dam breach scenarios
- Establishing flood elevations for inundation maps

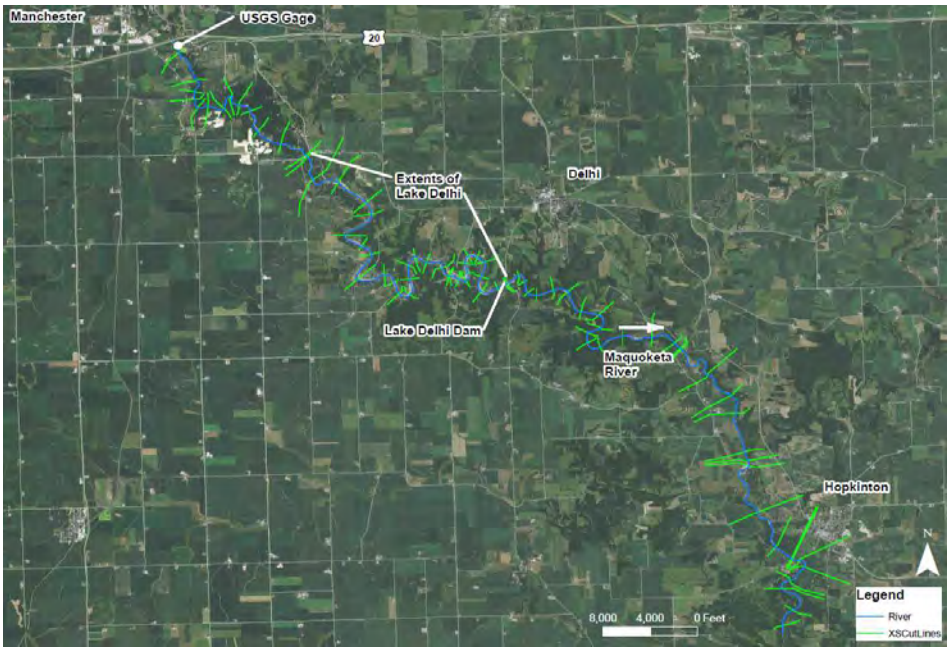
HEC-RAS software was used to establish a hydraulic model of the Maquoketa River and Lake Delhi Dam.
HEC-RAS is a USACE developed hydraulic analysis software that uses a series of river/stream cross-sections to model the hydraulics (water surface elevation, velocity, etc.) of a river segment under a user provided flow or hydrograph.

The HEC-RAS model of the Maquoketa River (including Lake Delhi Dam) was obtained from the DNR which had developed the model to analyze the 2010 flood and dam failure. The model was created using HEC-GeoRAS which involves layout out the river geometry in ArcGIS and exporting to HEC-RAS for analysis and importing results back to ArcGIS for mapping. Once the HEC-RAS model was established by the DNR it was checked and calibrated against the 2010 flood/breach event. Ref 4 provides a description of HEC-RAS model development. The model provided a good starting point for the dam reconstruction hydraulic analysis.

The following modifications/additions were made to the DNR HEC-RAS model:

- Cross-sections were regenerated using updated post-breach LiDAR (topography) data
- Cross-sections were extended up to elevations where the PMF was contained within the Cross-Section
- Bridge structures were added at Quarter Road, 295th Street and Hopkinton (using county-supplied Bridge plans)
- The inflow hydrograph near the dam was removed from the model* (see discussion below)
- The HEC-HMS PMF and 100-year hydrographs were input into HEC-RAS
- The unsteady analysis used the Mixed Flow Regime option (to account for critical flows, hydraulic jumps, etc.)
- The starting pool elevation was set to the normal pool of 896.3
- Lake Delhi Dam (represented as inline structure in HEC-RAS) was modified to proposed condition - see task 5

The HEC-RAS model extends from Hwy 20 to downstream of Hopkinton. The model cross-section locations and river alignment are shown in green and blue, respectively on the map below.



*The DNR HEC-RAS model used two inflow hydrographs, one at the upstresam end near the USGS gage and one near Lake Delhi Dam to reflect additional flows coming into the river. Given the relatively small increase in flows estimated (see Task 1 - drainage area ratio multiplier analysis) between the USGS gage and the dam, it was decided to use one inflow hydrograph at the upstream end of the model. This hydrograph represents the tributary drainage area at the dam. Locating the full tributary area hydrograph at the upstream end of the HEC-RAS model allows the hydrograph to be routed through Lake Delhi (9 miles of lake was excluded from travel time estimate in hydrologic analysis) which starts close to the upstream end of the HEC-RAS model.

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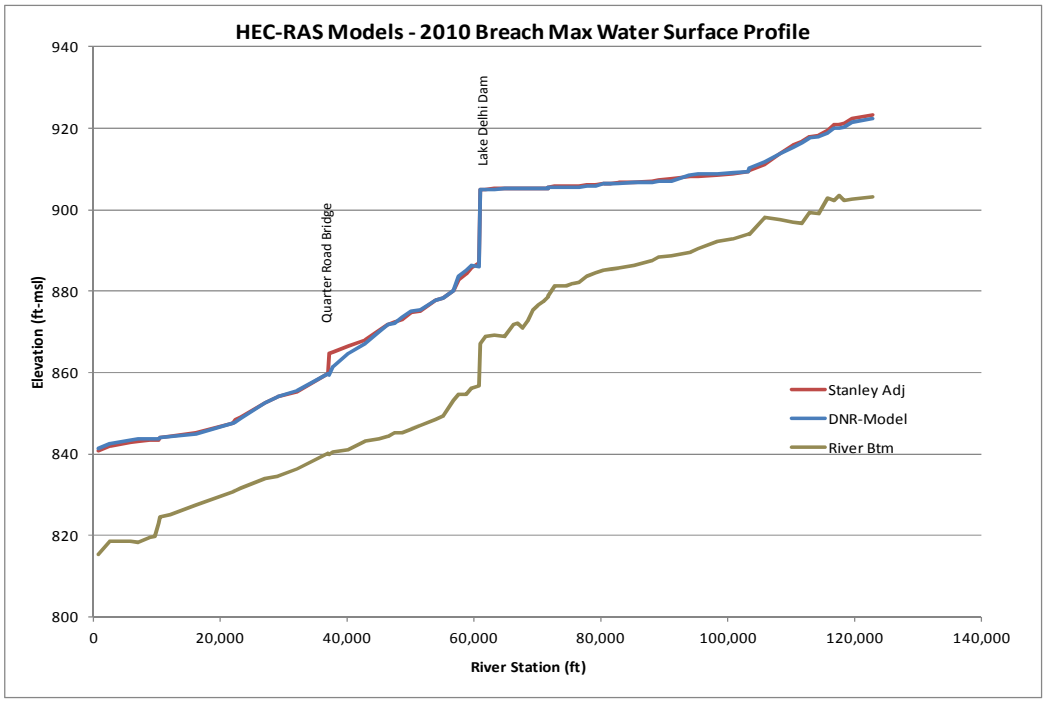
The Stanley Adjusted HEC-RAS model was compared to the DNR model for the 2010 breach and results corresponded fairly closely with the exception of the Quarter Road bridge added to the Stanley Model, which caused a rise in peak water surface.



Task 5 **Design Flood/Hazard Class**

Analysis and Design of the Lake Delhi Dam reconstruction will meet both DNR and FERC standards. The agencies' approach to defining a dam's design flood and hazard classification are shown below:

Iowa DNR (Ref 1)

Freeboard Design Flood

The specified freeboard design flood represents the greatest flood the dam must be designed to accommodate. The flood must be passed without overtopping of the dam and endangering its safety or the dam must be designed to withstand such overflow. Some erosion damage in earth emergency spillways will be tolerated, provided the safety of the dam would not be compromised.

FERC (Ref 3)

Inflow Design Flood (IDF) - The floodflow above which the incremental increase in water surface elevation due to failure of a dam or other water impounding structure is no longer considered to present an unacceptable threat to downstream life or property. The IDF of a dam or other water impounding structure flood hydrograph is used in the design of a dam and its appurtenant works particularly for sizing the spillway and outlet works, and for determining maximum height of a dam, freeboard, and temporary storage requirements.

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The DNR establishes the dam's design flood by its hazard classification

All high hazard dams: the probable maximum flood (PMF).

All moderate hazard dams, and low hazard dams classified as major structures: one-half (0.5) of the probable maximum flood (a flood hydrograph produced by multiplying the ordinates of the PMF hydrograph by a factor of 0.5).

Lake Delhi Dam is a major structure so it will have to pass either the half PMF or full PMF, depending on its hazard classification.

The DNR defines Hazard Class by

Low Hazard. Structures located in areas where damages from a failure would be limited to loss of the dam, loss of livestock, damages to farm out-buildings, agricultural lands, and lesser used roads, and where loss of human life is considered unlikely.

Moderate Hazard. Structures located in areas where failure may damage isolated homes or cabins, industrial or commercial buildings, moderately traveled roads or railroads, interrupt major utility services, but without substantial risk of loss of human life.

In addition, structures where the dam and its impoundment are of themselves of public importance, such as dams associated with public water supply systems, industrial water supply or public recreation, or which are an integral feature of a private development complex, shall be considered moderate hazard for design and regulatory purposes unless a higher hazard class is warranted by downstream conditions.

High Hazard. Structures located in areas where failure may create a serious threat of loss of human life or result in serious damage to residential, industrial or commercial areas, important public utilities, public buildings, or major transportation facilities.

FERC establishes the dam's design flood by a dam breach analysis. A series of floods (fractions of the PMF) are routed through the hydraulic model during a failure and non-failure scenario. The resulting inundations (flooding extents) downstream of the dam are compared and the greatest flood where failure of the dam makes a difference in the hazard (safety and damage) potential downstream is set as the design flood.

The selection of the appropriate IDF for a dam is related to the hazard classification for the dam. The IDF for a dam having a low hazard potential is selected primarily to protect against loss of the dam and its benefits should a failure occur. The IDF for high and significant hazard potential dams is the maximum flood above which there are no significant incremental impacts on downstream life and property.

So, similar to the DNR the design flood is linked to Hazard Classification. However, unlike the DNR, the design flood could be a fraction between the half PMF and full PMF.

FERC defines Hazard Class by

Dams conforming to criteria for the low hazard potential category generally are located in rural or agricultural areas where failure may damage farm buildings, limited agricultural land, or township and country roads. Low hazard potential dams have a small storage capacity, the release of which would be confined to the river channel in the event of a failure and therefore would represent no danger to human life.

Significant hazard potential category structures are usually located in predominately rural or agricultural areas where failure may damage isolated homes, secondary highways or minor railroads; cause interruption of use or service of relatively important public utilities; or cause some incremental flooding of structures with possible danger to human life.

Dams in the high hazard potential category are those located where failure may cause serious damage to homes, agricultural, industrial and commercial facilities, important public utilities, main highways, or railroads, and there would be danger to human life.

Definition of hazard classes are very similar between the two guidance documents so a DNR Moderate Hazard Class dam should correspond to a FERC Significant Hazard Class dam.

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The hazard class/design flood evaluation was performed by developing inundation maps of various flow scenarios for a failure (breach) and no failure (non-breach) condition.

Inundation maps were developed using ArcGIS/HEC-GeoRAS/HEC-RAS. The process entailed:

1 Inputting the hydrograph (typically a fraction of the PMF) into HEC-RAS.
2 Running HEC-RAS unsteady flow for a breach and non-breach condition.
3 Exporting breach and non-breach peak water surface profiles to ArcGIS.
4 Using HEC-GeoRAS component of ArcGIS to develop the extent of flooding (i.e. inundation) from the given breach and non-breach event.
5 Comparing inundation extents to determine during what flow conditions a potential breach would increase the downstream hazard.

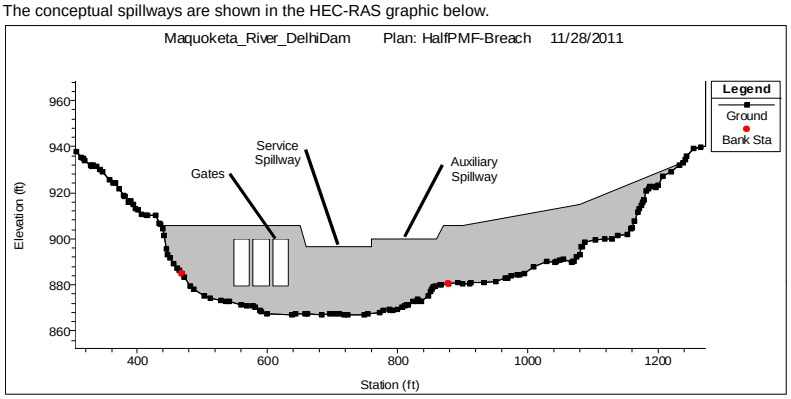
This evaluation includes several parameters, assumptions, and computations which are discussed below with results following

Service and Auxiliary Spillway Concept

Under previous conditions 3 spillway gates provided flood discharge (ignoring hydro) from Lake Delhi Dam. Once gate capacity was exceeded the dam was overtopped. The addition of a service and auxiliary spillway will supplement both the normal and flood discharge capacity of the dam. Per DNR/FERC guidelines the reconstructed dam will have to provide sufficient hydraulic capacity so the dam can safely passthe design flood.

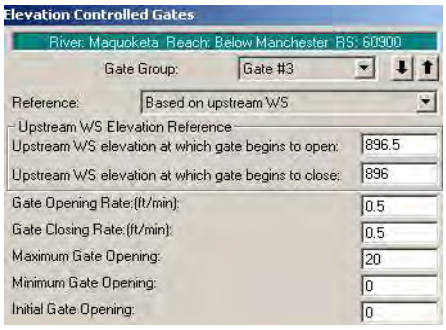
The initial establishiment of hazard class and design flood was performed prior to detailed design of the dam reconstruction. So a conceptual version (rough approximate) of the new service and auxiliary spillways were incorporated into the HEC-RAS model. Evaluation of the design flood/hazard class is focused more on the downstream impact of the dam than the specifics of the spillway so using a spillway approximation is reasonable for this phase of the analysis. The objective is to represent the influence of the spillways on the dam breach and downstream inundation/flooding (i.e. more flow in the downstream channel). Once the spillway designs are established the HEC-RAS model will be updated to reflect, but as long as a decent approximation of the proposed spillways are used the result of the design flood/hazard class evaluation should not change.

The conceptual spillways are shown in the HEC-RAS graphic below.



Normal Pool	896.3
Top of Dam	904.8
Gates	
Gate Crest	879.8
Gate Height	20 ft
Gate Width	25 ft
Service Spillway	
Spillway Crest	896.5
Spillway Width	100 ft
Weir Coeff.	3.4
Auxiliary Spillway	
Spillway Crest	900
Spillway Width	100 ft
Weir Coeff.	3.4

Note that the HEC-RAS model is an unsteady model, which means that a flood hydrograph is run through the dam and waterway. As such the gates are set with with operating parameters to represent how gates would be operated during a flood event. The model starts with the pool at normal elevation and gates closed. As the flows increase and pool elevation rises the gates are fully opened.



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Dam Breach Parameters

The timing and size of the dam breach has a significant impact on the stage and flow increase downstream of the dam. The shorter the time and greater the size, the larger the impact of the breach.

Once the dam was overtopped the 2010 breach took roughly 1.5 hours to fully form (Ref 4)
The extent of the 2010 breach was verified by survey and was roughly 200' wide.

HEC-RAS has the capability to model a dam breach. Inputs for breach modeling are breach geometry, start time, and time to form. There are a variety of equations that have been developed using past breach case studies to estimate breach parameters. 3 methods for predicting breach parameters were checked against the 2010 breach (FERC from Ref 3, Others from Ref 7).

Empirical Equations (English Units)

Breach Parameters	MacDonald & Langridge-Monopolis (1984)	FERC	Froehlich (2008)
Volume Eroded V_{er} (yd^3)	$V_{er} = 3.264BFF^{0.77}$ (best fit all data) $V_{er} = 0.714BFF^{0.852}$ (rockfill)		
Average Breach Width B_{avg} (ft)	$B_{avg} = \frac{V_{er}}{(H_b \times W_{avg})}$	HD $\leq B_{avg} \leq$ 5HD Earthen, Rockfill,	$B_{avg} = 8.239K_o V_w^{0.32} H_b^{0.04}$ K_o=1.0 for piping K_o=1.3 for overtopping
Breach Side slopes Z_b (H:V)	2.0:1	$1/4 \leq Z_b \leq 1$	0.7:1 - piping 1.0:1 - overtopping
Breach Development Time T_f (hr)	$T_f = 0.016V_{er}^{0.364}$	$0.1 \leq T_f \leq 1.0$	$T_f = 3.664\sqrt{\frac{V_w}{gH_b^2}}$

where:

V_w = Full Reservoir Volume
H_w = Full Water Height
H_b = Dam/Breach Height
W_{avg} = Crest Width

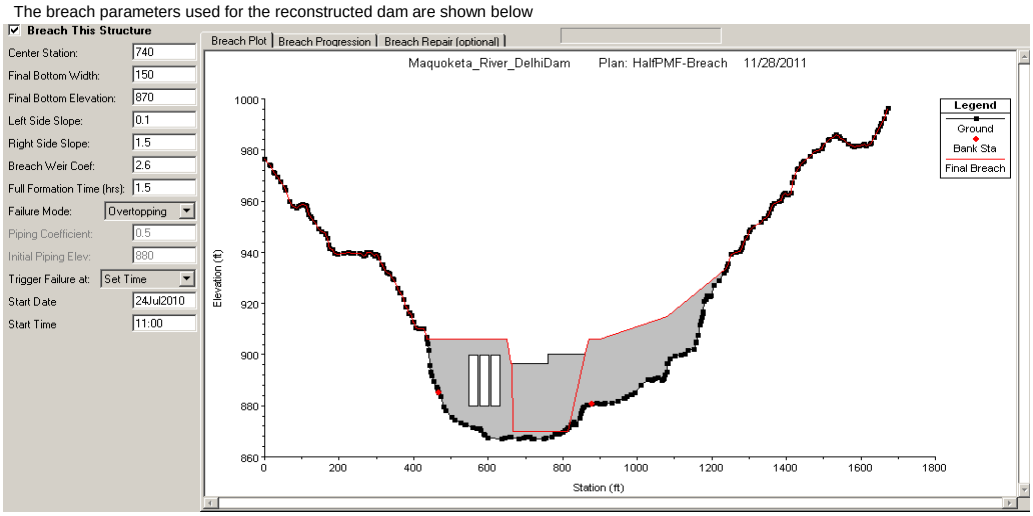
BFF = V_w*H_w (ac-ft^2)

	Delhi
V_w (ac-ft) =	9920
H_w (ft) =	40
H_b (ft) =	35
W_{avg} (ft) =	150

Breach Parameter	Method		
	M&L-M	FERC	Froelich
V_{er}	66781		
B_{avg}	343.4	120	235
Z_b	2	0.5	1
T_f	0.9	0.75	1.8

The Froelich method appears to replicate the parameters of the actual 2010 breach the closest.

The breach parameters used for the reconstructed dam are shown below



For the reconstructed dam the breach parameters were modified slightly to better reflect the proposed condition the breach center was shifted right to center on the service to auxiliary spillway transition. The breach bottom elev. was left as is but the bottom width was shortened to 150', which with a 1.5:1 right side slope gives an average breach width of 175'. This is slightly less than the 200' breach width of 2010, but the geometry fits in better with the proposed condition and still reflects estimated breach parameters. Formation time was left at 1.5 hours. Failure initiation was set to coincide with the peak of the hydrograph which yields the highest flood elevation for the breach scenario.

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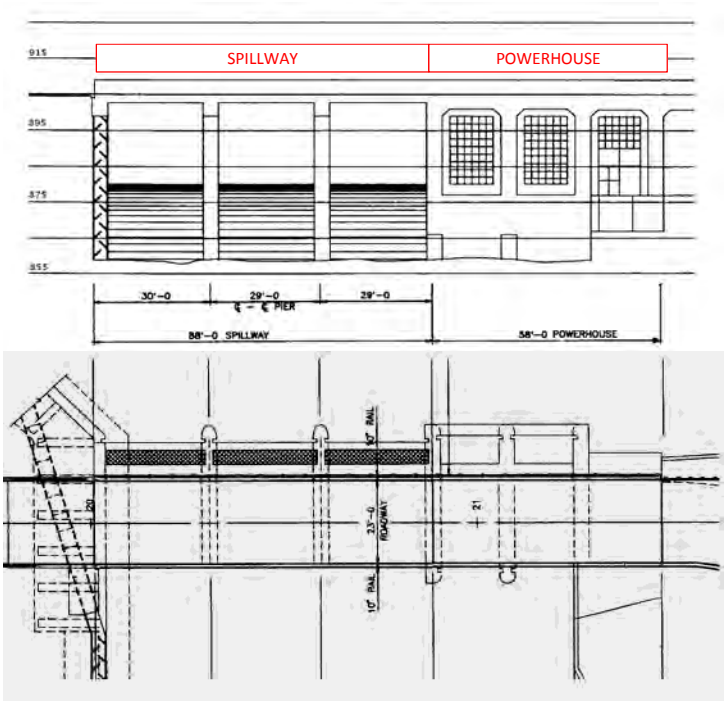
Several forms of failure were considered prior to utilizing the embankment breach. Failure of the powerhouse structure and the spillway structure were also evaluated.

The two most likely failure scenarios of the powerhouse structure would be tipping/sliding of the a section of the structure or an undermining of the foundation and failure/collapse of the above section of structure.



Parameter	Undermine	Tip/Slide
SPILLWAY		
Time to fail	0.5 hrs	0.5 hrs
Width (ft)	88	88
Sides	vertical	vertical
Bottom El.	870	870
POWERHOUSE		
Time to fail	0.5 hrs	0.5 hrs
Width (ft)	58	58
Sides	vertical	vertical
Bottom El.	875	875

Review of reference drawings indicates the powerhouse and spillway are singular structures, although the bridge over the top could be considered an indepedent structure that could remain even with failure of other structures. However to simplify and provide a conservative analysis, both structures were assumed to fully fail, with no debris or remaining structure left in the breach window. The undermine failure was modeled by a "piping failure" in HEC-RAS where the failure starts at the bottom elevation and expands to the top. The Tip/Slide failure was modeled as an overtop where the failure starts at the top and expands to the bottom. Again the failure was timed to occur at the peak of the flood hydrograph.

Flood Profile for 1/2 PMF with Breach

River Station	Distance D/S from Dam (ft)	Undermine		Tip/Slide		Embankment Failure
		Gated Spwy	Powerhouse	Gated Spwy	Powerhouse	
60797	103	892.58	890.92	892.63	890.94	892.82
59617	1,283	891.54	889.89	891.58	889.91	891.8
58696	2,204	889.98	888.42	890.02	888.44	890.3
57576	3,324	888.22	886.81	888.25	886.83	888.61
56760	4,140	885.35	884.12	885.37	884.13	885.95
55202	5,698	883.95	882.83	883.96	882.84	884.57
53880	7,020	883.39	882.25	883.4	882.25	884.04
51564	9,336	880.92	879.89	880.92	879.9	881.58
50035	10,865	880.72	879.68	880.73	879.68	881.41
48596	12,304	879.12	878.17	879.13	878.17	879.76
47429	13,471	878.31	877.39	878.32	877.39	878.93
46486	14,414	878.1	877.16	878.1	877.16	878.71
45029	15,871	876.5	875.62	876.51	875.62	877.05
42827	18,073	874.68	873.86	874.68	873.87	875.14
40097	20,803	873.48	872.69	873.48	872.7	873.9
37851	23,049	872.25	871.52	872.26	871.52	872.61
37165	23,735	871.89	871.18	871.9	871.18	872.22

Results indicate that the embankment failure would have the highest flood profile (i.e. most critical condition)

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HEC-RAS Flood Modeling

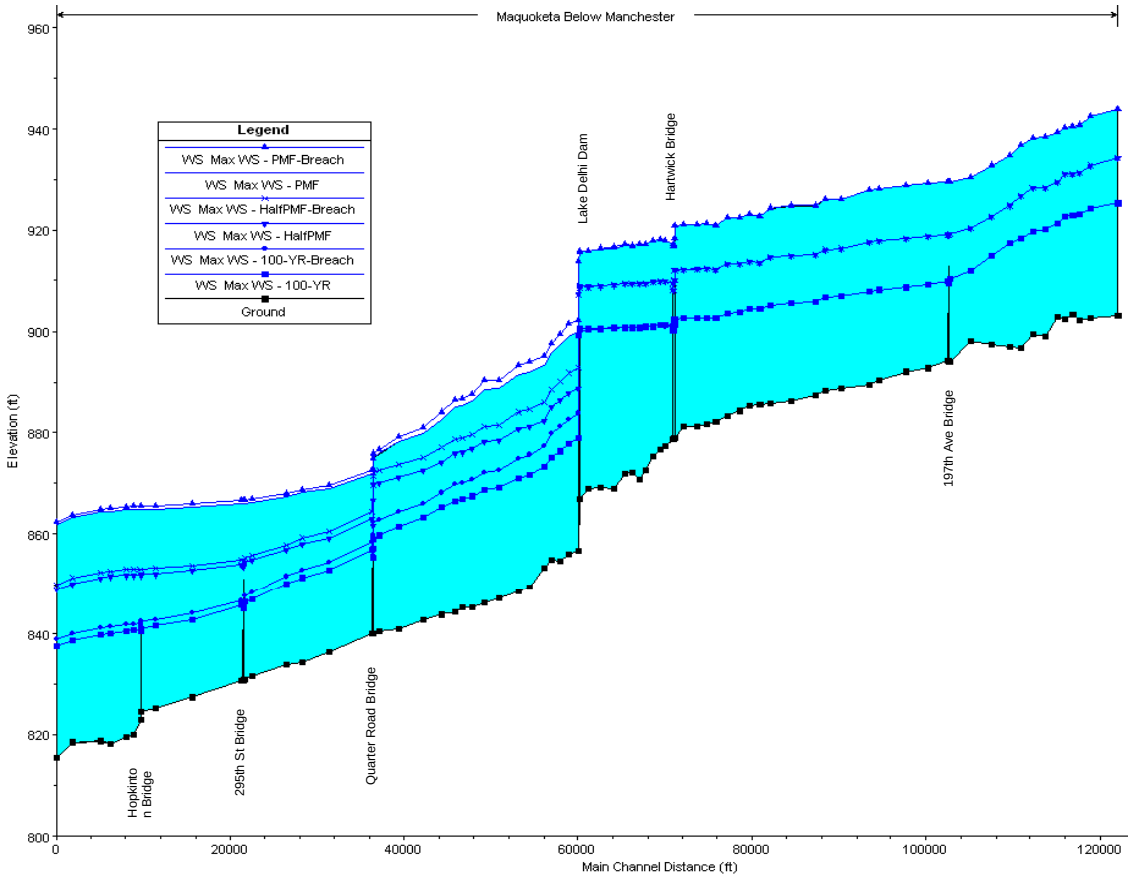
For the hazard class/design flood evaluation, peak flood elevation is the key result. Within HEC-RAS, results can be viewed in several formats. The two most useful for comparing flood scenarios are the river profile and cross-section hydrograph viewer.

The three flood scenarios analyzed initially were

Event	Q_{peak} (cfs)
PMF	143,900
1/2 PMF	71,950
100-YR	31,680

This flow is based off the USGS peak 100-year flow computed for the Manchester Gage, which is 29,610 cfs. To reflect flow at Lake Delhi Dam, this flow was multiplied by the drainage area factor of 1.07 to obtain a 100-year flow of 31,680 cfs at the dam.

The 100-year hydrograph used in HEC-RAS was established by taking the HEC-HMS 100-year hydrograph (developed using the 100-yr/24-hr rainfall) and multiplying by 1.13 to match the peak 100-year flow of 31,860 cfs.



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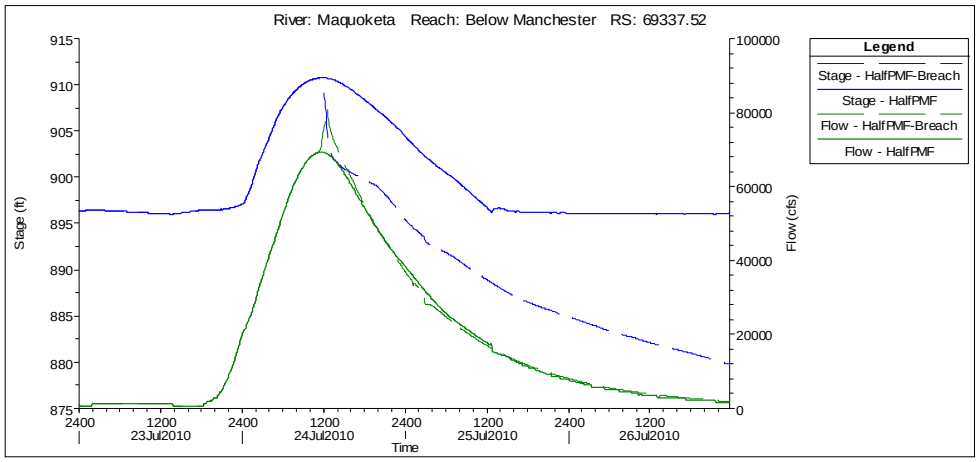
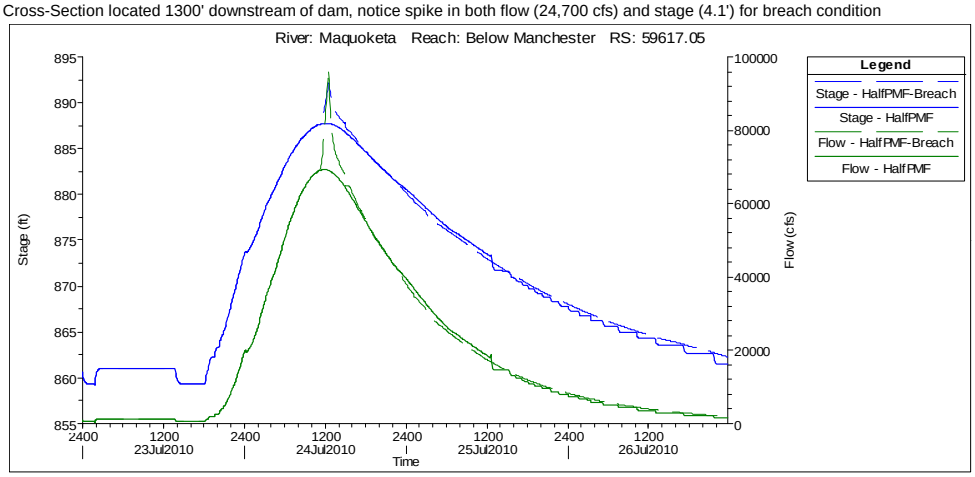
The Max WS profiles indicate that the greatest difference in flood profile for breach vs. non-breach is just downstream of the dam. The breach flood wave dissipates as it flows downstream. The Quarter Road Bridge also provides dissipation of the breach floodwave. Differences in breach vs. non-breach Max WS profiles are less downstream of the bridge. This dissipation can also be viewed in the cross-section stage/flow hydrographs

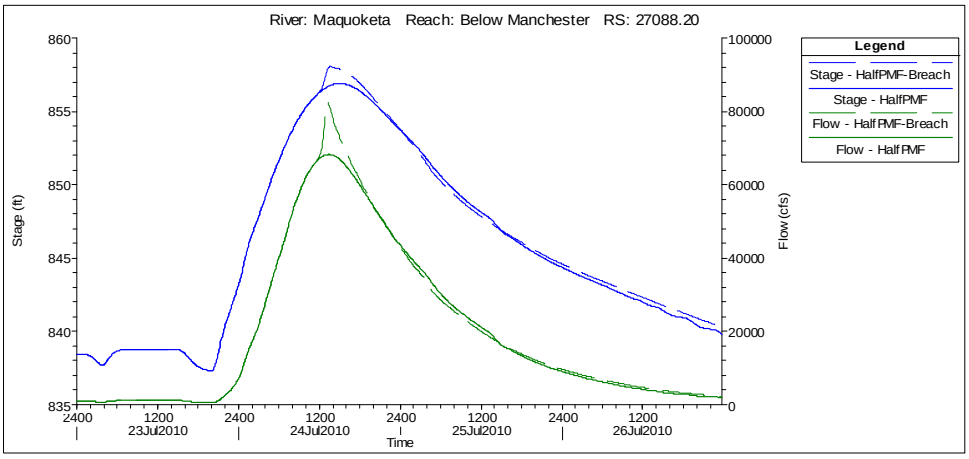
Cross-Section located 8400' upstream of dam, note the drop in WS and spike in flow (10,000 cfs) for the breach scenario.



Cross-Section located 1300' downstream of dam, notice spike in both flow (24,700 cfs) and stage (4.1') for breach condition



Cross-section located 6.4 miles downstream of dam notice dissipated spike in flow (12,300 cfs) and stage (1.0') for breach condition



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Tables summarizing the Flow and Max WS results for the 3 flood conditions are provided on the following pages
River Stations are listed in Upstream to Downstream Order.

HEC-RAS Geometry			PMF					
River Sta	Channel El	Distance (ft)	Q_{peak} (cfs)			Max W.S. Elev (ft)		
			No Breach	Breach	Difference	No Breach	Breach	Difference
122794	903.12	3193	143,285	143,285	0	943.79	943.79	0.00
119601	902.66	1247	143,113	143,113	0	942.39	942.39	0.00
118355	902.13	705	142,854	142,854	0	940.79	940.79	0.00
117650	903.32	866	142,798	142,798	0	940.56	940.56	0.00
116784	902.32	944	142,768	142,768	0	940.42	940.42	0.00
115840	902.85	1476	142,631	142,631	0	939.47	939.47	0.00
114364	898.99	1460	142,497	142,497	0	938.37	938.37	0.00
112904	899.45	1281	142,470	142,470	0	938.21	938.21	0.00
111623	896.76	1232	142,278	142,278	0	936.89	936.89	0.00
110391	896.88	2138	141,909	141,909	0	934.63	934.63	0.00
108252	897.43	2332	141,492	141,492	0	932.9	932.9	0.00
105920	898.14	2522	140,643	140,643	0	930.47	930.47	0.00
103398	893.84	230	140,092	140,092	0	929.44	929.44	0.00
103297					197th Ave Bridge			
103168	894.11	2261	140,144	140,144	0	929.49	929.49	0.00
100907	892.75	2573	140,043	140,043	0	929.22	929.22	0.00
98334	892.14	3014	139,999	139,999	0	928.83	928.83	0.00
95320	890.47	1181	139,928	139,928	0	928.23	928.23	0.00
94139	889.57	3127	139,909	139,909	0	927.85	927.85	0.00
91013	888.71	1851	139,797	139,797	0	926.14	926.14	0.00
89162	888.37	1152	139,778	139,778	0	926	926	0.00
88010	887.45	2752	139,725	139,725	0	924.9	924.9	0.00
85258	886.26	2488	139,706	139,706	0	924.92	924.92	0.00
82771	885.77	1206	139,695	139,695	0	924.3	924.3	0.00
81565	885.49	1142	139,661	139,635	-27	922.71	922.71	0.00
80423	885.18	1135	139,671	139,670	-1	923.24	923.24	0.00
79288	884.41	1483	139,661	139,588	-73	922.62	922.62	0.00
77805	883.49	1240	139,668	139,602	-65	922.47	922.47	0.00
76564	882.22	984	139,644	139,570	-75	920.99	920.99	0.00
75581	881.74	1122	139,649	139,566	-83	921.32	921.32	0.00
74458	881.24	1722	139,643	139,572	-71	921.1	921.11	0.01
72737	881.25	952	139,645	139,569	-76	921.1	921.1	0.00
71785	878.98	151	139,641	139,570	-72	920.78	920.78	0.00
71710					Hartwick Bridge			
71635	878.67	802	139,615	139,569	-46	917.08	917.1	0.02
70833	877.38	669	139,626	139,566	-59	917.92	917.93	0.01
70164	876.67	826	139,625	139,566	-60	918.14	918.15	0.01
69338	875.48	874	139,621	139,548	-73	917.74	917.75	0.01
68463	872.58	743	139,617	139,526	-91	917.12	917.13	0.01
67720	870.83	788	139,620	139,523	-98	917.06	917.07	0.01
66932	872.17	755	139,620	139,497	-123	916.87	916.88	0.01
66177	871.93	1387	139,617	139,494	-124	917.01	917.02	0.01
64790	868.85	1488	139,618	139,462	-156	916.53	916.55	0.02
63302	869.17	1516	139,616	139,431	-185	916.42	916.43	0.01
61786	868.81	783	139,615	139,396	-219	915.92	915.94	0.02
61003	866.93	206	139,615	139,364	-251	916.05	916.06	0.01
60900					Lake Delhi Dam			
60797	856.62	1180	139,586	160,629	21,043	899.92	902.16	2.24
59617	856.09	921	139,580	160,376	20,797	899.13	901.42	2.29
58696	854.52	1119	139,573	159,878	20,305	897.47	899.58	2.11
57576	854.78	816	139,554	159,822	20,269	895.7	897.68	1.98
56760	853.11	1557	139,513	159,407	19,893	893.27	895.01	1.74
55202	849.4	1322	139,503	157,626	18,123	892.1	893.83	1.73
53880	848.44	2316	139,492	157,564	18,072	891.5	893.23	1.73
51564	847.04	1528	139,451	155,601	16,150	888.76	890.41	1.65
50035	846.1	1439	139,456	155,537	16,081	888.65	890.32	1.67
48596	845.13	1168	139,445	155,059	15,614	886.14	887.69	1.55
47429	845.27	943	139,444	154,851	15,407	885.21	886.67	1.46
46486	844.35	1457	139,440	154,850	15,410	885	886.47	1.47
45029	843.77	2202	139,436	154,701	15,265	882.74	884.03	1.29
42827	842.98	2729	139,430	154,415	14,985	879.97	881.02	1.05
40097	841.02	2246	139,425	154,141	14,716	878.28	879.22	0.94
37851	840.48	686	139,422	154,102	14,681	876	876.66	0.66
37165	839.98	170	139,422	154,088	14,666	875.3	875.86	0.56
37080					Quarter Road Bridge			
36995	840.16	4909	135,554	152,346	16,791	871.84	872.68	0.84
32085	836.41	3037	131,454	138,369	6,916	868.9	869.48	0.58
29049	834.42	1961	130,095	135,201	5,105	868.07	868.66	0.59
27088	834.04	3792	129,017	133,684	4,667	867.42	868.02	0.60
23296	831.71	864	127,553	131,734	4,181	866.32	866.95	0.63
22433	830.86	449	127,351	131,380	4,029	866.09	866.73	0.64
22180					295th Street Bridge			
21984	830.78	5750	127,283	131,380	4,097	866.06	866.7	0.64
16234	827.37	4078	126,742	130,627	3,884	865.3	865.96	0.66
12156	825.22	1731	126,606	130,463	3,857	864.94	865.61	0.67
10425	824.55	109	126,540	130,422	3,882	864.84	865.51	0.67
10370					Hopkinton Bridge			
10316	823.07	643	126,568	130,455	3,887	864.84	865.51	0.67
9673	819.82	871	126,562	130,447	3,885	864.82	865.49	0.67
8802	819.52	1849	126,539	130,418	3,878	864.77	865.45	0.68
6953	818.29	1218	126,534	130,397	3,863	864.53	865.2	0.67
5735	818.68	3207	126,526	130,395	3,869	864.32	864.99	0.67
2528	818.58	1826	126,525	130,393	3,868	863.16	863.83	0.67
702	815.37	702	126,524	130,391	3,867	861.75	862.41	0.66

Stanley Consultants INC.

Computed by	A. Judd	Date	9/28/2011	Job No.	23601
Checked by	M. Weber	Date	10/12/2011	Subject	Lake Delhi Dam
Approved by		Date			Hydrology/Hydraulics
					Computation Summary

HEC-RAS Geometry			Half PMF					
River Sta	Channel El	Distance (ft)	Q_{peak} (cfs)			Max W.S. Elev (ft)		
			No Breach	Breach	Difference	No Breach	Breach	Difference
122794	903.12	3193	71,700	71,700	0	934.19	934.19	0.00
119601	902.66	1247	71,624	71,624	0	932.91	932.91	0.00
118355	902.13	705	71,535	71,535	0	931.4	931.4	0.00
117650	903.32	866	71,524	71,524	0	931.26	931.26	0.00
116784	902.32	944	71,502	71,502	0	931.1	931.1	0.00
115840	902.85	1476	71,423	71,423	0	929.51	929.51	0.00
114364	898.99	1460	71,358	71,358	0	928.44	928.44	0.00
112904	899.45	1281	71,357	71,357	0	928.26	928.26	0.00
111623	896.76	1232	71,269	71,269	0	926.74	926.74	0.00
110391	896.88	2138	71,111	71,111	0	924.8	924.8	0.00
108252	897.43	2332	70,803	70,803	0	922.79	922.79	0.00
105920	898.14	2522	70,243	70,243	0	920.36	920.36	0.00
103398	893.84	230	69,872	69,872	0	919.17	919.17	0.00
103297			197th Ave Bridge					
103168	894.11	2261	69,821	69,821	0	919.09	919.09	0.00
100907	892.75	2573	69,736	69,736	0	918.77	918.77	0.00
98334	892.14	3014	69,655	69,655	0	918.34	918.34	0.00
95320	890.47	1181	69,598	69,598	0	917.91	917.91	0.00
94139	889.57	3127	69,597	69,597	0	917.65	917.65	0.00
91013	888.71	1851	69,495	69,494	-1	916.44	916.44	0.00
89162	888.37	1152	69,493	69,492	-2	916.27	916.27	0.00
88010	887.45	2752	69,405	69,433	28	915.24	915.24	0.00
85258	886.26	2488	69,401	69,446	45	915.14	915.14	0.00
82771	885.77	1206	69,372	69,439	67	914.73	914.73	0.00
81565	885.49	1142	69,347	69,435	87	913.7	913.7	0.00
80423	885.18	1135	69,354	69,430	76	913.95	913.94	-0.01
79288	884.41	1483	69,332	69,427	95	913.4	913.39	-0.01
77805	883.49	1240	69,332	69,417	86	913.33	913.32	-0.01
76564	882.22	984	69,318	69,386	68	912.23	912.22	-0.01
75581	881.74	1122	69,317	69,382	65	912.44	912.42	-0.02
74458	881.24	1722	69,317	69,362	45	912.34	912.32	-0.02
72737	881.25	952	69,315	69,336	21	912.3	912.28	-0.02
71785	878.98	151	69,312	69,333	20	912.1	912.08	-0.02
71710			Hartwick Bridge					
71635	878.67	802	69,282	69,255	-27	909.45	909.42	-0.03
70833	877.38	669	69,290	69,250	-40	909.88	909.85	-0.03
70164	876.67	826	69,286	69,247	-39	909.96	909.94	-0.02
69338	875.48	874	69,282	69,212	-70	909.76	909.73	-0.03
68463	872.58	743	69,282	69,208	-74	909.51	909.48	-0.03
67720	870.83	788	69,279	69,168	-111	909.42	909.39	-0.03
66932	872.17	755	69,284	69,165	-119	909.33	909.3	-0.03
66177	871.93	1387	69,283	69,121	-162	909.37	909.33	-0.04
64790	868.85	1488	69,281	69,072	-208	909.14	909.1	-0.04
63302	869.17	1516	69,279	69,024	-255	909.02	908.98	-0.04
61786	868.81	783	69,279	68,971	-308	908.82	908.78	-0.04
61003	866.93	206	69,278	68,921	-357	908.85	908.81	-0.04
60900			Lake Delhi Dam					
60797	856.62	1180	69,242	94,152	24,910	888.74	892.82	4.08
59617	856.09	921	69,226	93,950	24,723	887.75	891.8	4.05
58696	854.52	1119	69,216	93,757	24,541	886.4	890.3	3.90
57576	854.78	816	69,204	93,356	24,151	885.03	888.61	3.58
56760	853.11	1557	69,154	92,912	23,758	882.57	885.95	3.38
55202	849.4	1322	69,114	91,109	21,995	881.37	884.57	3.20
53880	848.44	2316	69,098	91,026	21,928	880.77	884.04	3.27
51564	847.04	1528	69,032	87,871	18,839	878.55	881.58	3.03
50035	846.1	1439	69,030	87,810	18,780	878.31	881.41	3.10
48596	845.13	1168	68,996	86,933	17,937	876.87	879.76	2.89
47429	845.27	943	68,982	86,658	17,676	876.14	878.93	2.79
46486	844.35	1457	68,975	86,638	17,663	875.88	878.71	2.83
45029	843.77	2202	68,956	86,007	17,052	874.35	877.05	2.70
42827	842.98	2729	68,923	85,435	16,511	872.54	875.14	2.60
40097	841.02	2246	68,905	85,226	16,321	871.29	873.9	2.61
37851	840.48	686	68,902	85,120	16,218	870.06	872.61	2.55
37165	839.98	170	68,902	85,110	16,209	869.7	872.22	2.52
37080			Quarter Road Bridge					
36995	840.16	4909	68,697	84,251	15,554	862.95	864.52	1.57
32085	836.41	3037	67,965	81,349	13,385	859.19	860.44	1.25
29049	834.42	1961	67,493	80,245	12,752	858.01	859.15	1.14
27088	834.04	3792	66,942	79,206	12,264	856.91	857.92	1.01
23296	831.71	864	65,703	71,331	5,628	854.81	855.71	0.90
22433	830.86	449	65,437	70,627	5,190	854.41	855.33	0.92
22180			295th Street Bridge					
21984	830.78	5750	65,033	69,906	4,873	853.89	854.87	0.98
16234	827.37	4078	64,374	68,874	4,499	852.61	853.66	1.05
12156	825.22	1731	64,214	68,614	4,400	852.06	853.14	1.08
10425	824.55	109	64,195	68,563	4,368	851.9	852.99	1.09
10370			Hopkinton Bridge					
10316	823.07	643	64,181	68,563	4,382	851.85	852.94	1.09
9673	819.82	871	64,178	68,559	4,381	851.81	852.91	1.10
8802	819.52	1849	64,174	68,555	4,380	851.75	852.84	1.09
6953	818.29	1218	64,158	68,537	4,379	851.45	852.55	1.10
5735	818.68	3207	64,161	68,535	4,373	851.24	852.33	1.09
2528	818.58	1826	64,157	68,529	4,372	850.23	851.32	1.09
702	815.37	702	64,155	68,528	4,372	848.88	849.96	1.08

Stanley Consultants INC.

Computed by	A. Judd	Date	9/28/2011	Job No.	23601
Checked by	M. Weber	Date	10/12/2011	Subject	Lake Delhi Dam
Approved by		Date			Hydrology/Hydraulics
					Computation Summary

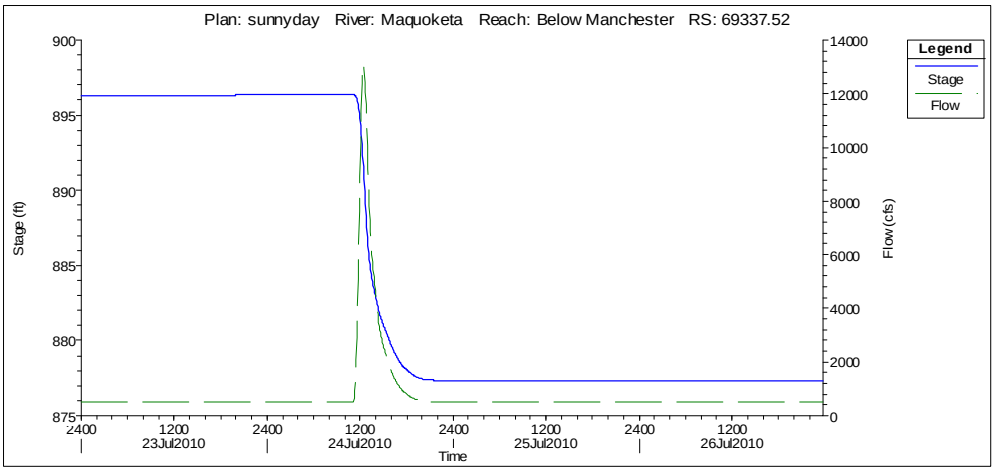
HEC-RAS Geometry			100-YR					
River Sta	Channel El	Distance (ft)	Q_{peak} (cfs)			Max W.S. Elev (ft)		
			No Breach	Breach	Difference	No Breach	Breach	Difference
122794	903.12	3193	31,699	31,699	0	925.41	925.41	0.00
119601	902.66	1247	31,687	31,687	0	924.34	924.34	0.00
118355	902.13	705	31,667	31,667	0	923.14	923.14	0.00
117650	903.32	866	31,663	31,663	0	922.88	922.88	0.00
116784	902.32	944	31,661	31,661	0	922.68	922.68	0.00
115840	902.85	1476	31,649	31,649	0	921.29	921.29	0.00
114364	898.99	1460	31,639	31,639	0	920.06	920.06	0.00
112904	899.45	1281	31,638	31,638	0	919.82	919.82	0.00
111623	896.76	1232	31,632	31,632	0	918.39	918.39	0.00
110391	896.88	2138	31,626	31,626	0	917.31	917.31	0.00
108252	897.43	2332	31,580	31,580	0	914.95	914.95	0.00
105920	898.14	2522	31,444	31,444	0	912.07	912.07	0.00
103398	893.84	230	31,231	31,231	0	910.26	910.26	0.00
103297					197th Ave Bridge			
103168	894.11	2261	31,096	31,096	0	909.82	909.82	0.00
100907	892.75	2573	30,967	30,967	0	909.22	909.22	0.00
98334	892.14	3014	30,899	30,899	0	908.65	908.65	0.00
95320	890.47	1181	30,868	30,867	-1	908.21	908.21	0.00
94139	889.57	3127	30,848	30,860	12	908	908	0.00
91013	888.71	1851	30,812	30,813	0	907.04	907.04	0.00
89162	888.37	1152	30,794	30,808	13	906.72	906.72	0.00
88010	887.45	2752	30,757	30,803	45	905.91	905.91	0.00
85258	886.26	2488	30,737	30,800	64	905.59	905.59	0.00
82771	885.77	1206	30,730	30,794	64	905.22	905.22	0.00
81565	885.49	1142	30,702	30,785	83	904.46	904.45	-0.01
80423	885.18	1135	30,711	30,783	73	904.46	904.44	-0.02
79288	884.41	1483	30,683	30,771	89	903.75	903.73	-0.02
77805	883.49	1240	30,682	30,762	80	903.63	903.6	-0.03
76564	882.22	984	30,646	30,733	87	902.58	902.54	-0.04
75581	881.74	1122	30,655	30,721	66	902.72	902.68	-0.04
74458	881.24	1722	30,654	30,704	50	902.65	902.6	-0.05
72737	881.25	952	30,653	30,683	30	902.56	902.51	-0.05
71785	878.98	151	30,650	30,659	9	902.39	902.34	-0.05
71710					Hartwick Bridge			
71635	878.67	802	30,623	30,586	-37	900.94	900.88	-0.06
70833	877.38	669	30,626	30,583	-43	901.14	901.08	-0.06
70164	876.67	826	30,629	30,580	-48	901.17	901.11	-0.06
69338	875.48	874	30,625	30,552	-73	901.05	900.98	-0.07
68463	872.58	743	30,625	30,522	-103	900.94	900.87	-0.07
67720	870.83	788	30,623	30,520	-103	900.85	900.78	-0.07
66932	872.17	755	30,622	30,489	-133	900.78	900.71	-0.07
66177	871.93	1387	30,624	30,488	-136	900.78	900.71	-0.07
64790	868.85	1488	30,622	30,421	-200	900.65	900.58	-0.07
63302	869.17	1516	30,620	30,384	-236	900.52	900.44	-0.08
61786	868.81	783	30,619	30,344	-275	900.43	900.35	-0.08
61003	866.93	206	30,619	30,305	-314	900.43	900.35	-0.08
60900					Lake Delhi Dam			
60797	856.62	1180	30,612	51,053	20,442	878.96	883.94	4.98
59617	856.09	921	30,608	50,306	19,698	877.76	882.75	4.99
58696	854.52	1119	30,603	50,174	19,571	876.28	881.2	4.92
57576	854.78	816	30,599	49,765	19,166	875.14	879.9	4.76
56760	853.11	1557	30,583	49,470	18,886	873.28	877.38	4.10
55202	849.4	1322	30,568	48,656	18,088	871.78	875.76	3.98
53880	848.44	2316	30,558	48,159	17,601	871	874.98	3.98
51564	847.04	1528	30,531	46,175	15,644	869.08	872.63	3.55
50035	846.1	1439	30,519	45,688	15,169	868.63	872.23	3.60
48596	845.13	1168	30,506	45,159	14,653	867.42	870.79	3.37
47429	845.27	943	30,504	44,687	14,183	866.89	870.14	3.25
46486	844.35	1457	30,501	44,663	14,162	866.45	869.72	3.27
45029	843.77	2202	30,494	44,258	13,764	865.29	868.24	2.95
42827	842.98	2729	30,473	42,349	11,876	863.18	865.97	2.79
40097	841.02	2246	30,452	40,997	10,545	861.33	864.31	2.98
37851	840.48	686	30,448	40,949	10,501	859.68	862.84	3.16
37165	839.98	170	30,448	40,922	10,475	859.07	862.32	3.25
37080					Quarter Road Bridge			
36995	840.16	4909	30,430	40,689	10,259	856.85	858.48	1.63
32085	836.41	3037	30,304	38,885	8,581	852.81	854.25	1.44
29049	834.42	1961	30,240	38,329	8,090	851.26	852.75	1.49
27088	834.04	3792	30,206	38,123	7,917	850.06	851.5	1.44
23296	831.71	864	30,070	37,554	7,484	846.87	848.19	1.32
22433	830.86	449	30,031	37,449	7,417	846.27	847.54	1.27
22180					295th Street Bridge			
21984	830.78	5750	29,904	37,024	7,119	845.57	846.7	1.13
16234	827.37	4078	29,284	33,965	4,681	842.91	844.06	1.15
12156	825.22	1731	28,944	32,815	3,871	841.59	842.91	1.32
10425	824.55	109	28,893	32,646	3,753	841.11	842.53	1.42
10370					Hopkinton Bridge			
10316	823.07	643	28,848	32,483	3,635	840.76	842.06	1.30
9673	819.82	871	28,831	32,457	3,626	840.65	841.97	1.32
8802	819.52	1849	28,829	32,437	3,608	840.56	841.89	1.33
6953	818.29	1218	28,814	32,422	3,608	840.15	841.5	1.35
5735	818.68	3207	28,815	32,412	3,597	839.9	841.26	1.36
2528	818.58	1826	28,811	32,402	3,591	838.75	840.15	1.40
702	815.37	702	28,810	32,402	3,591	837.48	838.88	1.40

Stanley Consultants INC.

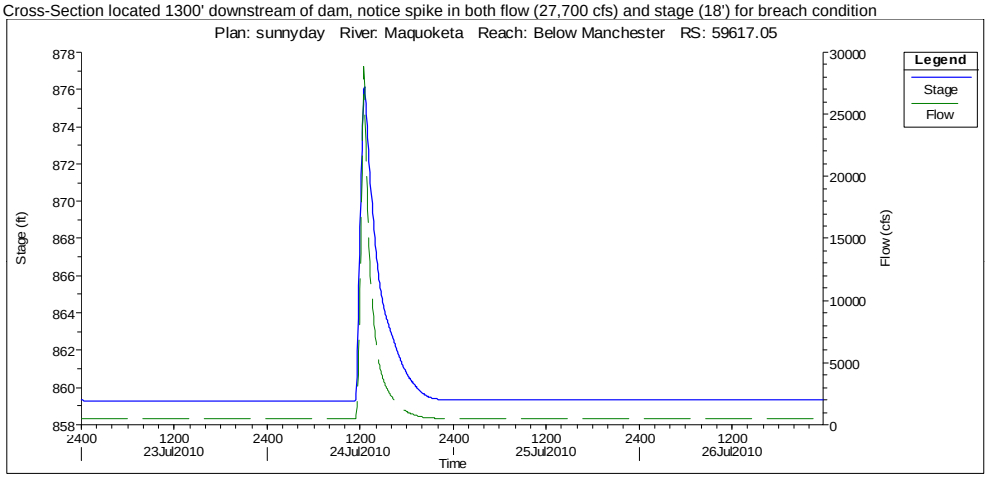
Computed by	A. Judd	Date	9/28/2011	Job No.	23601
Checked by	M. Weber	Date	10/12/2011	Subject	Lake Delhi Dam
Approved by		Date			Hydrology/Hydraulics
					Computation Summary

A Sunny Day Breach was also included in the analysis. This represents a dam failure during normal flow conditions. The HEC-RAS model was set to the normal pool with a constant flow of 500 cfs with the dam breach/failure set to occur midway through the model's time period. The hydrograph results are shown below.

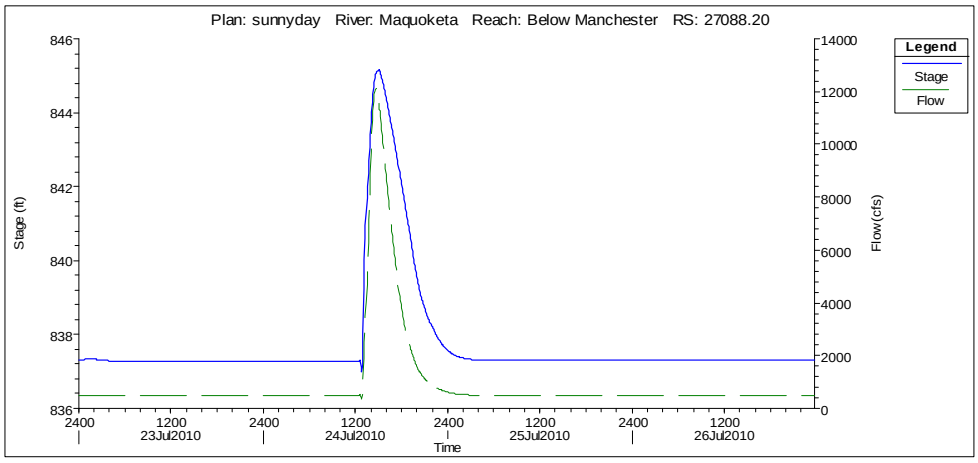
Cross-Section located 8400' upstream of dam, note the drop in WS and spike in flow (13,000 cfs) for the breach scenario.



Cross-Section located 1300' downstream of dam, notice spike in both flow (27,700 cfs) and stage (18') for breach condition



Cross-section located 6.4 miles downstream of dam notice dissipated spike in flow (12,300 cfs) and stage (8') for breach condition



Several breach scenarios were analyzed for the Sunny Day, both a piping failure (failure starts within the embankment) and an overtopping failure (failure starts at the top of the embankment) were used with a 1.5 hour and 1 hour breach formation time. The piping failure resulted in a lower Max W.S. and the 1 hour vs. 1.5 hour did not make an appreciable difference in Max W.S., so the breach/failure was left consistent between the Sunny Day and Flood events.

Something to note, the spike in flow was roughly the same between Sunny Day and Half PMF, but the stage increase for the Sunny Day was much greater due to the narrower channel width at the bottom. This quick and significant increase could be of concern if structures or facilities are located within the Sunny Day Breach's inundation limits. A Table of HEC-RAS results is provided.

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Computed by	A. Judd	Date	9/28/2011	Job No.	23601
Checked by	M. Weber	Date	10/12/2011	Subject	Lake Delhi Dam
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HEC-RAS Geometry			Sunny Day Breach			
River Sta	Channel El	Distance (ft)	Q_{peak} (cfs)	Max W.S. (ft) No Breach	Max W.S. (ft) Breach	Difference
122794	903.12	3193	500	908.44	908.44	0.0
119601	902.66	1247	500	908.27	908.27	0.0
118355	902.13	705	500	908.14	908.14	0.0
117650	903.32	866	500	907.98	907.98	0.0
116784	902.32	944	500	907.83	907.83	0.0
115840	902.85	1476	500	906.32	906.32	0.0
114364	898.99	1460	458	904.17	904.17	0.0
112904	899.45	1281	510	904.01	904.01	0.0
111623	896.76	1232	502	903.88	903.88	0.0
110391	896.88	2138	520	903.83	903.83	0.0
108252	897.43	2332	518	902.73	902.73	0.0
105920	898.14	2522	507	899.8	899.81	0.0
103398	893.84	230	523	897.23	897.22	0.0
103297				197th Ave Bridge		
103168	894.11	2261	523	897.14	897.12	0.0
100907	892.75	2573	499	896.71	896.64	-0.1
98334	892.14	3014	499	896.53	896.43	-0.1
95320	890.47	1181	498	896.49	896.39	-0.1
94139	889.57	3127	498	896.49	896.39	-0.1
91013	888.71	1851	497	896.48	896.38	-0.1
89162	888.37	1152	497	896.48	896.37	-0.1
88010	887.45	2752	497	896.48	896.37	-0.1
85258	886.26	2488	496	896.47	896.37	-0.1
82771	885.77	1206	496	896.47	896.37	-0.1
81565	885.49	1142	495	896.47	896.36	-0.1
80423	885.18	1135	495	896.47	896.36	-0.1
79288	884.41	1483	495	896.47	896.36	-0.1
77805	883.49	1240	494	896.47	896.36	-0.1
76564	882.22	984	494	896.47	896.36	-0.1
75581	881.74	1122	494	896.47	896.36	-0.1
74458	881.24	1722	494	896.47	896.36	-0.1
72737	881.25	952	494	896.47	896.36	-0.1
71785	878.98	151	493	896.47	896.36	-0.1
71710				HartwickBridge		
71635	878.67	802	493	896.46	896.36	-0.1
70833	877.38	669	493	896.46	896.36	-0.1
70164	876.67	826	492	896.46	896.36	-0.1
69338	875.48	874	492	896.46	896.36	-0.1
68463	872.58	743	492	896.46	896.36	-0.1
67720	870.83	788	492	896.46	896.36	-0.1
66932	872.17	755	492	896.46	896.36	-0.1
66177	871.93	1387	491	896.46	896.36	-0.1
64790	868.85	1488	491	896.46	896.36	-0.1
63302	869.17	1516	490	896.46	896.36	-0.1
61786	868.81	783	490	896.46	896.36	-0.1
61003	866.93	206	490	896.46	896.36	-0.1
60900				Lake Delhi Dam		
60797	856.62	1180	29,244	859.93	877.44	17.5
59617	856.09	921	28,027	859.35	876.04	16.7
58696	854.52	1119	27,193	859.02	874.29	15.3
57576	854.78	816	27,124	857.47	872.94	15.5
56760	853.11	1557	26,465	855.49	870.6	15.1
55202	849.4	1322	25,007	852.96	868.53	15.6
53880	848.44	2316	23,841	851.79	867.28	15.5
51564	847.04	1528	20,444	850.31	864.93	14.6
50035	846.1	1439	19,662	849.46	864.29	14.8
48596	845.13	1168	18,699	848.52	862.92	14.4
47429	845.27	943	18,558	848.03	862.42	14.4
46486	844.35	1457	18,309	847.7	861.9	14.2
45029	843.77	2202	17,917	847.17	860.89	13.7
42827	842.98	2729	16,590	845.91	858.59	12.7
40097	841.02	2246	15,614	844.23	856.67	12.4
37851	840.48	686	15,472	843.46	855.19	11.7
37165	839.98	170	15,466	843.1	854.57	11.5
37080				Quarter Road Bridge		
36995	840.16	4909	15,460	842.9	853.48	10.6
32085	836.41	3037	13,234	839.3	848.67	9.4
29049	834.42	1961	12,185	838.21	846.59	8.4
27088	834.04	3792	11,298	837.33	845.22	7.9
23296	831.71	864	11,072	834.2	841.95	7.8
22433	830.86	449	10,662	833.64	841.41	7.8
22180				295th Street Bridge		
21984	830.78	5750	10,662	833.37	840.9	7.5
16234	827.37	4078	9,055	830.31	837.9	7.6
12156	825.22	1731	9,079	829.53	836.16	6.6
10425	824.55	109	9,143	827.19	832.61	5.4
10370				Hopkinton Bridge		
10316	823.07	643	9,168	827.12	832.68	5.6
9673	819.82	871	8,944	827.08	832.38	5.3
8802	819.52	1849	8,898	827.07	832.24	5.2
6953	818.29	1218	8,817	827.05	831.75	4.7
5735	818.68	3207	8,779	827.03	831.44	4.4
2528	818.58	1826	8,736	821.77	829.34	7.6
702	815.37		8,731	818.26	827.74	9.5

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					Computation Summary

The time for the peak flood stage to travel downstream was computed by determining the time difference between peaks between cross-sections. Travel times at HEC-RAS model cross-sections are shown in the following table.

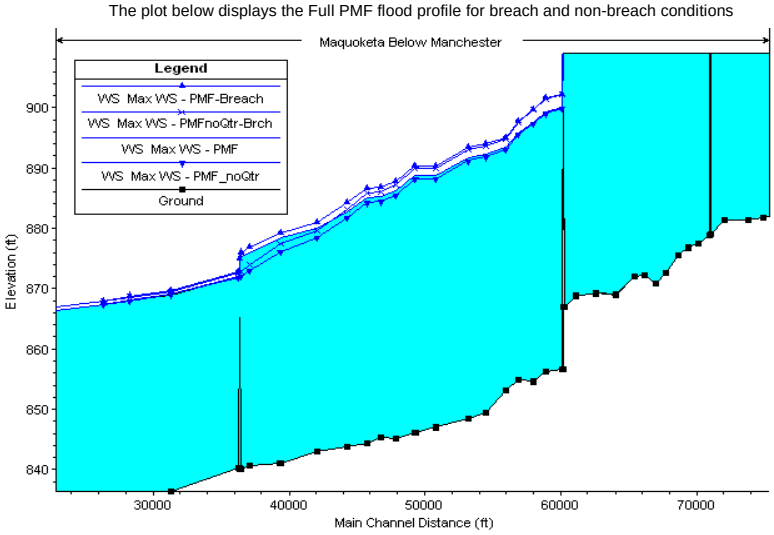
Maquoketa River Station	Distance Downstream (mi)	Peak Flood Stage Travel Time (Hours)					
		PMF		1/2 PMF		100 YR	
		No Breach	Breach	No Breach	Breach	No Breach	Breach
60900	Lake Delhi Dam						
60797	0.0	0.0	0.0	0.0	0.0	0.0	0.0
59617	0.2	0.0	0.0	0.1	0.0	0.0	0.0
58696	0.4	0.0	0.0	0.1	0.0	0.1	0.0
57576	0.6	0.1	0.0	0.2	0.0	0.1	0.0
56760	0.8	0.1	0.0	0.2	0.0	0.1	0.0
55202	1.1	0.2	0.1	0.3	0.1	0.3	0.1
53880	1.3	0.2	0.1	0.3	0.1	0.3	0.1
51564	1.7	0.3	0.2	0.5	0.2	0.5	0.2
50035	2.0	0.3	0.2	0.5	0.2	0.5	0.2
48596	2.3	0.3	0.2	0.6	0.3	0.6	0.3
47429	2.5	0.3	0.2	0.7	0.3	0.7	0.3
46486	2.7	0.3	0.2	0.7	0.3	0.7	0.3
45029	3.0	0.3	0.3	0.8	0.3	0.8	0.3
42827	3.4	0.3	0.3	0.8	0.3	0.8	0.4
40097	3.9	0.3	0.3	0.8	0.3	0.9	0.6
37851	4.3	0.4	0.3	0.8	0.4	1.0	0.6
37165	4.5	0.4	0.3	0.8	0.4	1.0	0.6
37080	Quarter Road Bridge						
36995	4.5	2.3	0.5	1.5	0.5	1.3	0.7
32085	5.4	3.2	1.8	2.3	0.8	1.8	1.2
29049	6.0	3.5	2.2	2.6	1.0	2.0	1.3
27088	6.4	3.7	2.4	2.9	1.1	2.1	1.4
23296	7.1	4.0	2.8	3.7	2.6	2.6	1.8
22433	7.3	4.1	2.8	3.8	2.8	2.8	1.8
22180	295th Street Bridge						
21984	7.4	4.1	2.8	4.1	3.1	3.0	2.0
16234	8.4	4.3	3.0	4.4	3.4	4.1	3.2
12156	9.2	4.3	3.0	4.6	3.5	4.6	3.6
10425	9.5	4.3	3.1	4.6	3.6	4.7	3.8
10370	Hopkinton Bridge						
10316	9.6	4.3	3.1	4.6	3.6	4.8	3.8
9673	9.7	4.3	3.1	4.6	3.6	4.8	3.8
8802	9.8	4.3	3.1	4.6	3.6	4.8	3.8
6953	10.2	4.3	3.1	4.6	3.6	4.8	3.9
5735	10.4	4.3	3.1	4.6	3.6	4.8	3.9
2528	11.0	4.3	3.1	4.7	3.6	4.9	3.9
702	11.4	4.3	3.1	4.7	3.6	4.9	3.9

Overall, the breach peak stage (flood wave) traveled downstream more rapidly than the no breach condition flood wave. Once the breach occurs, HEC-RAS shows it would take between 3-4 hours for the breach floodwave to reach Hopkinton. The Quarter Road bridge appears to provide significant attenuation of the floodwave.

Impact of Quarter Road Bridge

The HEC-RAS flood hydrographs/profiles indicated the Quarter Road Bridge has a significant impact on water surface (backwater) el. upstream of the bridge. The bridge is overtopped for both the Full and Half PMF but not the 100YR (both breach and non-breach) so the impact of a potential failure of the bridge was evaluated by removing the bridge from the HEC-RAS model and comparing bridge and no bridge conditions.

The plot below displays the Full PMF flood profile for breach and non-breach conditions



Max. W.S. Difference (Breach - NonBreach)		
River Sta	Qtr Rd Bridge	No Qtr Rd
60900	Lake Delhi Dam	
60797	2.24	2.28
59617	2.29	2.33
58696	2.11	2.14
57576	1.98	2.02
56760	1.74	1.79
55202	1.73	1.79
53880	1.73	1.78
51564	1.65	1.74
50035	1.67	1.75
48596	1.55	1.65
47429	1.46	1.58
46486	1.47	1.6
45029	1.29	1.46
42827	1.05	1.27
40097	0.94	1.21
37851	0.66	0.94
37165	0.56	0.8
37080	Quarter Road Bridge	
36995	0.84	0.81
32085	0.58	0.58
29049	0.59	0.58
27088	0.6	0.59
23296	0.63	0.62
22433	0.64	0.63

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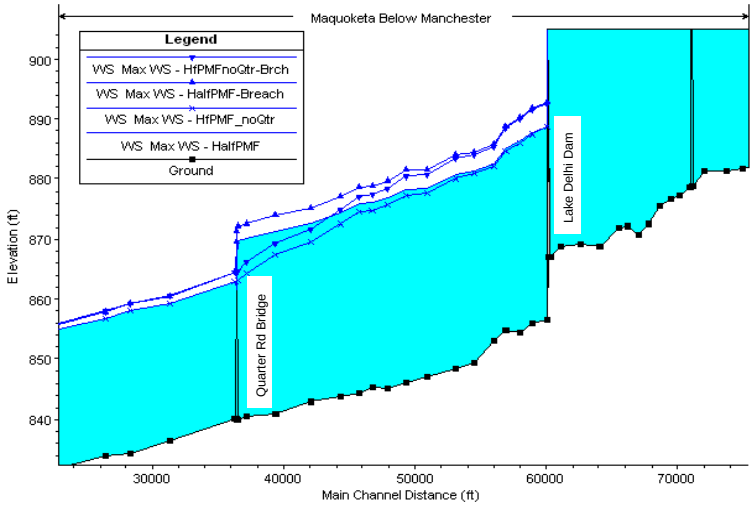
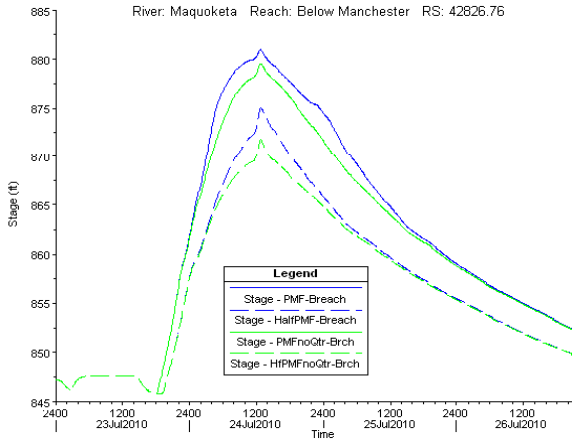
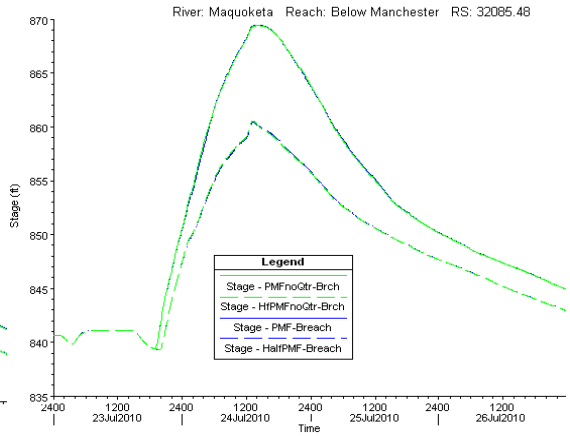
The plot below displays the Half PMF flood profile for breach and non-breach conditions



Max. W.S. Difference (Breach - NonBreach)		
River Sta	Qtr Rd Bridge	No Qtr Rd
60900	Lake Delhi Dam	
60797	4.08	4.15
59617	4.05	4.14
58696	3.9	4
57576	3.58	3.68
56760	3.38	3.4
55202	3.2	3.26
53880	3.27	3.33
51564	3.03	3.05
50035	3.1	3.12
48596	2.89	2.87
47429	2.79	2.7
46486	2.83	2.76
45029	2.7	2.49
42827	2.6	2.21
40097	2.61	2.13
37851	2.55	1.83
37165	2.52	1.71
37080	Quarter Road Bridge	
36995	1.57	1.68
32085	1.25	1.37
29049	1.14	1.26
27088	1.01	1.14
23296	0.9	0.98
22433	0.92	0.97

Timing of the peak stage was also reviewed for the No Quarter Road Bridge model. Removal (failure) of the Quarter Road Bridge did not impact the timing of the peak of the floodwave significantly. As shown in the stage hydrographs below, river segments just upstream of the Quarter Road bridge saw the greatest impact to stage and a slight impact to timing of the peak, but once downstream of the bridge, the impact was minimal.





Results show that a failure of the Quarter Road Bridge during a flood event would decrease the overall flood elevation and would reduce the potential increase in flood elevation due to breach for floods of a Half PMF magnitude. For floods of a full PMF magnitude failure of the Quarter Road Bridge would likely add to the potential increase in flood elevation due to breach, HOWEVER the flood profile would drop by roughly 1.5' with failure of the bridge so the overall flood profile would still be lower for a full PMF with failure of the Quarter Road Bridge.

So the analysis indicates that failure Quarter Road Bridge does not increase the downstream hazard.

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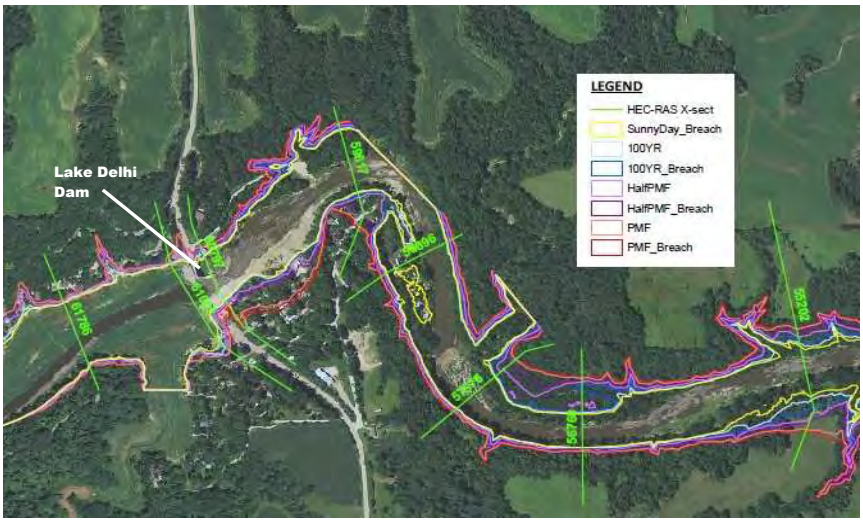
Inudation Mapping

HEC-RAS results are exported to ArcGIS to develop inundation limits for each flood condition (using HEC-GeoRAS).

HEC-GeoRAS establishes inundation limits by the following process

- Results of HEC-RAS are read and each river cross-section in ArcGIS is assigned a Max W.S. Elev for a given flood.
- A 3-D surface of the Max W.S. is created in ArcGIS along the River alignment (i.e. a 3-D version of the Max WS profile).
- The Max W.S. is then laid over the LiDAR topography of the river valley and shapefiles and rasters are created representing the inundation limits and depths, respectively for the given flood.

Note that upstream of the dam the inundation limits are identical for breach/non breach conditions. Downstream of the dam the breach condition causes a rise in flood elevation which translates to an expanded inundation area.



The results of the inundation mapping are provided in a set of maps following these computations.

Hazard Classification

Hazard Classification involves reviewing the properties/structures inundated for the various flood scenarios and evaluating the increase in hazard (i.e. damage/safety risk) caused by failure of the dam during that flood event. Theoretically, the less increase in flood profile, the less increase in hazard. But inundation maps provide a more quantifiable method for evaluating what additional properties and structures would be impacted by a breach during a flood event.

Due to their location and lack of potential warning time, the homes directly downstream of the dam were examined in closer detail for the theoretical breach (failure) event.
Flood impacts to individual homes downstream of the dam are provided in the following tables.

Property				PMF Event				
				No Breach		Breach		Depth
Street Number	Street Name	F.F. Elev.	Dist. from Dam	W.S. Elev.	Flood Depth	W.S. Elev.	Flood Depth	Increase
2636	230th Ave	901.9	150	899.8	0.0	902.1	0.2	0.2
2638	230th Ave	913.7	150	899.8	0.0	902.1	0.0	0.0
23082	263rd St	914.7	340	899.7	0.0	901.9	0.0	0.0
23089	263rd St	914.4	200	899.8	0.0	902.0	0.0	0.0
23094	263rd St	906.1	340	899.7	0.0	901.9	0.0	0.0
23099	263rd St	912.1	200	899.8	0.0	902.0	0.0	0.0
23102	263rd St	903.4	340	899.7	0.0	901.9	0.0	0.0
23105	263rd St	905.9	200	899.8	0.0	902.0	0.0	0.0
23110	263rd St	898.7	340	899.7	1.0	901.9	3.2	2.3
23111	263rd St	902.4	200	899.8	0.0	902.0	0.0	0.0
23116	263rd St	895.3	340	899.7	4.4	901.9	6.6	2.3
23119	263rd St	901.3	200	899.8	0.0	902.0	0.7	0.7
23124	263rd St	896.4	340	899.7	3.3	901.9	5.5	2.3
23128	263rd St	893.3	340	899.7	6.4	901.9	8.6	2.3
23129	263rd St	895.9	200	899.8	3.9	902.0	6.1	2.2
23137	263rd St	891.3	340	899.7	8.4	901.9	10.6	2.3
23157	263rd St	892.4	430	899.6	7.2	901.9	9.5	2.3
23162	263rd St	896.7	430	899.6	0.0	901.9	0.0	0.0
23168	263rd St	904.7	490	899.6	2.9	901.9	5.2	2.3
23181	263rd St	893.3	490	899.6	6.3	901.9	8.6	2.3
23049	264th St	924.8	150	899.8	0.0	902.1	0.0	0.0
23077	264th St	917	200	899.8	0.0	902.0	0.0	0.0
23105	264th St	911.9	430	899.6	0.0	901.9	0.0	0.0
23168	264th St	904.7	430	899.6	0.0	901.9	0.0	0.0
23133	264th St	905.2	490	899.6	0.0	901.9	0.0	0.0
23157	264th St	899.6	540	899.6	0.0	901.8	2.2	2.2
23172	264th St	883.1	630	899.5	16.4	901.8	18.7	2.3
26287	231st Ave	889.1	1180	899.1	10.0	901.4	12.3	2.3
26294	231st Ave	894.4	1180	899.1	4.7	901.4	7.0	2.3
26299	231st Ave	892.4	1180	899.1	6.7	901.4	9.0	2.3
26269	232nd Ave	879.8	1180	899.1	19.3	901.4	21.6	2.3

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					Computation Summary

Property				Half PMF Event				
				No Breach		Breach		Depth
Street Number	Street Name	F.F. Elev.	Dist. from Dam	W.S. Elev.	Flood Depth	W.S. Elev.	Flood Depth	Increase
2636	230th Ave	901.9	150	888.6	0.0	892.7	0.0	0.0
2638	230th Ave	913.7	150	888.6	0.0	892.7	0.0	0.0
23082	263rd St	914.7	340	888.5	0.0	892.5	0.0	0.0
23089	263rd St	914.4	200	888.6	0.0	892.6	0.0	0.0
23094	263rd St	906.1	340	888.5	0.0	892.5	0.0	0.0
23099	263rd St	912.1	200	888.6	0.0	892.6	0.0	0.0
23102	263rd St	903.4	340	888.5	0.0	892.5	0.0	0.0
23105	263rd St	905.9	200	888.6	0.0	892.6	0.0	0.0
23110	263rd St	898.7	340	888.5	0.0	892.5	0.0	0.0
23111	263rd St	902.4	200	888.6	0.0	892.6	0.0	0.0
23116	263rd St	895.3	340	888.5	0.0	892.5	0.0	0.0
23119	263rd St	901.3	200	888.6	0.0	892.6	0.0	0.0
23124	263rd St	896.4	340	888.5	0.0	892.5	0.0	0.0
23128	263rd St	893.3	340	888.5	0.0	892.5	0.0	0.0
23129	263rd St	895.9	200	888.6	0.0	892.6	0.0	0.0
23137	263rd St	891.3	340	888.5	0.0	892.5	1.2	1.2
23157	263rd St	892.4	430	888.4	0.0	892.4	0.0	0.0
23162	263rd St	896.7	430	888.4	0.0	892.4	0.0	0.0
23168	263rd St	904.7	490	888.3	0.0	892.4	0.0	0.0
23181	263rd St	893.3	490	888.3	0.0	892.4	0.0	0.0
23049	264th St	924.8	150	888.6	0.0	892.7	0.0	0.0
23077	264th St	917	200	888.6	0.0	892.6	0.0	0.0
23105	264th St	911.9	430	888.4	0.0	892.4	0.0	0.0
23168	264th St	904.7	430	888.4	0.0	892.4	0.0	0.0
23133	264th St	905.2	490	888.3	0.0	892.4	0.0	0.0
23157	264th St	899.6	540	888.3	0.0	892.4	0.0	0.0
23172	264th St	883.1	630	888.2	5.1	892.3	9.2	4.1
26287	231st Ave	889.1	1180	887.8	0.0	891.8	2.7	2.7
26294	231st Ave	894.4	1180	887.8	0.0	891.8	0.0	0.0
26299	231st Ave	892.4	1180	887.8	0.0	891.8	0.0	0.0
26269	232nd Ave	879.8	1180	887.8	8.0	891.8	12.0	4.0

Property				100YR Flood Event				
				No Breach		Breach		Depth
Street Number	Street Name	F.F. Elev.	Dist. from Dam	W.S. Elev.	Flood Depth	W.S. Elev.	Flood Depth	Increase
2636	230th Ave	901.9	150	878.8	0.0	883.8	0.0	0.0
2638	230th Ave	913.7	150	878.8	0.0	883.8	0.0	0.0
23082	263rd St	914.7	340	878.6	0.0	883.6	0.0	0.0
23089	263rd St	914.4	200	878.8	0.0	883.7	0.0	0.0
23094	263rd St	906.1	340	878.6	0.0	883.6	0.0	0.0
23099	263rd St	912.1	200	878.8	0.0	883.7	0.0	0.0
23102	263rd St	903.4	340	878.6	0.0	883.6	0.0	0.0
23105	263rd St	905.9	200	878.8	0.0	883.7	0.0	0.0
23110	263rd St	898.7	340	878.6	0.0	883.6	0.0	0.0
23111	263rd St	902.4	200	878.8	0.0	883.7	0.0	0.0
23116	263rd St	895.3	340	878.6	0.0	883.6	0.0	0.0
23119	263rd St	901.3	200	878.8	0.0	883.7	0.0	0.0
23124	263rd St	896.4	340	878.6	0.0	883.6	0.0	0.0
23128	263rd St	893.3	340	878.6	0.0	883.6	0.0	0.0
23129	263rd St	895.9	200	878.8	0.0	883.7	0.0	0.0
23137	263rd St	891.3	340	878.6	0.0	883.6	0.0	0.0
23157	263rd St	892.4	430	878.5	0.0	883.5	0.0	0.0
23162	263rd St	896.7	430	878.5	0.0	883.5	0.0	0.0
23168	263rd St	904.7	490	878.5	0.0	883.4	0.0	0.0
23181	263rd St	893.3	490	878.5	0.0	883.4	0.0	0.0
23049	264th St	924.8	150	878.8	0.0	883.8	0.0	0.0
23077	264th St	917	200	878.8	0.0	883.7	0.0	0.0
23105	264th St	911.9	430	878.5	0.0	883.5	0.0	0.0
23168	264th St	904.7	430	878.5	0.0	883.5	0.0	0.0
23133	264th St	905.2	490	878.5	0.0	883.4	0.0	0.0
23157	264th St	899.6	540	878.4	0.0	883.4	0.0	0.0
23172	264th St	883.1	630	878.3	0.0	883.3	0.2	0.2
26287	231st Ave	889.1	1180	877.8	0.0	882.8	0.0	0.0
26294	231st Ave	894.4	1180	877.8	0.0	882.8	0.0	0.0
26299	231st Ave	892.4	1180	877.8	0.0	882.8	0.0	0.0
26269	232nd Ave	879.8	1180	877.8	0.0	882.8	3.0	3.0

The two critical items being evaluated for these properties are

- Number of additional properties being flooded due to breach
- Increase in depth of flooding at properties being impacted

	Full PMF	Half PMF	100YR Flood
Non-Breach Flooded	13	2	0
Additional by Breach	3	2	2
Greatest Increase in Flood Depth	2.3'	4.1'	3.0'

Results indicate the PMF is a hazardous condition without breach and the breach would impact an additional 3 homes with the 2.3' increase in flood level. The half PMF without breach impacts 2 homes, which would increase to 4 with the 4.1' increase in flood level. The 100 YR flood does not impact any downstream homes, but results show if a breach occurs two downstream homes could be flooded by the breach flood wave.

For increase in flood hazard due to breach, the 100YR has a definite increase in hazard due to the fact that prior to breach no homes are impacted and then with breach the flood level jumps by 5'. The flooding due to the full PMF is widespread and the breach increases the flood level by 2.3' which is significant but relative to the flooding already occurring likely does not constitute as great of an increase in flood hazard as at the Half PMF when flood levels are also high but then increase by over 4' due to a breach event. Qualitatively the Half PMF appears to provide the greatest increase in downstream hazard potential due to the larger increase in flood depth that could catch more properties if the downstream area is further developed.

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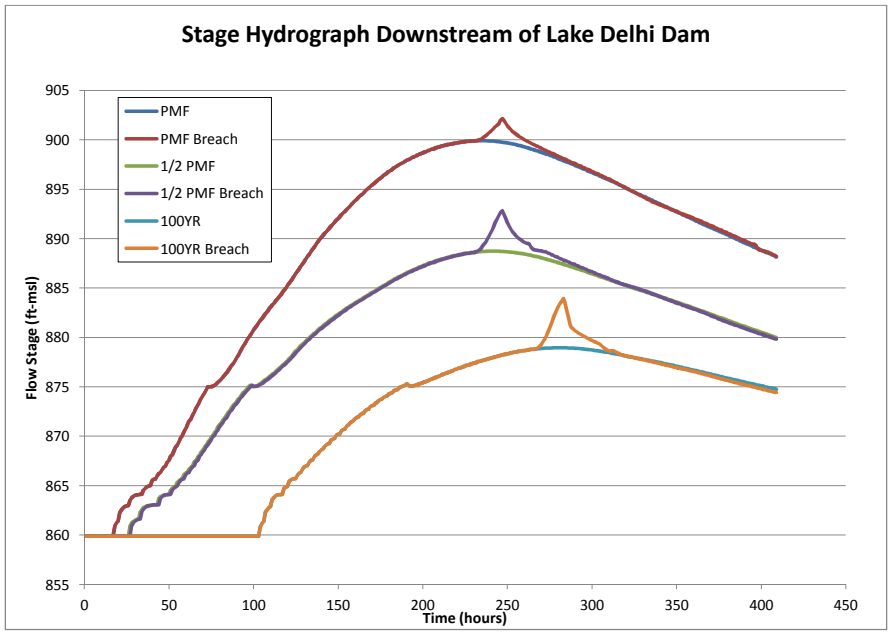
Computed by	A. Judd	Date	9/28/2011	Job No.	23601
Checked by	M. Weber	Date	10/12/2011	Subject	Lake Delhi Dam
Approved by		Date			Hydrology/Hydraulics
					Computation Summary

The graph below displays the stage hydrograph during the theoretical design floods downstream of the proposed dam. The rise is roughly 1.5 ft/hr with slight variations between events. With the breach, the peak flood/flow occurs within roughly an hour of the initiation of breach, so the rise would be roughly 2 ft/hr for the PMF and 4 ft/hr with the 1/2 PMF during breach.



	RISE (ft/hr)
PMF	1.7
1/2 PMF	1.3
100YR	1.5

Downstream impacts were also evaluated for the entire downstream reach of the Maquoketa River from the Dam to just downstream of Hopkinton. The table below provides a summary of the structures impacted for the flood events. Structures were inventoried using the inundation maps referencing 2011 aerial photos. The summary table is based off of a detailed inundation table where structures were counted cross-section to cross-section.

Buildings and Infrastructure Inundated					
Event	Scenario	Residential	Comm/Ag	Bridges	Roads
PMF	No Breach	104	30	3	12
	Breach	107	30	3	12
Half PMF	No Breach	27	8	3	8
	Breach	29	8	3	8
100-YR	No Breach	3	1	1	5
	Breach	5	2	1	5
Sunny Day	Breach	0	0	0	1

Dam breach during the flood events does not appear to cause a significant increase in structures inundated. The majority of additional structures inundated are the homes wihin 1500' of the dam which should be evacuated during extreme flood events.

Downstream of the near-dam residential area, homes and buildings that are inundated for the breach condition are generally also inundated for the non-breach condition. This can be seen on the inundation maps provided.

The Sunny Day Breach did not appear to cause significant hazard to downstream permanent structures.

Hazard classification is based on the additional hazard created by failure of the dam. So by this criteria, given the number of additional buildings inundated over the range of failure events, and the low density, rural setting of the majority of the downstream channel, Lake Delhi Dam is best classified as a

MODERATE/SIGNIFICANT HAZARD STRUCTURE

DNR

Low Hazard. Structures located in areas where damages from a failure would be limited to loss of the dam, loss of livestock, damages to farm out-buildings, agricultural lands, and lesser used roads, and where loss of human life is considered unlikely.

Moderate Hazard. Structures located in areas where failure may damage isolated homes or cabins, industrial or commercial buildings, moderately traveled roads or railroads, interrupt major utility services, but without substantial risk of loss of human life.

High Hazard. Structures located in areas where failure may create a serious threat of loss of human life or result in serious damage to residential, industrial or commercial areas, important public utilities, public buildings, or major transportation facilities.

FERC

Dams conforming to criteria for the low hazard potential category generally are located in rural or agricultural areas where failure may damage farm buildings, limited agricultural land, or township and country roads. Low hazard potential dams have a small storage capacity, the release of which would be confined to the river channel in the event of a failure and therefore would represent no danger to human life.

Significant hazard potential category structures are usually located in predominately rural or agricultural areas where failure may damage isolated homes, secondary highways or minor railroads; cause interruption of use or service of relatively important public utilities; or cause some incremental flooding of structures with possible danger to human life.

Dams in the high hazard potential category are those located where failure may cause serious damage to homes, agricultural, industrial and commercial facilities, important public utilities, main highways, or railroads, and there would be danger to human life.

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Checked by	M. Weber	Date	10/12/2011	Subject	Lake Delhi Dam
Approved by		Date			Hydrology/Hydraulics
					Computation Summary

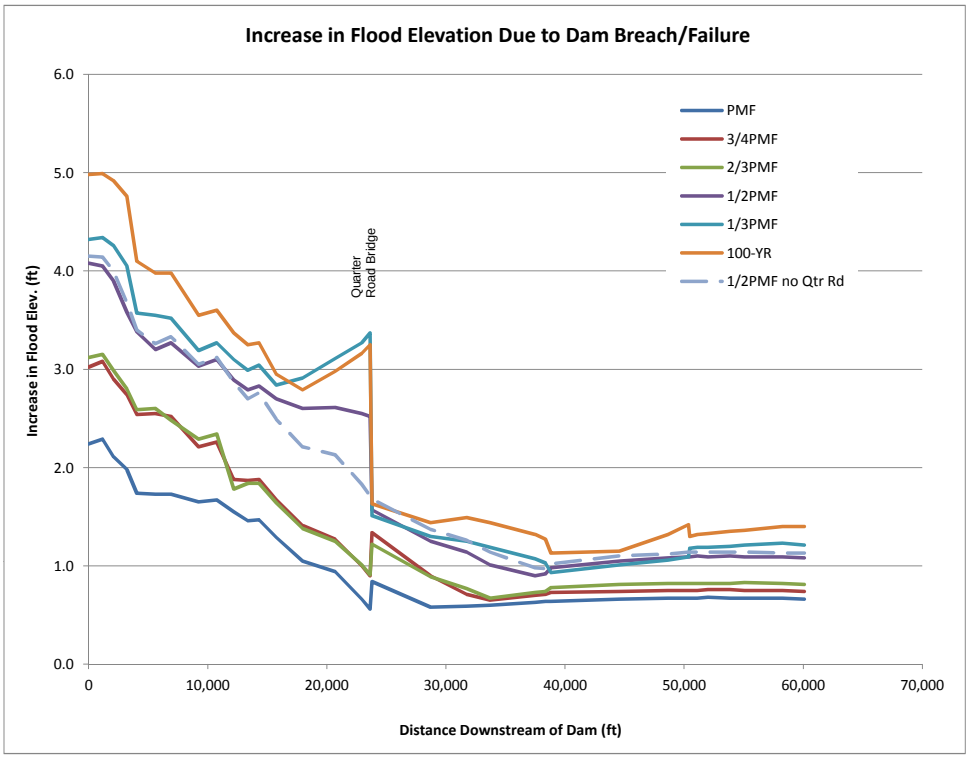
Design Flood

The DNR specifies that a Moderate Hazard dam be able to safely pass the Half PMF for its design flood.

FERC uses an incremental analysis to determine the largest flood where failure causes an increase in downstream hazard.

From the inundation analysis, dam failure did not suggest a significant increase in downstream hazard, mostly due to the lack of structures located in the river valley and the minimal difference in additional structures inundated by the dam's failure/breach during the given flood. The exception is homes immediately downstream of the dam, which do experience additional inundation/hazard due to dam failure. It is assumed that these homes would be evacuated during an extreme flood.

To further analyze the incremental hazard, the rise in flood elevation due to breach was evaluated for a series of floods.



As illustrated in the graph above, the 100YR results in the greatest increase in flood elevation due to breach. This increase is over 5 feet just downstream of the dam and diminishes to less than 2 feet by Hopkinton. FERC recommends using 2 feet as the minimum flood rise where hazard potential is considered to be impacted by breach, however downstream of the immediate homes at the dam, there are no buildings inundated by any of the flood events until the flood is past the Quarter Road Bridge. By the Quarter Road Bridge, the rise due to breach for floods greater than the 1/2 PMF has decreased to less than 2 feet. Floods less than the1/2 PMF have a rise greater than 2 feet at the bridge, but the bridge dissipates the rise due to breach downstream.

To analyze the impact of the bridge a HEC-RAS run was developed with the Quarter Road Bridge removed. This is also shown on the graph. HEC-RAS shows that the bridge causes a rise in flood elevation due to breach upstream of the bridge. Removing the bridge causes a slight increase in flood elevation due to breach downstream of the bridge, but the bridge looks to have more impact on the upstream flood elevation increases.

So given that the greatest increases in flood elevation due to breach are from floods of a lesser magnitude than the1/2 PMF, using the **1/2 PMF** as the design flood should also meet FERC criteria.

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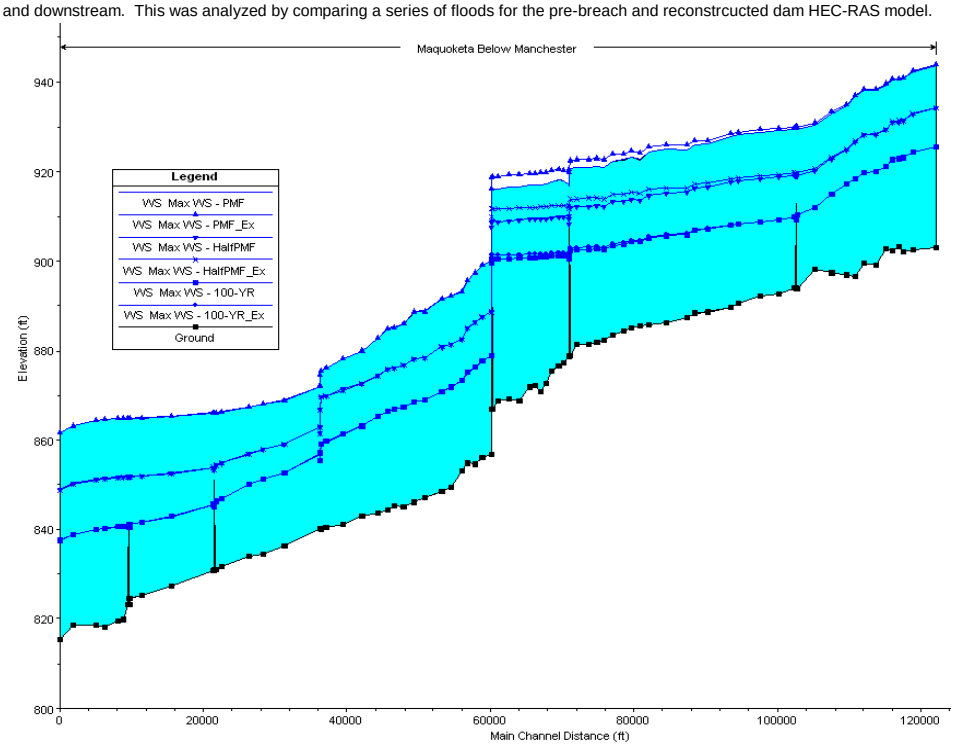
Checked by	M. Weber	Date	10/12/2011	Subject	Lake Delhi Dam
Approved by		Date			Hydrology/Hydraulics
					Computation Summary

Upstream/Downstream Impacts

Hazard classification compares a failure to non-failure condition. Another important comparison is the impact of the reconstructed dam relative to the pre-breach condition. The objective is to provide a reconstructed dam that minimizes negative impacts upstream and downstream. This was analyzed by comparing a series of floods for the pre-breach and reconstrcucted dam HEC-RAS model.



Without the principal/auxiliary spillway and gate system fully defined this is a preliminary evaluation, so will require updating once the auxiliary spillway and gates are designed for reconstruction.

As can be expected with an increase in hydraulic capacity, the reconstructed dam provides lower peak flood elevations upstream, with minimal increase in flood elevation downstream. So HEC-RAS results indicate that the reconstructed dam will improve upstream flood conditions by

The following table provides flood elevation differences at a series of HEC-RAS cross-sections

Stretch of Maquoketa River	HEC-RAS X-SECT ID	Change in Flood Elev Due w/ New Dam		
		PMF (ft)	1/2 PMF (ft)	100-YR (ft)
Upstream of 197th Ave Bridge	122794	-0.06	-0.03	0
	103398	-0.59	-0.64	-0.04
Between 197th Ave Bridge and Hartwick Bridge	103168	-0.6	-0.65	-0.06
	100907	-0.61	-0.69	-0.08
	94139	-0.71	-0.81	-0.12
	82771	-1.08	-1.23	-0.27
	77805	-1.32	-1.49	-0.43
	71785	-1.61	-1.76	-0.58
Between Hartwick Bridge and Lake Delhi Dam	71635	-2.65	-2.73	-0.77
	67720	-2.65	-2.75	-0.79
	63302	-2.83	-2.87	-0.85
	61003	-2.9	-2.92	-0.87
Between Lake Delhi Dam and Quarter Road Bridge	60797	0.05	0.06	0.11
	56760	0.04	0.05	0.11
	50035	0.05	0.06	0.11
	45029	0.03	0.06	0.1
	37165	0.02	0.07	0.1
Between Quarter Road Bridge and 295th Street Bridge	36995	0.05	0.05	0.06
	27088	0.06	0.07	0.07
	22433	0.07	0.09	0.06
Between 295th Street Bridge and Hopkinton Bridge	21984	0.07	0.1	0.05
	10425	0.07	0.11	0.06
Downstream of Hopkinton Bridge	10316	0.07	0.12	0.07
	702	0.07	0.11	0.07

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				Job No.	23601
Computed by	A. Judd	Date	11/10/2011	Subject	Lake Delhi Dam
Checked by	M Weber	Date	12/9/2011		Hydraulics
Approved by		Date			Spillway

Description:

Summary of spillway hydraulic analysis for Lake Delhi reconstruction project.

Reference:

(1) Iowa DNR, ***Design Criteria and Guidelines for Iowa Dams***, T.B. 16, 1990.
(2) USBR, ***Design of Small Dams,*** 1987.
(3) Chow Ven Te; ***Open Channel Hydraulics***; McGraw-Hill, 1958.
(4) Henry T. Falvey, ***Hydraulic Design of Labyrinth Weirs***, ASCE Press, 2003.

Analysis:

Lake Delhi Dam is classified as a moderate hazard structure. Per Ref 1, the dam must pass the 1/2 PMF safely. Hydrologic analysis has established the 1/2 PMF as roughly 70,000 cfs at the dam. The following analysis evaluates the hydraulics of spillway alternatives for the reconstructed dam to pass the 1/2 PMF safely.

Option 1 **Use two, tiered labyrinth weirs + gates**

A labyrinth weir spillway consists of a sharp-crested weir (e.g. concrete wall) set in a series of trapezoidal folds which fits a longer crest length into a shorter spillway breadth.

Several methods are shown in Ref 4 for estimating flow over a Labyrinth Spillway
This analysis uses the recommended method proposed by Tullis in 1995.
The geometry (i.e. variables) of a Labyrinth Spillway is given by the following

Flow over a Labyrinth Spillway is given by:

$$Q_L = C_D \cdot L_c \cdot \frac{2}{3} \cdot \sqrt{2g} \cdot H^{1.5}$$ Ref 4 5.15

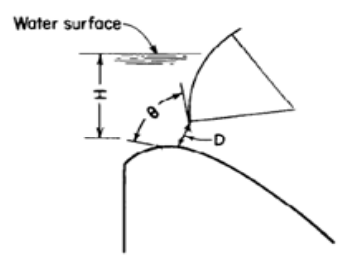
where:

$$C_D = A_1 + A_2\left(\frac{H}{P}\right) + A_3\left(\frac{H}{P}\right)^2 + A_4\left(\frac{H}{P}\right)^3 + A_5\left(\frac{H}{P}\right)^4$$ (1) 5.16

The parameters are defined by the following table (1) table 5.1

a	A_1	A_2	A_3	A_4	A_5
6	0.49	-0.24	-1.20	2.17	-1.03
8	0.49	1.08	-5.27	6.79	-2.83
12	0.49	1.06	-4.43	5.18	-1.97
15	0.49	1.00	-3.57	3.82	-1.38
18	0.49	1.32	-4.13	4.24	-1.50
25	0.49	1.51	-3.83	3.40	-1.05
35	0.49	1.69	-4.05	3.62	-1.10
90	0.49	1.46	-2.56	1.44	0.00

Flow through Gates are given by:



EQUATION FOR DISCHARGE

$Q = CDL\sqrt{2gH}$

D = Net gate opening
L = Crest width
H = Head to center of gate opening
C = 0.68

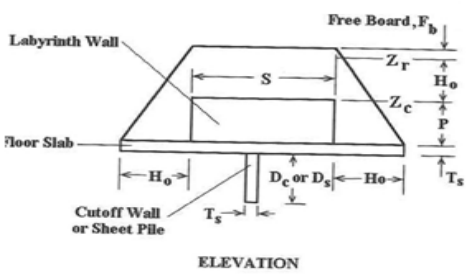
Existing spillway gates at Lake Delhi

Crest Elev.	879.8 ft
# of Gates	3
Open Width	25.0 ft
Open Height	20.0 ft
C	0.68

DESIGN PARAMETER	LABYRINTH SPILLWAY		
	PRINCIPAL	AUX.	EQUATION
Design Pool	904	904	
Crest Elev.	896.3	900	
Floor Elev.	886.3	892	
Wc	124	110	
α	25	15	
a	1.5	1.5	
n	2	4	
$T_{weir\ wall}$	1.5	1.5	
$T_{end\ wall}$	1.5	1.5	
T_s	2	2	
H_0 (ft)	7.7	4	(4) 8.1
P (ft)	10	8	El_{crest}-El_{floor}
W (ft)	62.0	27.5	(4) 8.5
L (ft)	138.5	89.1	(4) 5.2
L_c (ft)	277.0	356.3	(4) 8.6
B (ft)	66.3	41.5	(4) 8.8
S (ft)	60.0	40.1	(1) 8.9
A_1	0.49	0.49	Table 5.1
A_2	1.51	1	Table 5.1
A_3	-3.83	-3.57	Table 5.1
A_4	3.4	3.82	Table 5.1
A_5	-1.05	-1.38	Table 5.1
PARAMETER CHECK			
$H_0/P < 0.7$	0.8	0.5	
$2 < L/W < 9.5$	2.2	3.2	
$L_{de}/B \leq 0.35$	0.14	0.20	

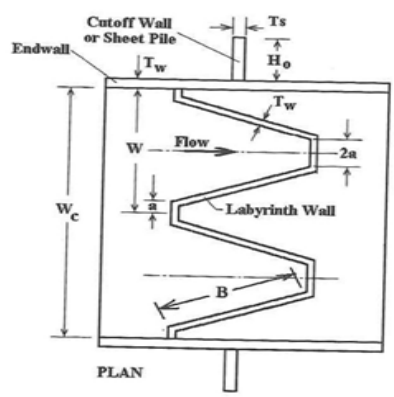
$$\frac{L_{de}}{B} = \frac{h}{B} \cdot 6.1 \cdot e^{-0.052 \cdot \alpha}$$

h is the head at the weir, which is close to critical depth (~2/3H)
0.75*H is used for a conservative ***h***



L = crest length of weir in 1 cycle

L_c = total crest length of weir



Spillway Width
$W_c = n*W$
n = No. Cycles
(2 Cycles are shown)

Gates assumed completely open

POOL ELEVATION (ft-msl)	GATE FLOW 3 GATES H_{gates} (ft)	Q_{gates} (cfs)	SPILLWAY FLOW SERVICE H_{princ} (ft)	C_D	Q_{princ} (cfs)	AUX. H_{aux} (ft)	C_D	Q_{aux} (cfs)	TOTAL Q_{total} (cfs)
896	16.2	17,116	0	0.00	0	0	0.0	0	17,116
896.2	16.4	18,430	0	0.00	0	0	0.00	0	18,430
896.4	16.6	17,247	0.1	0.50	24	0	0.00	0	17,270
896.6	16.8	17,559	0.3	0.53	130	0	0.00	0	17,689
896.8	17	17,874	0.5	0.56	292	0	0.00	0	18,165
897	17.2	18,190	0.7	0.58	502	0	0.00	0	18,692
897.2	17.4	18,508	0.9	0.60	756	0	0.00	0	19,264
897.4	17.6	18,828	1.1	0.61	1,050	0	0.00	0	19,878
897.6	17.8	19,150	1.3	0.63	1,381	0	0.00	0	20,531
897.8	18	19,474	1.5	0.64	1,746	0	0.00	0	21,220
898	18.2	19,799	1.7	0.65	2,141	0	0.00	0	21,940
898.2	18.4	20,126	1.9	0.66	2,564	0	0.00	0	22,690
898.4	18.6	20,455	2.1	0.67	3,011	0	0.00	0	23,467
898.6	18.8	20,786	2.3	0.67	3,480	0	0.00	0	24,266
898.8	19	21,119	2.5	0.68	3,967	0	0.00	0	25,086
899	19.2	21,453	2.7	0.68	4,470	0	0.00	0	25,923
899.2	19.4	21,789	2.9	0.68	4,986	0	0.00	0	26,776
899.4	19.6	22,127	3.1	0.68	5,514	0	0.00	0	27,641
899.6	19.8	22,467	3.3	0.68	6,050	0	0.00	0	28,516
899.8	20	22,808	3.5	0.68	6,592	0	0.00	0	29,400
900	20.2	26,142	3.7	0.68	7,140	0	0.00	0	33,282
900.2	20.4	26,397	3.9	0.67	7,690	0.2	0.51	87	34,175
900.4	20.6	26,650	4.1	0.67	8,243	0.4	0.53	256	35,149
900.6	20.8	26,900	4.3	0.67	8,795	0.6	0.55	484	36,179
900.8	21	27,148	4.5	0.66	9,347	0.8	0.56	761	37,256
901	21.2	27,394	4.7	0.66	9,897	1	0.57	1,079	38,371
901.2	21.4	27,637	4.9	0.65	10,445	1.2	0.57	1,433	39,516
901.4	21.6	27,879	5.1	0.64	10,991	1.4	0.57	1,815	40,684
901.6	21.8	28,118	5.3	0.64	11,533	1.6	0.58	2,220	41,871
901.8	22	28,355	5.5	0.63	12,072	1.8	0.57	2,643	43,071
902	22.2	28,591	5.7	0.63	12,608	2	0.57	3,079	44,278
902.2	22.4	28,824	5.9	0.62	13,141	2.2	0.57	3,524	45,489
902.4	22.6	29,056	6.1	0.61	13,672	2.4	0.56	3,973	46,701
902.6	22.8	29,285	6.3	0.61	14,200	2.6	0.55	4,424	47,909
902.8	23	29,513	6.5	0.60	14,726	2.8	0.55	4,874	49,113
903	23.2	29,739	6.7	0.59	15,252	3	0.54	5,320	50,311
903.2	23.4	29,964	6.9	0.59	15,777	3.2	0.53	5,761	51,502
903.4	23.6	30,186	7.1	0.58	16,303	3.4	0.52	6,195	52,684
903.6	23.8	30,408	7.3	0.58	16,830	3.6	0.51	6,622	53,859
903.8	24	30,627	7.5	0.57	17,359	3.8	0.50	7,041	55,027
904	24.2	30,845	7.7	0.56	17,891	4	0.49	7,453	56,189
904.2	24.4	31,062	7.9	0.56	18,427	4.2	0.48	7,858	57,346
904.4	24.6	31,277	8.1	0.56	18,967	4.4	0.47	8,257	58,500
904.6	24.8	31,490	8.3	0.55	19,511	4.6	0.46	8,651	59,653
904.8	25	31,702	8.5	0.55	20,062	4.8	0.45	9,042	60,806
905	25.2	31,913	8.7	0.54	20,618	5	0.44	9,430	61,961
905.2	25.4	32,122	8.9	0.54	21,180	5.2	0.43	9,818	63,121
905.4	25.6	32,330	9.1	0.53	21,749	5.4	0.43	10,208	64,286
905.6	25.8	32,537	9.3	0.53	22,323	5.6	0.42	10,599	65,459
905.8	26	32,742	9.5	0.53	22,903	5.8	0.41	10,995	66,639
906	26.2	32,946	9.7	0.52	23,487	6	0.41	11,396	67,829
906.2	26.4	33,149	9.9	0.52	24,075	6.2	0.40	11,802	69,026
906.4	26.6	33,350	10.1	0.52	24,666	6.4	0.40	12,215	70,230
906.6	26.8	33,550	10.3	0.52	25,256	6.6	0.39	12,633	71,439
906.8	27	33,750	10.5	0.51	25,846	6.8	0.39	13,055	72,651

Option 2 **Use single labyrinth weir + gates**

Step 1 **Enter Design Flow Data/Limitations**

Design Pool	904	ft
Weir Crest	896.3	ft

Step 2 **Enter, Compute, and Check Geometry Data**

LABYRINTH WEIR

Enter			***Compute***			***eqn***
Floor Elev.	886.3	ft	H_0	5.00	ft	(1) 8.1
Wc	180	ft	P	10.00	ft	El_{crest}-El_{floor}
α	18	°	W	36.00	ft	(1) 8.5
a	1.5	ft	L	103.08	ft	(1) 5.2
n	5	cycles	L_c	515.41	ft	(1) 8.6
$T_{weir\ wall}$	1.5	ft	B	48.54	ft	(1) 8.8
$T_{end\ wall}$	1.5	ft	S	46.17	ft	(1) 8.9
T_s	2	ft	q_0	51.8	cfs/ft	Q/Lc

Check		
Recommended Ratio		Result
Headwater	$H_0/P < 0.7$	0.5
Magnification	2< L/W <9.	2.9
Interference Lengt	$L_{de}/B \leq 0.35$	0.14

SPILLWAY GATES

Crest Elev.	879.8	ft
# of Gates	3	
Gate Width	25.0	ft
Gate Height	20.0	ft
C	0.68	

where:

$$\frac{L_{de}}{B} = \frac{h}{B} \cdot 6.1 \cdot e^{-0.052 \cdot \alpha} \qquad \text{(1) 8.4}$$

h = head on the weir

head on a labyrinth weir varies over the length of the weir. To simplify, critical depth was assumed, i.e.

$h = h_{crit}$ this condition suggests

critical depth eqn's from (2) App. C

$v_{crit} = \sqrt[3]{gq_0} =$ 11.9 ft/s

$h_{vel} = \frac{v_{crit}^2}{2g} =$ 2.2 ft

$h_{crit} = H_0 - h_{vel} =$ 2.8 ft

Due to the variability of h, this comp. is an estimate Also, h_{crit} could be solved for directly with q_0 but the method used is more conservative (i.e. higher h_{crit})

Step 3 **Structure Rating Curve**

Coefficients for (1) 5.16				
A_1	A_2	A_3	A_4	A_5
0.49	1.32	-4.13	4.24	-1.5

POOL	LABYRINTH WEIR					GATES			TOTAL
Elevation (ft)	Head [H] (ft)	H/P	C_D	Q_L (cfs)	q (cfs/ft)	Open [D] (ft)	Head [H] (ft)	Q_{gates} (cfs)	Q_{total} (cfs)
896.0	0	0.00	0.49	0.00	0.00	0.00	16.20	0.0	0
896.2	0	0.00	0.49	0.00	0.00	0.00	16.40	0.0	0
896.4	0.1	0.01	0.50	43.84	0.09	0.00	16.60	0.0	44
896.6	0.3	0.03	0.53	238.32	0.46	0.00	16.80	0.0	238
896.8	0.5	0.05	0.55	532.49	1.03	0.00	17.00	0.0	532
897.0	0.7	0.07	0.56	910.14	1.77	0.10	17.15	169.5	1080
897.2	0.9	0.09	0.58	1361.61	2.64	0.20	17.30	340.5	1702
897.4	1.1	0.11	0.59	1878.99	3.65	0.30	17.45	512.9	2392
897.6	1.3	0.13	0.60	2455.10	4.76	0.40	17.60	686.8	3142
897.8	1.5	0.15	0.61	3083.13	5.98	0.50	17.75	862.1	3945
898.0	1.7	0.17	0.61	3756.52	7.29	1.00	17.70	1721.9	5478
898.2	1.9	0.19	0.62	4468.99	8.67	2.00	17.40	3414.4	7883
898.4	2.1	0.21	0.62	5214.54	10.12	3.00	17.10	5077.3	10292
898.6	2.3	0.23	0.62	5987.50	11.62	4.00	16.80	6710.1	12698
898.8	2.5	0.25	0.62	6782.51	13.16	5.00	16.50	8312.4	15095
899.0	2.7	0.27	0.62	7594.64	14.74	6.00	16.20	9883.8	17478
899.2	2.9	0.29	0.62	8419.34	16.34	7.00	15.90	11423.8	19843
899.4	3.1	0.31	0.61	9252.49	17.95	8.00	15.60	12932.0	22184
899.6	3.3	0.33	0.61	10090.42	19.58	9.00	15.30	14407.9	24498
899.8	3.5	0.35	0.61	10929.93	21.21	10.00	15.00	15851.1	26781
900.0	3.7	0.37	0.60	11768.25	22.83	10.00	15.20	15956.4	27725
900.2	3.9	0.39	0.59	12603.09	24.45	10.00	15.40	16061.0	28664
900.4	4.1	0.41	0.59	13432.60	26.06	10.00	15.60	16165.0	29598
900.6	4.3	0.43	0.58	14255.37	27.66	10.00	15.80	16268.3	30524
900.8	4.5	0.45	0.57	15070.44	29.24	10.00	16.00	16370.9	31441
901.0	4.7	0.47	0.57	15877.23	30.81	10.00	16.20	16472.9	32350
901.2	4.9	0.49	0.56	16675.55	32.35	10.00	16.40	16574.3	33250
901.4	5.1	0.51	0.55	17465.56	33.89	10.00	16.60	16675.1	34141
901.6	5.3	0.53	0.54	18247.76	35.40	10.00	16.80	16775.2	35023
901.8	5.5	0.55	0.53	19022.92	36.91	15.00	14.50	23377.0	42400
902.0	5.7	0.57	0.53	19792.06	38.40	15.00	14.70	23537.6	43330
902.2	5.9	0.59	0.52	20556.43	39.88	15.00	14.90	23697.2	44254
902.4	6.1	0.61	0.51	21317.40	41.36	15.00	15.10	23855.7	45173
902.6	6.3	0.63	0.51	22076.50	42.83	15.00	15.30	24013.2	46090
902.8	6.5	0.65	0.50	22835.28	44.31	15.00	15.50	24169.6	47005
903.0	6.7	0.67	0.49	23595.33	45.78	15.00	15.70	24325.1	47920
903.2	6.9	0.69	0.49	24358.16	47.26	15.00	15.90	24479.5	48838
903.4	7.1	0.71	0.48	25125.21	48.75	15.00	16.10	24633.0	49758
903.6	7.3	0.73	0.48	25897.72	50.25	15.00	16.30	24785.5	50683
903.8	7.5	0.75	0.47	26676.69	51.76	20.00	14.00	30627.2	57304
904.0	7.7	0.77	0.47	27462.83	53.28	20.00	14.20	30845.2	58308
904.2	7.9	0.79	0.46	28256.48	54.82	20.00	14.40	31061.6	59318
904.4	8.1	0.81	0.46	29057.53	56.38	20.00	14.60	31276.6	60334
904.6	8.3	0.83	0.45	29865.33	57.94	20.00	14.80	31490.1	61355
904.8	8.5	0.85	0.45	30678.67	59.52	20.00	15.00	31702.2	62381
905.0	8.7	0.87	0.45	31495.63	61.11	20.00	15.20	31912.8	63408
905.2	8.9	0.89	0.44	32313.54	62.69	20.00	15.40	32122.1	64436
905.4	9.1	0.91	0.44	33128.92	64.28	20.00	15.60	32330.0	65459
905.6	9.3	0.93	0.43	33937.31	65.85	20.00	15.80	32536.6	66474
905.8	9.5	0.95	0.43	34733.29	67.39	20.00	16.00	32741.8	67475
906.0	9.7	0.97	0.43	35510.31	68.90	20.00	16.20	32945.8	68456
906.2	9.9	0.99	0.42	36260.63	70.35	20.00	16.40	33148.6	69409
906.4	10.1	1.01	0.42	36975.22	71.74	20.00	16.60	33350.1	70325
906.6	10.3	1.03	0.41	37643.67	73.04	20.00	16.80	33550.4	71194
906.8	10.5	1.05	0.41	38254.10	74.22	20.00	17.00	33749.5	72004
907.0	10.7	1.07	0.40	38793.04	75.27	20.00	17.20	33947.5	72741

<u>Option 3</u> **Use Obermeyer Gates**

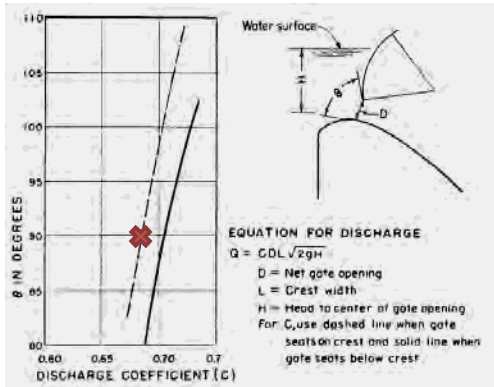
Obermeyer gates consist of a series of steel gates, held up by a series of air bladders that sit on the downstream side of the gate and control the gates position by the level of their inflation. When fully inflated, the gates are up, when deflated the gates are down.

The gates act as a sharp crested weir with discharge defined by:

$$Q = C_D \cdot L_c \cdot H^{1.5}$$ Ref 1

where:

C_D = Discharge Coefficient
L_c = Crest Length
H = Head



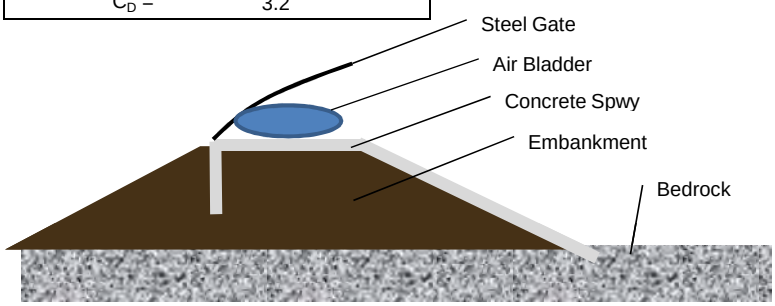
Step 1 **Enter Geometry Data**

OBERMEYER GATES

Raised Elev.	896.3	ft-msl
Lowered Elev	888.3	ft-msl
L_c	155	ft
C_D =	3.2	

SPILLWAY GATES

Crest Elev.	879.8	ft
# of Gates	3	
Gate Width	25.0	ft
Gate Height	20.0	ft
C	0.68	



Step 2 **Structure Rating Curve**

Obermeyer gates assumed to be completely down

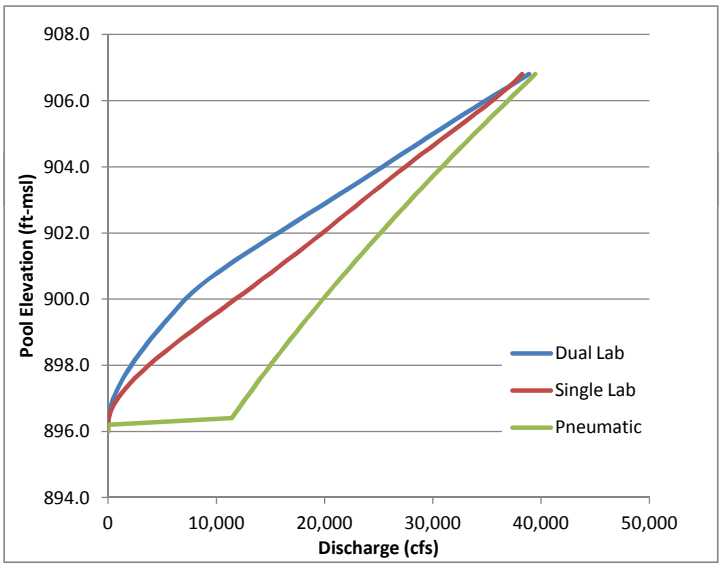
POOL	OBERMEYER			SLIDE GATES				TOTAL
Elevation (ft)	[H] (ft)	C	Q (cfs)	[D] (ft)	[H] (ft)	C	Q_{gates} (cfs)	Q_{total} (cfs)
896.0	0	3.2	0.00	0.00	16.20	0.7	0.0	0
896.2	0	3.2	0.00	0.00	16.40	0.7	0.0	0
896.4	8.1	3.2	11434.29	0.00	16.60	0.7	0.0	11434
896.6	8.3	3.2	11860.39	0.00	16.80	0.7	0.0	11860
896.8	8.5	3.2	12291.65	0.00	17.00	0.7	0.0	12292
897.0	8.7	3.2	12728.01	0.10	17.15	0.7	169.5	12898
897.2	8.9	3.2	13169.42	0.20	17.30	0.7	340.5	13510
897.4	9.1	3.2	13615.82	0.30	17.45	0.7	512.9	14129
897.6	9.3	3.2	14067.15	0.40	17.60	0.7	686.8	14754
897.8	9.5	3.2	14523.36	0.50	17.75	0.7	862.1	15386
898.0	9.7	3.2	14984.40	1.00	17.70	0.7	1721.9	16706
898.2	9.9	3.2	15450.21	2.00	17.40	0.7	3414.4	18865
898.4	10.1	3.2	15920.76	3.00	17.10	0.7	5077.3	20998
898.6	10.3	3.2	16395.99	4.00	16.80	0.7	6710.1	23106
898.8	10.5	3.2	16875.85	5.00	16.50	0.7	8312.4	25188
899.0	10.7	3.2	17360.30	6.00	16.20	0.7	9883.8	27244
899.2	10.9	3.2	17849.31	7.00	15.90	0.7	11423.8	29273
899.4	11.1	3.2	18342.82	8.00	15.60	0.7	12932.0	31275
899.6	11.3	3.2	18840.80	9.00	15.30	0.7	14407.9	33249
899.8	11.5	3.2	19343.21	10.00	15.00	0.7	15851.1	35194
900.0	11.7	3.2	19850.00	10.00	15.20	0.7	15956.4	35806
900.2	11.9	3.2	20361.14	10.00	15.40	0.7	16061.0	36422
900.4	12.1	3.2	20876.60	10.00	15.60	0.7	16165.0	37042
900.6	12.3	3.2	21396.33	10.00	15.80	0.7	16268.3	37665
900.8	12.5	3.2	21920.31	10.00	16.00	0.7	16370.9	38291
901.0	12.7	3.2	22448.50	10.00	16.20	0.7	16472.9	38921
901.2	12.9	3.2	22980.86	10.00	16.40	0.7	16574.3	39555
901.4	13.1	3.2	23517.36	10.00	16.60	0.7	16675.1	40192
901.6	13.3	3.2	24057.98	10.00	16.80	0.7	16775.2	40833
901.8	13.5	3.2	24602.67	15.00	14.50	0.7	23377.0	47980
902.0	13.7	3.2	25151.42	15.00	14.70	0.7	23537.6	48689
902.2	13.9	3.2	25704.19	15.00	14.90	0.7	23697.2	49401
902.4	14.1	3.2	26260.94	15.00	15.10	0.7	23855.7	50117
902.6	14.3	3.2	26821.66	15.00	15.30	0.7	24013.2	50835
902.8	14.5	3.2	27386.32	15.00	15.50	0.7	24169.6	51556
903.0	14.7	3.2	27954.88	15.00	15.70	0.7	24325.1	52280
903.2	14.9	3.2	28527.33	15.00	15.90	0.7	24479.5	53007
903.4	15.1	3.2	29103.63	15.00	16.10	0.7	24633.0	53737
903.6	15.3	3.2	29683.75	15.00	16.30	0.7	24785.5	54469
903.8	15.5	3.2	30267.69	20.00	14.00	0.7	30627.2	60895
904.0	15.7	3.2	30855.40	20.00	14.20	0.7	30845.2	61701
904.2	15.9	3.2	31446.87	20.00	14.40	0.7	31061.6	62509
904.4	16.1	3.2	32042.06	20.00	14.60	0.7	31276.6	63319
904.6	16.3	3.2	32640.97	20.00	14.80	0.7	31490.1	64131
904.8	16.5	3.2	33243.57	20.00	15.00	0.7	31702.2	64946
905.0	16.7	3.2	33849.82	20.00	15.20	0.7	31912.8	65763
905.2	16.9	3.2	34459.72	20.00	15.40	0.7	32122.1	66582
905.4	17.1	3.2	35073.24	20.00	15.60	0.7	32330.0	67403
905.6	17.3	3.2	35690.35	20.00	15.80	0.7	32536.6	68227
905.8	17.5	3.2	36311.05	20.00	16.00	0.7	32741.8	69053
906.0	17.7	3.2	36935.30	20.00	16.20	0.7	32945.8	69881
906.2	17.9	3.2	37563.08	20.00	16.40	0.7	33148.6	70712
906.4	18.1	3.2	38194.39	20.00	16.60	0.7	33350.1	71544
906.6	18.3	3.2	38829.19	20.00	16.80	0.7	33550.4	72380
906.8	18.5	3.2	39467.47	20.00	17.00	0.7	33749.5	73217

Comparison of the 3 Alternatives

Excludes 3 spillway gates which are constant in the three alternatives

Pneumatic gates assumed to be fully lowered at 896.4

	Discharge (cfs)		
POOL Elevation (ft)	**Dual Labyrinth**	**Single Labyrinth**	**Pneum Gates**
896.0	0	0	0
896.2	0	0	0
896.4	24	44	11,434
896.6	130	238	11,860
896.8	292	532	12,292
897.0	502	910	12,728
897.2	756	1,362	13,169
897.4	1,050	1,879	13,616
897.6	1,381	2,455	14,067
897.8	1,746	3,083	14,523
898.0	2,141	3,757	14,984
898.2	2,564	4,469	15,450
898.4	3,011	5,215	15,921
898.6	3,480	5,987	16,396
898.8	3,967	6,783	16,876
899.0	4,470	7,595	17,360
899.2	4,986	8,419	17,849
899.4	5,514	9,252	18,343
899.6	6,050	10,090	18,841
899.8	6,592	10,930	19,343
900.0	7,140	11,768	19,850
900.2	7,778	12,603	20,361
900.4	8,499	13,433	20,877
900.6	9,279	14,255	21,396
900.8	10,108	15,070	21,920
901.0	10,977	15,877	22,448
901.2	11,878	16,676	22,981
901.4	12,806	17,466	23,517
901.6	13,753	18,248	24,058
901.8	14,715	19,023	24,603
902.0	15,687	19,792	25,151
902.2	16,665	20,556	25,704
902.4	17,645	21,317	26,261
902.6	18,624	22,076	26,822
902.8	19,600	22,835	27,386
903.0	20,572	23,595	27,955
903.2	21,538	24,358	28,527
903.4	22,498	25,125	29,104
903.6	23,452	25,898	29,684
903.8	24,400	26,677	30,268
904.0	25,344	27,463	30,855
904.2	26,285	28,256	31,447
904.4	27,223	29,058	32,042
904.6	28,162	29,865	32,641
904.8	29,103	30,679	33,244
905.0	30,048	31,496	33,850
905.2	30,999	32,314	34,460
905.4	31,956	33,129	35,073
905.6	32,922	33,937	35,690
905.8	33,898	34,733	36,311
906.0	34,883	35,510	36,935
906.2	35,877	36,261	37,563
906.4	36,880	36,975	38,194
906.6	37,889	37,644	38,829
906.8	38,901	38,254	39,467



Stanley Consultants INC.

Computed	A. Judd	Date	10/12/11	Subject	Lake Delhi Dam Reconstructin
Checked b	M. Weber	Date	12/9/2011		Dam Hydraulics - Sluice Pipes
Approved by		Date			

Description:

Analyze stage-discharge for Lake Delhi Sluice Pipes

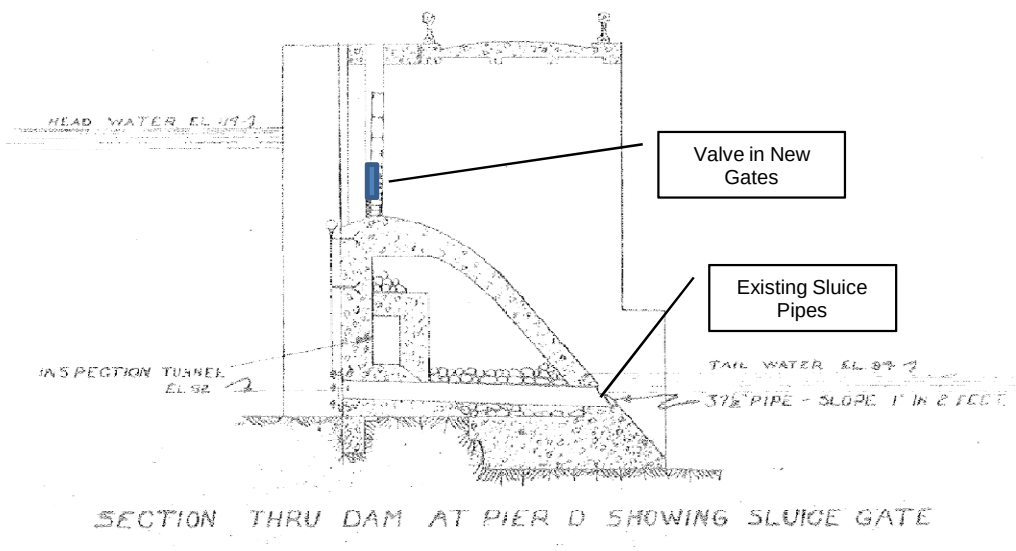
Reference:

(1) USBR, ***Design of Small Dams,*** 1987.
(2) Chow Ven Te; ***Open Channel Hydraulics***; McGraw-Hill, 1958.

Analysis:

Existing drawings show that there may be dual sluice pipes through one of the spillway piers

In addition valves are being considered for installation within the new spillway gates



Elevations provided are on local datum which converts to NGVD 29
by Local Datum + 774.8

Sluice Pipe Info.			Valve info.		
Number	2		Number	2	
Center Elev	856.8	'	Center Elev	884	'
Dia.	37.5	"	Dia.	30	"
Length	40	'+/-	Length	2	'+/-

Discharge through the sluice pipe is defined by:

$$Q = CA\sqrt{2gH}$$

where:

A = area of the opening,
H = difference between the upstream and downstream water levels, and,
C = discharge coefficient for the submerged orifice or the tube flow.

Where C = 0.62

Stage-Discharge

Normal Tailwater assumed to be 860 ft

Headwater El. (ft)	Tailwater El. (ft)	Q_{sluice} (cfs)	Q_{valves} (cfs)	
862	860	104	0	
863	860	128	0	
864	860	148	0	
865	860	165	0	
866	860	181	0	
867	860	195	0	
868	860	209	0	
869	860	221	0	
870	860	233	0	
871	860	245	0	
872	860	256	0	
873	860	266	0	
874	860	276	0	
875	860	286	0	
876	860	295	0	
877	860	304	0	
878	860	313	0	
879	860	322	0	
880	860	330	0	
881	860	338	0	
882	860	346	0	
883	860	354	0	
884	860	362	0	
885	860	369	47	
886	860	376	67	
887	860	384	82	
888	860	391	94	
889	860	398	106	
890	860	404	116	
891	860	411	125	
892	860	418	134	
893	860	424	142	
894	860	430	149	
895	860	437	157	
896	860	443	164	Normal Pool
897	862	437	170	
898	864	430	177	
899	866	424	183	
900	868	418	189	

Stanley Consultants INC

				Job No.	23601
Computed by	A. Judd	Date	10/27/2011	Subject	Lake Delhi Dam
Checked by	M. Weber	Date	12/9/2011		Hydraulics
Approved by		Date			Cofferdam Reqm'ts

Description:

Review cofferdam reqm'ts using 3 gates as flow bypass

Reference:

(1) Iowa DNR, ***Design Criteria and Guidelines for Iowa Dams***, T.B. 16, 1990.
(2) USBR, ***Design of Small Dams,*** 1987.
(3) Chow Ven Te; ***Open Channel Hydraulics***; McGraw-Hill, 1958.
(4) Henry T. Falvey, ***Hydraulic Design of Labyrinth Weirs***, ASCE Press, 2003.

Analysis:

First stage of construction will be repair of the powerhouse, a D/S cofferdam will be provided at the stilling basin. Once the powerhouse repairs are complete, construction of the south embankment and spillways will commence. Cofferdams will be constructed both upstream and downstream of the embankment

Task 1 **Estimate height of cofferdams required for given flows**

Return Period	Flow (cfs)
1	1400
2	4500
5	8700
10	12300

PHASE I - Open river bypass, small cofferdam U/S and D/S of Gates/Powerhouse

PHASE II - Gate bypass, larger cofferdam around south embankment/spwy construction

Spwy Gates	
Crest Elev.	879.8 ft
# of Gates	3
Open Width	25.0 ft
Open Height	20.0 ft
C	0.68

	PHASE I		PHASE II		
U/S W.S.E (ft-msl)	FLOW Q_{river} (cfs)	D/S W.S.E. (ft-msl)	GATE FLOW 3 GATES H_{gates} (ft)	Q_{gates} (cfs)	D/S W.S.E. (ft-msl)
868.0	276	859.1	0	0	858.5
868.5	603	859.8	0	0	858.5
869.0	991	860.6	0	0	858.5
869.5	1,440	861.5	0	0	858.5
870.0	1,952	862.4	0	0	858.5
870.5	2,524	863.3	0	0	858.5
871.0	3,159	864.2	0	0	858.5
871.5	3,855	865.2	0	0	858.5
872.0	4,612	866.1	0	0	858.5
872.5	5,432	867.0	0	0	858.5
873.0	6,312	867.9	0	0	858.5
873.5	7,255	868.7	0	0	858.5
874.0	8,259	869.5	0	0	858.5
874.5	9,324	870.3	0	0	858.5
875.0	10,452	871.0	0	0	858.5
875.5	11,641	871.6	0	0	858.5
876.0	12,891	872.2	0	0	858.5
876.5	14,203	872.8	0	0	858.5
877.0	15,577	873.4	0	0	858.5
877.5	17,012	874.0	0	0	858.5
878.0	18,509	874.6	0	0	858.5
878.5	20,067	875.2	0	0	858.5
879.0	21,687	875.8	0	0	858.5
879.5	23,369	876.4	0	0	858.5
880.0	25,112	877.1	0.2	23	858.6
880.5	26,917	877.8	0.7	149	858.9
881.0	28,783	878.5	1.2	335	859.3
881.5	30,711	879.2	1.7	565	859.8
882.0	32,701	879.9	2.2	832	860.3
882.5	34,752	880.6	2.7	1,131	860.9
883.0	36,865	881.2	3.2	1,460	861.5
883.5	39,039	881.8	3.7	1,815	862.1
884.0	41,275	882.4	4.2	2,195	862.8
884.5	43,573	882.9	4.7	2,598	863.4
885.0	45,932	883.5	5.2	3,024	864.0
885.5	48,353	884.0	5.7	3,470	864.7
886.0	50,835	884.5	6.2	3,937	865.3
886.5	53,379	885.1	6.7	4,422	865.9
887.0	55,985	885.8	7.2	4,927	866.5
887.5	58,652	886.5	7.7	5,448	867.1
888.0	61,381	887.3	8.2	5,988	867.6
888.5			8.7	6,544	868.1
889.0			9.2	7,116	868.6
889.5			9.7	7,704	869.1
890.0			10.2	8,307	869.6
890.5			10.7	8,925	870.0
891.0			11.2	9,558	870.4
891.5			11.7	10,205	870.8
892.0			12.2	10,866	871.2
892.5			12.7	11,541	871.5
893.0			13.2	12,229	871.9
893.5			13.7	12,931	872.2
894.0			14.2	13,645	872.6
894.5			14.7	14,372	872.9
895.0			15.2	15,111	873.2

PHASE I FLOW estimated from HEC-RAS model of post-breach river channel conditions at dam

D/S W.S.E.'s estimated from HEC-RAS model of river channel conditions just downstream of Lake Delhi Dam

GATE FLOW computed using Ref 2 Equations, assuming open gates

Stanley Consultants INC.

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Computed	A. Judd	Date	11/30/2011	Subject	Lake Delhi Dam
Checked by	M. Weber	Date	12/9/2011		Hydraulics
Approved by		Date			Headwater/Tailwater

Description:

Estimate Headwater and Tailwater elevations at Lake Delhi Dam for given flows

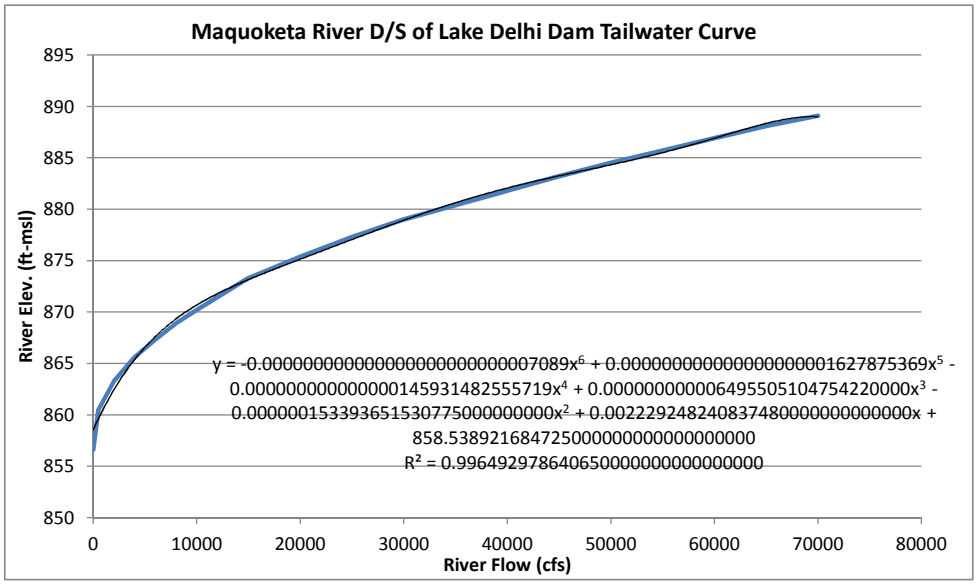
Analysis:

A HEC-RAS model was developed of the proposed lake delhi dam which was used to establish tailwater/headwater curves for a given dam discharge or river flow

Tailwater Curve
From HEC-RAS Model
At River Sta. 60797

Just downstream of dam
W/ Proposed Dam

Q (cfs)	W.S.E. (ft)
0	856.6
500	860.5
1000	861.4
2000	863.3
4000	865.6
6000	867.3
8000	868.9
10000	870.2
15000	873.3
20000	875.4
25000	877.3
30000	879
35000	880.4
40000	881.8
45000	883.2
50000	884.5
55000	885.7
60000	886.9
65000	888.1
70000	889.1



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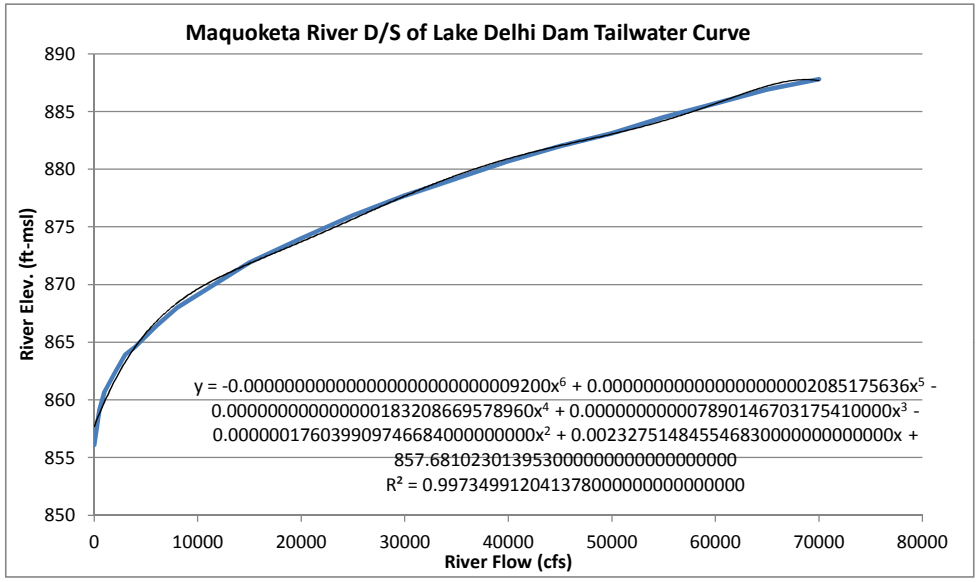
Checked by	M. Weber	Date	12/9/2011		Hydraulics
Approved by		Date			Headwater/Tailwater

Tailwater Curve
From HEC-RAS Model
At River Sta. 59617

~1200' downstream of dam
W/ Proposed Dam

Q (cfs)	W.S.E. (ft)
0	856.1
500	859.1
1000	860.7
2000	862.4
3000	863.9
4000	864.6
6000	866.4
8000	868
10000	869.1
15000	871.9
20000	874
25000	876
30000	877.7
35000	879.2
40000	880.7
45000	882
50000	883.1
55000	884.5
60000	885.7
65000	886.9
70000	887.8



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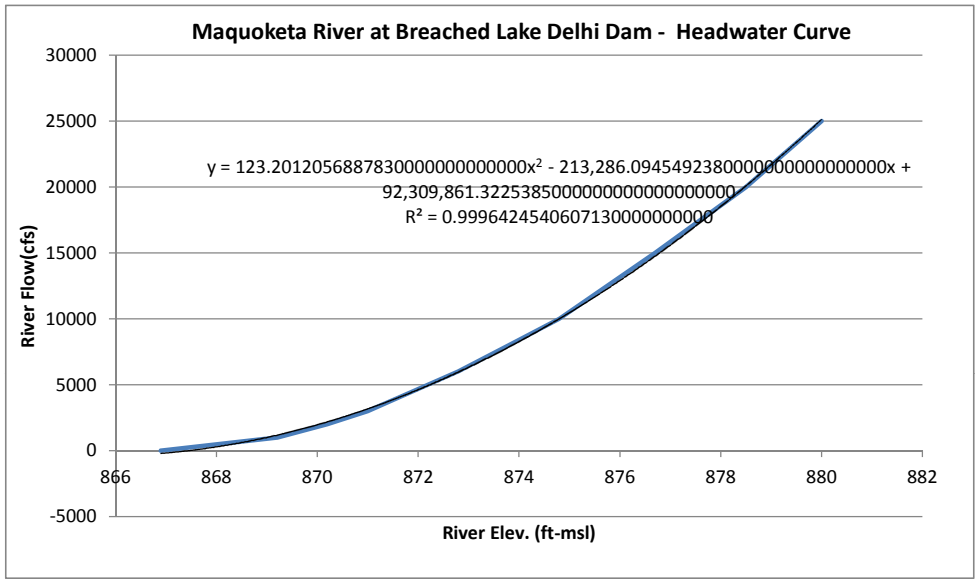
Computed	A. Judd	Date	11/30/2011	Subject	Lake Delhi Dam
Checked by	M. Weber	Date	12/9/2011		Hydraulics
Approved by		Date			Headwater/Tailwater

Headwater Curve
From HEC-RAS Model
At River Sta. 61003.11

This curve is used to develop cofferdam height requirements on the upstream side of the dam during Phase I construction

Just Upstream of Dam
Existing (breached dam) Condition

Q (cfs)	W.S.E. (ft)
0	866.9
992	869.2
2000	870.2
3000	871
4000	871.6
6000	872.8
8000	873.8
10000	874.8
15000	876.7
20000	878.5
25000	880