APPENDIX G

Hydraulic and Hydrology Analysis

Flood Frequency Analysis

A review of the USGS stream gauge data was performed along the Maquoketa River to estimate the flood frequency of the breach event at Delhi Dam. Figure 1 below shows the active and discontinued gauges evaluated. The two gauges were synthesized in HEC-SSP by importing them together. This provides a 63-year period of record consisting of 1925-1930, 1933-1982, and 2001-2010. Although the drainage areas associated with the active and discontinued gauge differ (305 mi^2 at 05417000-discontinued versus 294 mi^2 at 05416900-active), the difference is expected to be minor. The discontinued gauge that was near the dam (05417500) was correlated with the upstream gauge at Manchester for the few years that the two coexisted. As shown in Table 1 below, although there is a short record with which to correlate, those few show that the Manchester flow gauge is a good indicator of what to expect at the dam.

Table 1: Correlation between Manchester and Delhi Dam

USGS 05417000 Maquoketa River near		USGS 05417500 Maquoketa River near	
Drainage Area	305 mi^2	Drainage Area	347 mi^2
3/13/1929	5600	3/14/1929	7360
5/20/1933	2850	3/30/1933	5350
1/22/1934	695	8/11/1934	531
3/5/1935	4880	3/5/1935	4700
3/10/1936	4280	3/11/1936	2900
3/4/1937	8150	3/4/1937	6740
2/6/1938	5860	2/6/1938	5300
5/27/1939	1790	5/27/1939	1640
6/23/1940	2770	6/23/1940	3330

Figure 2 shows the flood frequency for the entire Manchester USGS gauge record computed using HEC-SSP 1.1 software. Although the USGS has yet to publish the peak flow for 2010, it was assumed to be the breach event peak of 24,900 cfs.

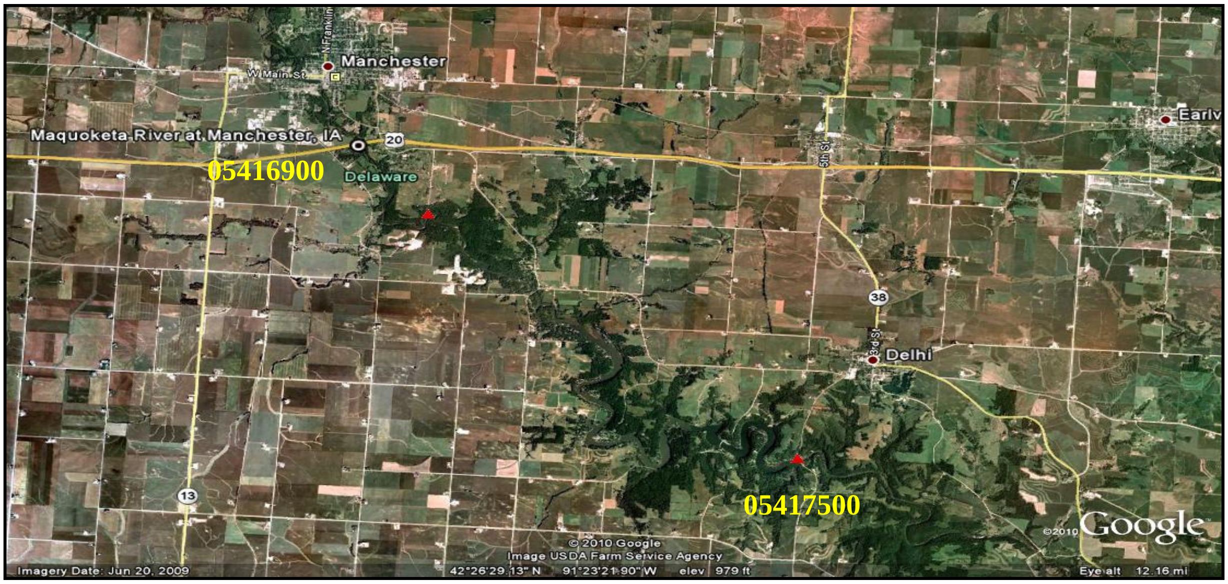


Figure 1: Goggle Earth image of the Makoqueta River

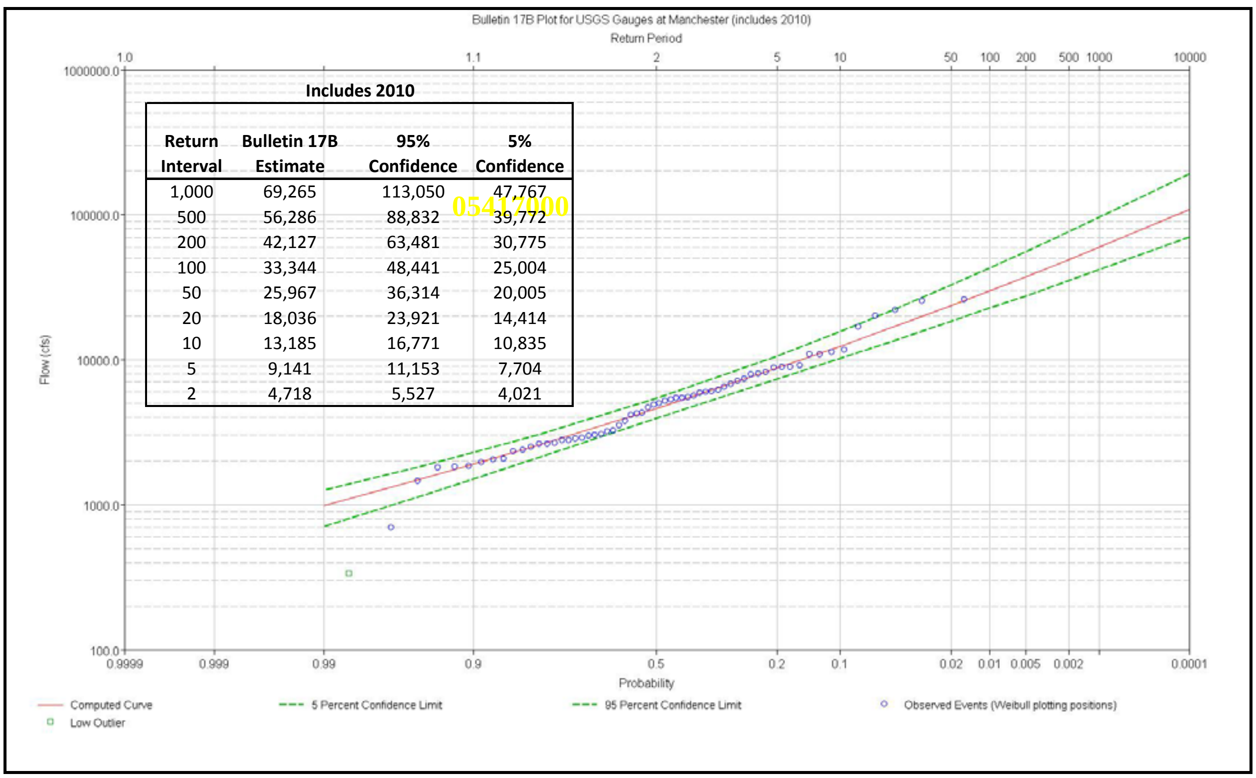


Figure 2: Makoqueta River Flood Frequency Estimate at Manchester, IA

From an historical perspective, the breach event was flood level that has been exceeded twice before in the short record (1925 and 2004) and there were flood of at least 20,000 cfs twice also (1947 and 2008).

Observed Precipitation

Total daily precipitation for the breach event was obtained from http://water.weather.gov/precip/ as shown in Figures 1-3. Total observed precipitation over the 3-day event totals around 10 inches based on this dataset.

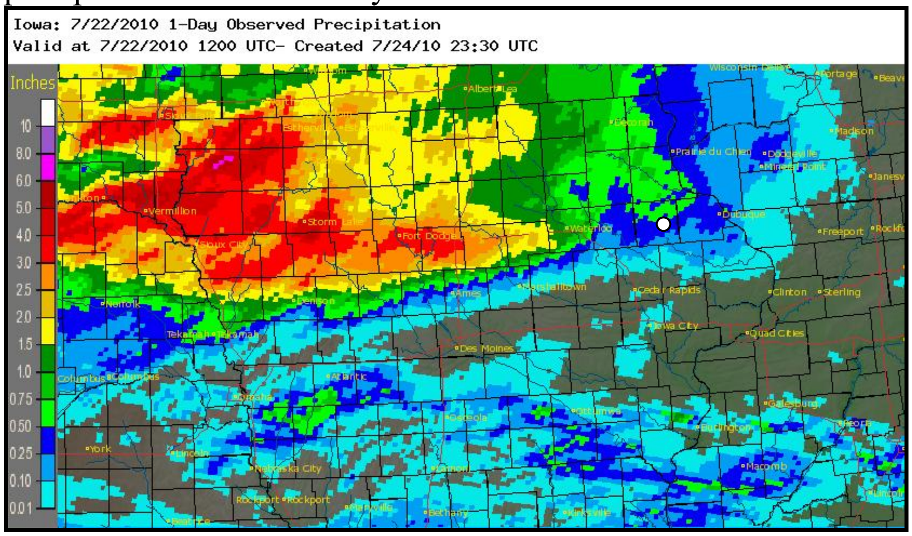


Figure 1: 1-day Observed Precipitation Total - July 22, 2010

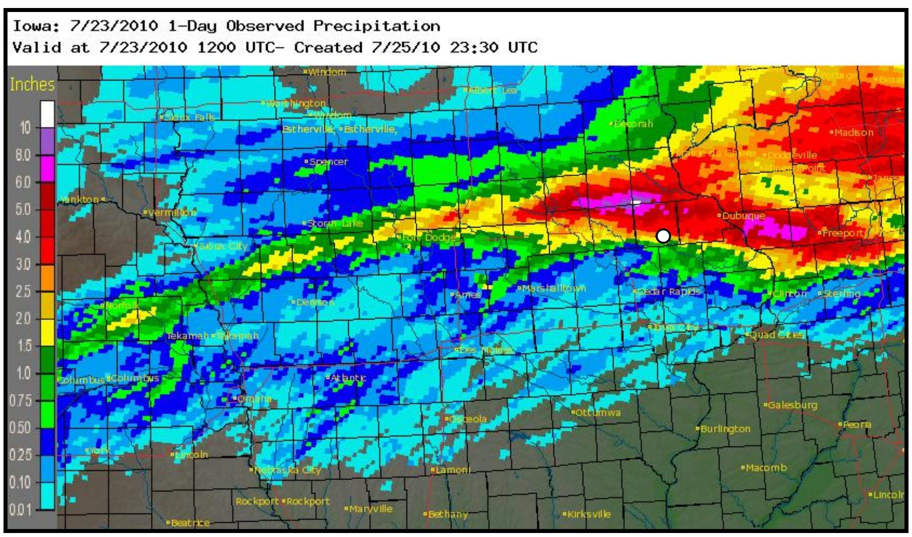


Figure 2: 1-day Observed Precipitation Total - July 23, 2010

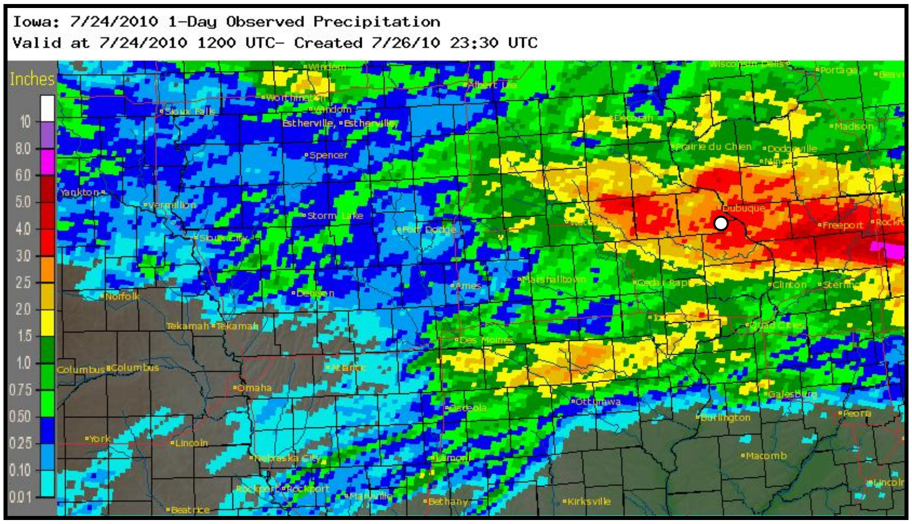


Figure 3: 1-day Observed Precipitation Total - July 24, 2010

HEC-RAS Model

The base model used for the investigation was provided by Mr. Jonathan D. Garton, P.E. from the Water Resources Section (Dam Safety) of the Iowa Department of Natural Resources. As indicated in his September 28, 2010 forwarding email, Mr. Garton used Geo-RAS input layers obtained from LIDAR as the cross-sections for the HEC-RAS model. The assumed Manning's n values associated with each cross-section are typical and expected given the land cover. Lake Delhi reservoir bathymetry was not obtained through survey and was modeled assuming a flat bottom in the channel. As shown in Figures 2-5 of the USGS Water-Resources Investigations Report 03-4085, Bathymetric Mapping, Sediment Quality, and Water Quality of Lake Delhi, Iowa, 2001-02, the riverine reservoir is mostly shallow and this assumption is reasonable.

The model's cross-sections begin upstream of the Delhi Dam just below Manchester, IA at US 20 and extend downstream to the City of Hopkinton (see Figure 1). The model contains two bridges and one inline structure (Delhi Dam cross-section 60900). There is one uniform lateral inflow at cross-section 63301.63 evenly distributed to cross-section 61003.11. As Mr. Garton acknowledges in his email, there was no documented basis for the uniform lateral inflow estimate as there is no gauging station. The intervening flows were estimated by applying the ratio of the drainage basin area between the Manchester gage and Delhi Dam to the drainage basin area upstream of the Manchester gage and then factoring the inflow hydrograph at the Manchester gaging station by this number. This resulted in a rough estimated hydrograph to approximate the intervening flow, which was adjusted in order to replicate the observed event.

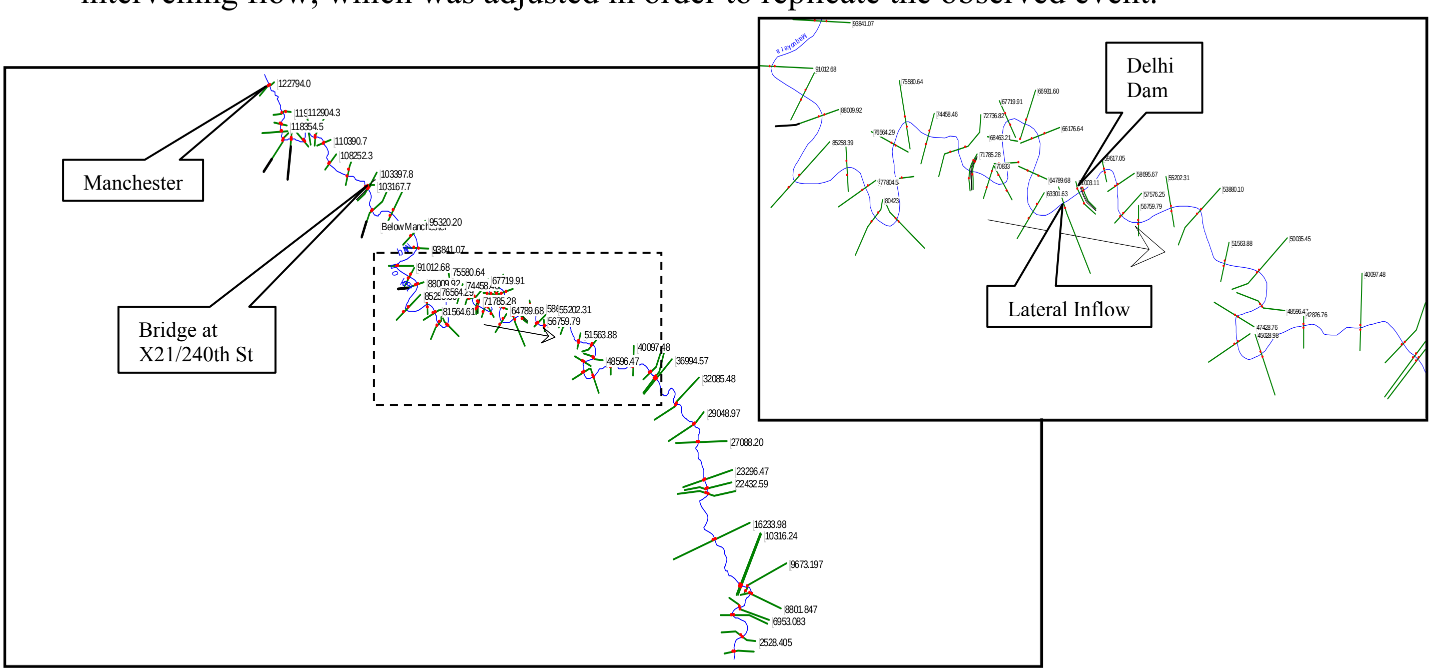


Figure 1: Base Geometry used in the HEC-RAS model

The inflow hydrograph for the breach event was taken directly from the USGS gauge data on the Maquoketa River at Manchester, IA (05416900) upstream of the reservoir (see Figure 2). The final revision of the assumed lateral inflow is shown in Figure 3.

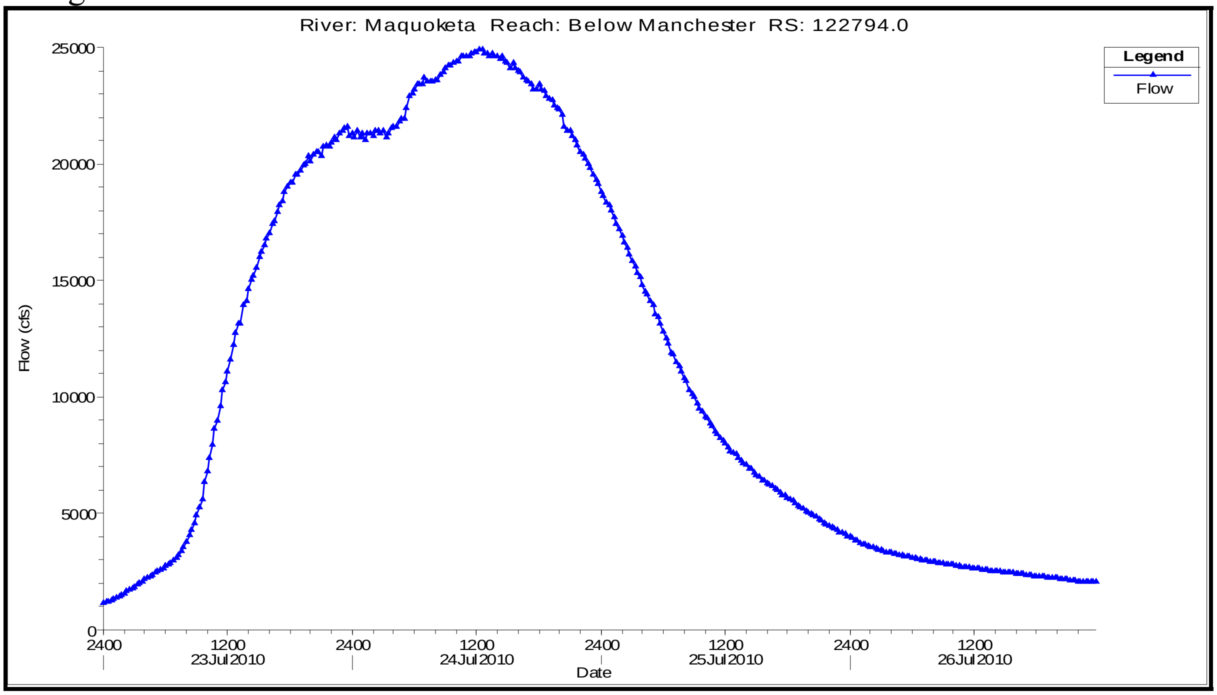


Figure 2: Inflow hydrograph at the most upstream cross-section in the HEC-RAS model

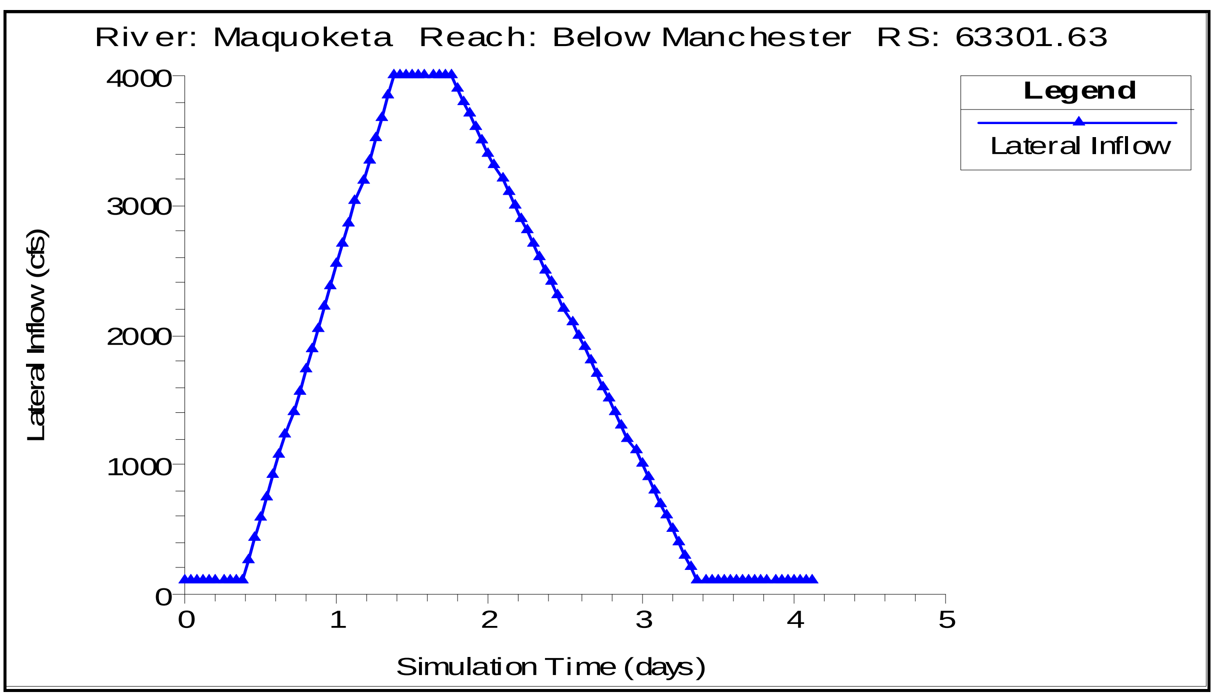


Figure 3: Uniform Lateral inflow hydrograph at Cross-Section 63301.63

The USGS hydrograph does not show a well-defined breach surge from the failure/overtopping of the rock dike at Quaker Mill Dam in the morning hours of July 23, 2010 (see Figures 4-5). The surge was either drowned out by the additional flow from the unnamed tributary to the Maquoketa River (denoted by the yellow arrow in Figure 5), or attenuated at the Manchester Mill Dam, or a combination of both. Since the downstream USGS gauge data at Manchester, IA did not have any reported problems, one can conclude that its usage as the inflow hydrograph in to the HEC-RAS model is appropriate.

Based on the model runs, the travel time required from Manchester to the dam is approximately 4-5 hrs. This was determined by comparing the peak of the inflow hydrograph at Manchester, which occurred around noon on July 24, 2010, while the peak at the dam (under the no-breach scenario) occurred between 4-5 pm in the various sensitivity runs.

Figure 4: Overtopping of the rock dike at Quaker Mill Dam

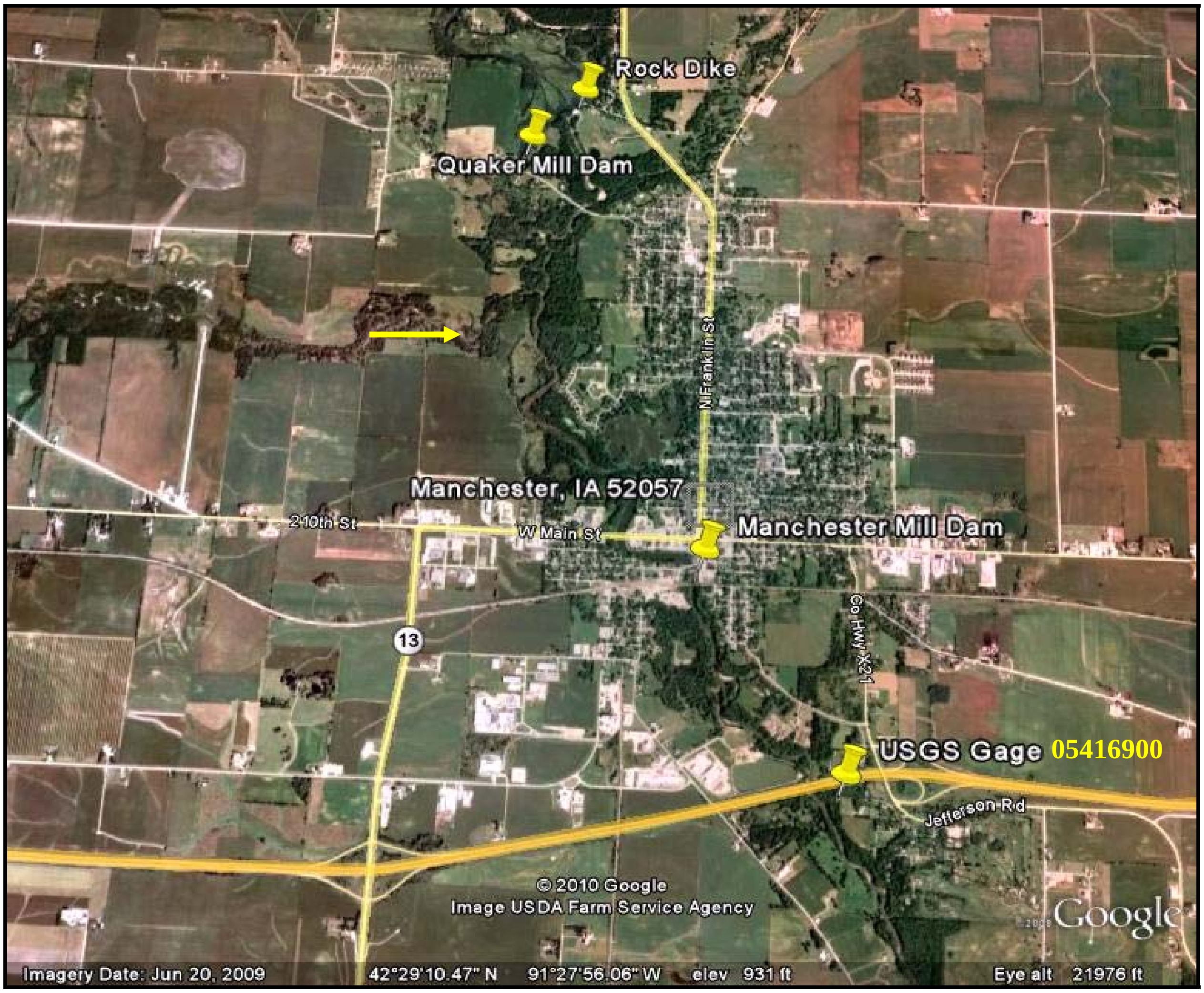
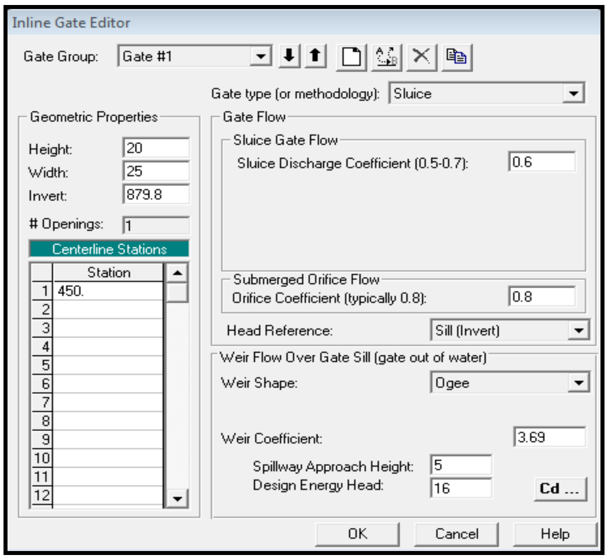


Figure 5: Overtopping of the rock dike at Quaker Mill Dam

The Iowa DNR base model does not include any flow through the dam's wicket gates. Each of these gates has a reported capacity of approximately 270 cfs. Relative to the inflow hydrograph and the estimated lateral inflow, this omission would be expected to have a negligible effect on the model results. Typically in HEC-RAS, the inline structure must usually be passing at least some flow for the model to properly run. In the Iowa DNR model, this was accomplished with the Gate #1 having an initial gate opening of 2.35 ft. This is comparable to the wicket gates being open at the start of the simulation time. However, the revised model prepared for the IPE evaluation added a fourth gate to represent the wicket gate discharge instead.

The base model included the 3 vertical lift spillway gates at the dam. In HEC-RAS, these lift gates were modeled as sluice gates and their typical geometry along with hydraulic assumptions are shown in Figure 6. The original Iowa DNR model used elevation controlled gates as shown in Figure 7. In the HEC-RAS software, elevation controlled gates begin opening once a specified water surface elevation is reached. The rate of gate opening is also input and once the gate opening is initiated, the gates were allowed to open to the fully open position. The trigger elevation and rate of closing can be similarly set but was not. The revised model used the time series option to replicate the operator's logbook (see Figure 8) except that the final openings were set to the measured, post-failure openings as follows: Gate #1 at 18', Gate #2 at 17' 1", Gate #3 at 4'3". The time series graphs for the 4 modeled gates in the revised model are shown in Figure 9. Note that Gate #4 simulates the nearly constant wicket gate flow and therefore, the opening does not change throughout the simulation time.



Almost all of the spillway computations are based on orifice conditions (see Figure 11) except for a brief period on July 23.

Max TW = 877.52 ft. This is below the gates' invert EL and therefore, never used.

According to the operator's logbook, Gate # 2 would have been fully out of the water briefly on July 23 (about 8 am until about 10 am) and thus, these weir flow assumptions were used those hrs.

Figure 6: Iowa DNR HEC-RAS screenshot of the spillway gate assumptions

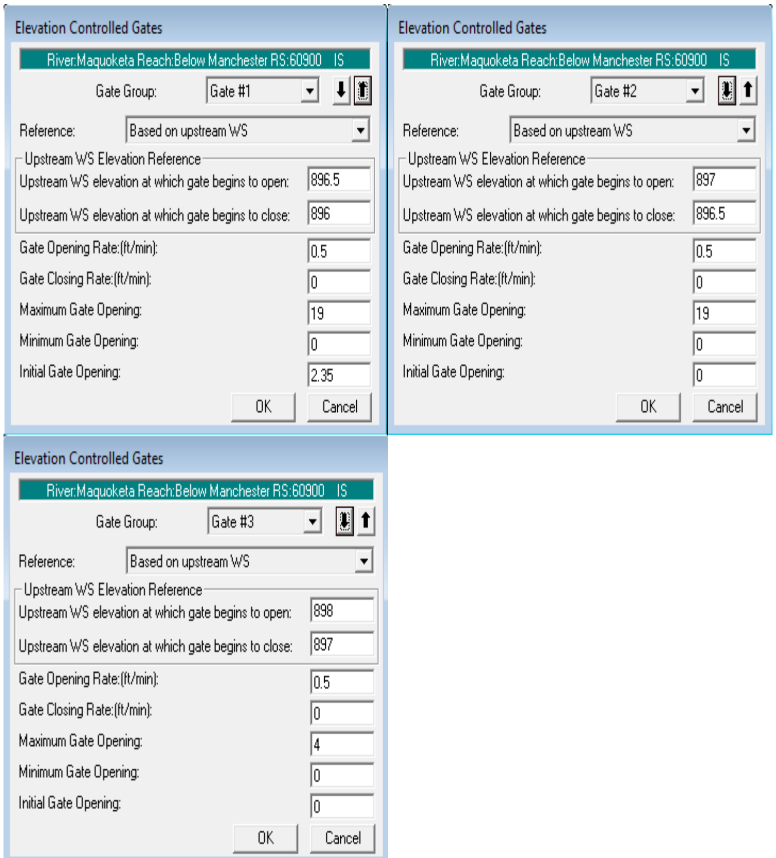


Figure 7: Iowa DNR HEC-RAS screenshot of the elevation controlled gate assumptions

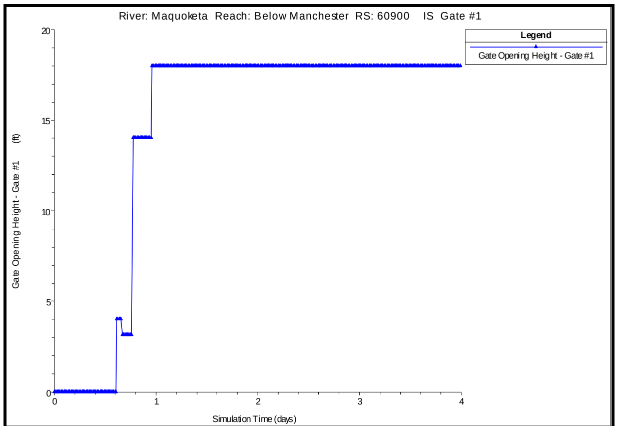
DATE	TIME AM/PM	YOUR NAME	WATER LEVEL	WICKET #1 NORTH	WICKET #2 SOUTH	FLOOD GATE ACTIVITY COMMENTS
7/18	10:30am	CW	121.65	59	20	#2 4"
7/18	8:00pm	CW	121.6	34	20	#2 closed
7/19	7:35am	mzw	121.6	81	20	" "
7/19	8:15 am	Dave		83%	11%	[illegible]
7/19	5:10Am	mzw	121.650	90	18%	
7/20	7:30Am	mzw	121.625	87	11	
7/20	5:45Pm	mzw	121.65	87	11	
7/21	8:30Am	mzw	121.625	83	11	
7/21	6:45Pm	mzw	121.625	77	11	
7/22	8:30 A.M	Dave	121.75	91%	[illegible] 100% of [illegible]	#2 - 8" 1.2%
7/22	9:15Am	mzw	121.75	91	100	" "
7/22	12:30Pm	mzw	121.6	25	10 Put on	outs 2° 8"
7/22	6:00Pm	mzw	121.7	89	10	2# 8"
7/22	9:15Pm	mzw	121.725	90	20	2" 20"
7/23	12:15Am	mzw	121.950	90	97	2" 48" 7.4

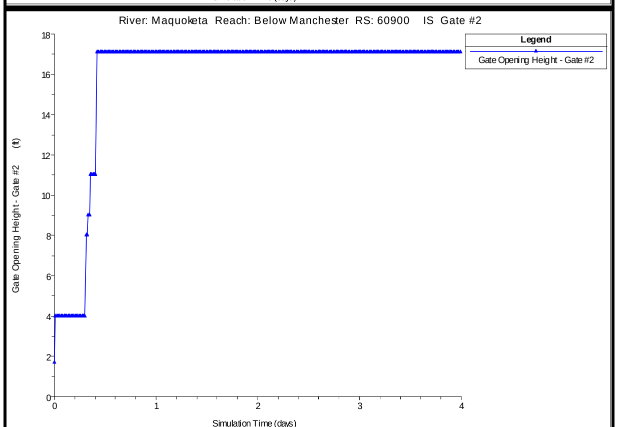
DATE	TIME AM/PM	YOUR NAME	WATER LEVEL	WICKET #1 NORTH	WICKET #2 SOUTH	FLOOD GATE ACTIVITY COMMENTS
7/23	6:30Am	mzw	121.50	8	10	#2 48" 7.4%
7/23	7:30 AM	Dave	121.65	—	—	#2 96" 14.8%
7/23	8:00 AM	Dave	121.57	—	—	#2 9' 108" 16.7%
7/23	9:30 AM	Dave	—	—	—	#2 " 132" 20.4%
7/23	10:00 AM	Dave	—	—	—	#2 open 32%
7/23	10:40Am	mzw	121.0	—	—	
7/23	11:35Am	mzw	121.0	—	—	# " [illegible] 24"
7/23	12:45Pm	mzw	120.7	—	—	# 2 83% [illegible] 1
7/23	2:45Pm	mzw	121.7	100%	100%	#1 - 48 #2 - open 40.4% [illegible]
7/23	4:00Pm	mzw	121.4	"	"	#1 38" #2 83% 3 9.3 48%
7/23	6:30pm	Dave	122.1	"	"	#1 - 14' 33 #2 33 66%
7/23	9:30pm	Dave	122.175	"	"	" " #3 6 11.2%
7/23	11:00pm	CW	122.9	"	"	14' 18' "
7/24	2:30am	CW	123.3	"	"	" " "
7/24	4:30am	CW	122.9	"	"	" " "

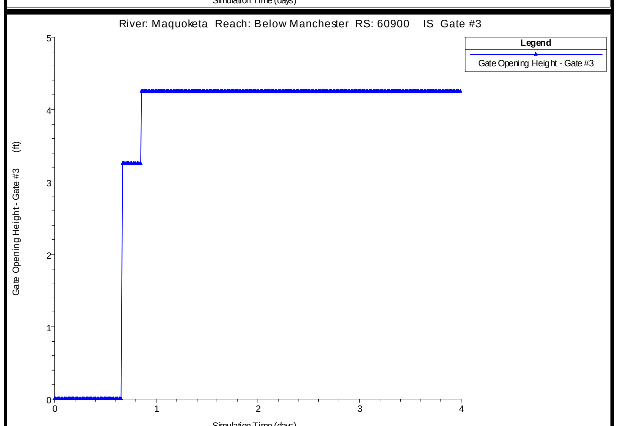
7/24 8:00 CW 122.9 [illegible] not sure gauge works " " 6½ feet high @ Dam

@ 1:00 Everyone off Dam

Figure 8: Log book of the wicket and spillway gate operations


River: Maquoketa Reach: Below Manchester RS: 60900 IS Gate #1
Legend
Gate Opening Height - Gate #1
Gate Opening Height - Gate #1 (ft)
Simulation Time (days)


River: Maquoketa Reach: Below Manchester RS: 60900 IS Gate #2
Legend
Gate Opening Height - Gate #2
Gate Opening Height - Gate #2 (ft)
Simulation Time (days)


River: Maquoketa Reach: Below Manchester RS: 60900 IS Gate #3
Legend
Gate Opening Height - Gate #3
Gate Opening Height - Gate #3 (ft)
Simulation Time (days)

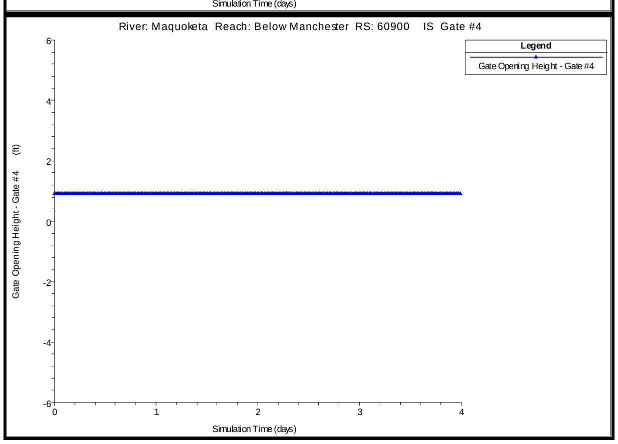

River: Maquoketa Reach: Below Manchester RS: 60900 IS Gate #4
Legend
Gate Opening Height - Gate #4
Gate Opening Height - Gate #4 (ft)
Simulation Time (days)

Figure 9: Spillway gate and wicket operations using operator's logbook with field-measured final openings

The spillway rating curve is computed directly in the HEC-RAS software based on the geometry and gate operation settings. The maximum discharges for the measured gate operations based on field measurements are shown in Figure 10. Aside from the brief period (≈8 am – 10 am) on July 23, 2010 when the logbook indicates that Gate # 2 would have fully been out of the water (weir flow regime), the entire flood event would have been passed under orifice flow conditions. The spillway rating table per gate shown in Figure 11 was computed using the orifice equations listed in the HEC-RAS Hydraulic Reference Manual.

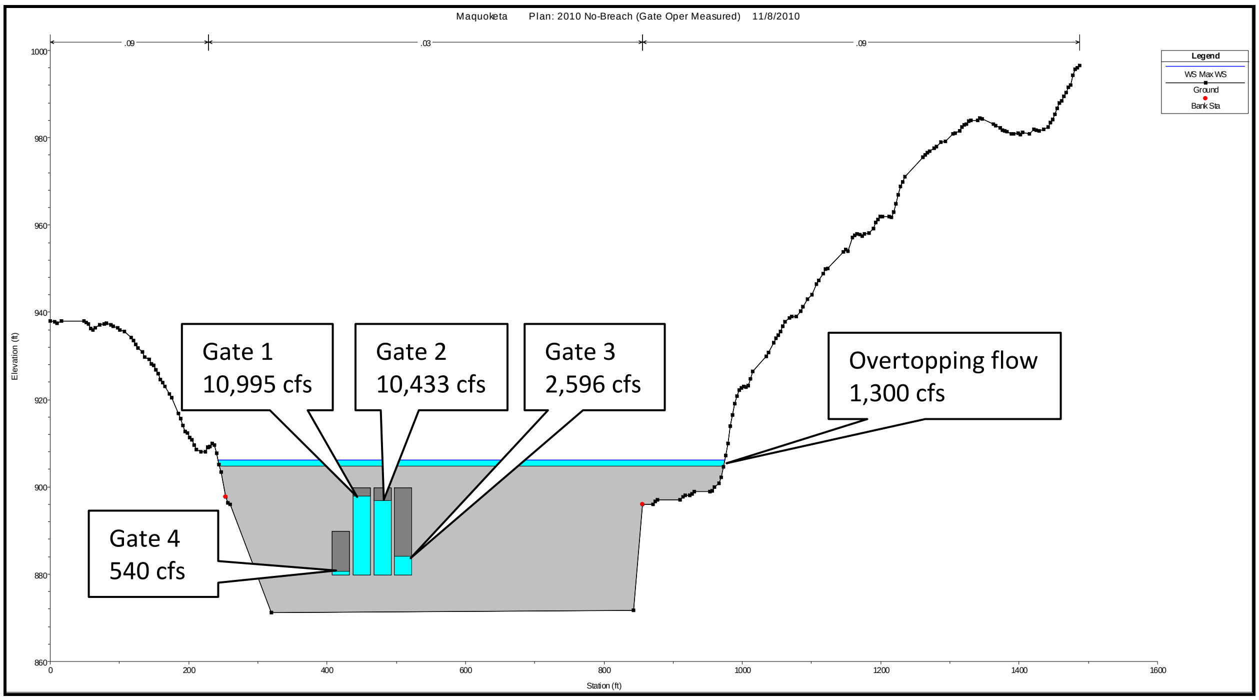


Figure 10: Maximum Discharge Computed at Peak Observed WSEL (905.55 ft NGVD29)

Sluice Gate

An example sluice gate with a broad crest is shown in Figure 8-7.

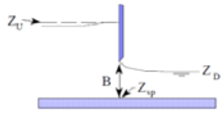


Figure 8-7 Example Sluice Gate with Broad Crested Spillway

The equation for a free flowing sluice gate is as follows:

$$Q = C\,W\,B\sqrt{2gH} \qquad (8\text{-}4)$$

Where: H = Upstream energy head above the spillway crest (Z_U - Z_{sp})

Sill EL = 879.8
C = 0.6
W = 25
B varies to a max of 18, 17.08 & 4.25

Gate 1
Gate 2
Gate 3
Gate 4 wicket 540 cfs

Total Discharge @ 905.55 ft
24,564 cfs

	Normal Pool					HWEL					Dam Crest	High Water Mark
	894	**895**	**896.3**	**897**	**898**	**899**	**900**	**901**	**902**	**903**	**904**	**904.8**	**905.55**
B	**Q (cfs)**	**Q (cfs)**	**Q (cfs)**	**Q (cfs)**	**Q (cfs)**	**Q (cfs)**	**Q (cfs)**	**Q (cfs)**	**Q (cfs)**	**Q (cfs)**	**Q (cfs)**	**Q (cfs)**	**Q (cfs)**
1	454	469	489	499	514	527	541	554	567	580	592	602	611
1.5	680	704	733	749	770	791	812	831	851	870	888	903	916
2	907	939	978	998	1,027	1,055	1,082	1,108	1,134	1,160	1,184	1,204	1,222
4	1,814	1,877	1,956	1,997	2,054	2,110	2,164	2,217	2,269	2,319	2,369	2,407	2,443
4.25	1,928	1,995	2,078	2,122	2,183	2,242	2,299	2,356	2,410	2,464	2,517	2,558	2,596
6	2,722	2,816	2,934	2,995	3,081	3,165	3,246	3,325	3,403	3,479	3,553	3,611	3,665
8	3,629	3,754	3,912	3,994	4,108	4,220	4,328	4,434	4,537	4,638	4,737	4,815	4,887
10	4,536	4,693	4,890	4,992	5,135	5,275	5,410	5,542	5,672	5,798	5,922	6,019	6,108
12	5,443	5,632	5,868	5,991	6,162	6,329	6,492	6,651	6,806	6,958	7,106	7,222	7,330
14	6,350	6,570	6,845	6,989	7,189	7,384	7,574	7,759	7,940	8,117	8,290	8,426	8,552
16	7,258	7,509	7,823	7,988	8,217	8,439	8,656	8,868	9,075	9,277	9,475	9,630	9,773
17.08	7,748	8,016	8,351	8,527	8,771	9,009	9,241	9,467	9,687	9,903	10,114	10,280	10,433
18	8,165	8,448	8,801	8,986	9,244	9,494	9,738	9,976	10,209	10,436	10,659	10,834	10,995

Figure 11: Discharge Curve per Gate for Orifice Flow Conditions

Based the input assumptions as described above, the revised Iowa DNR model was able to provide a good replication of the observed events (see Figure 12). A comparison of the model's and observed high water marks at other cross-sections is shown in Table 1. Another example of model accuracy comes from eyewitness reports which stated that the upstream bridge at County Road X21/240th St had "4 to 5 feet" of water flowing over the top of it at 9:40 am on July 24, 2010. Figure 13 below shows a picture of that bridge during the event as compared to the HEC-RAS model.

Table 1: Comparison between revised HEC-RAS model and observed events (at peak stage)

River Mile	Model Cross-Section	Location	Surveyed WSEL	Model WSEL
113.25	122794	U.S. Highway 20/USGS stream gauge 05416900	924.89	922.30
109.40	103297	County Road X21/240th St southeast of Manchester	911.43	910.65
101.58	60900	County Road X31/230th Ave crossing Lake Delhi	905.55[1]	906.17

		Dam		

[1]Maximum Depth of Overtopping Observed at Delhi Dam.

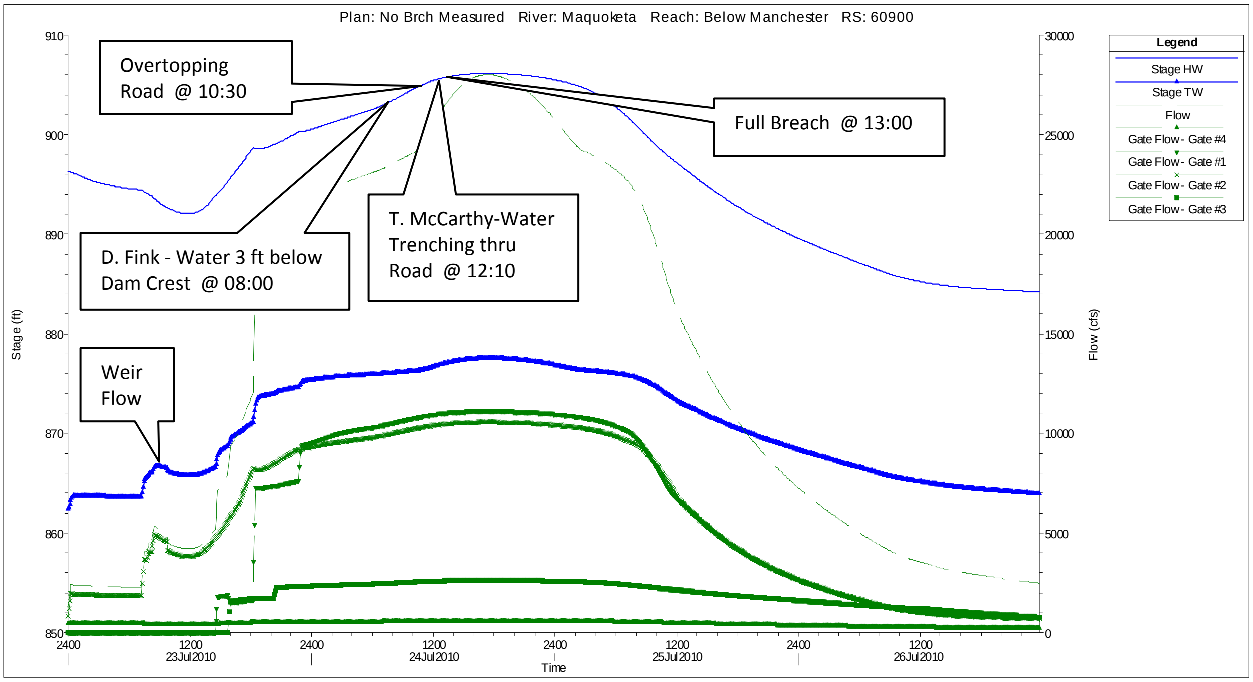


Figure 12: Revised HEC-RAS Stage and Flow Hydrograph

Although there are no eyewitness accounts of the reservoir lowering as far as shown on July 23, the weir flow conditions that produce it are a result of replicating the operator's logbook.

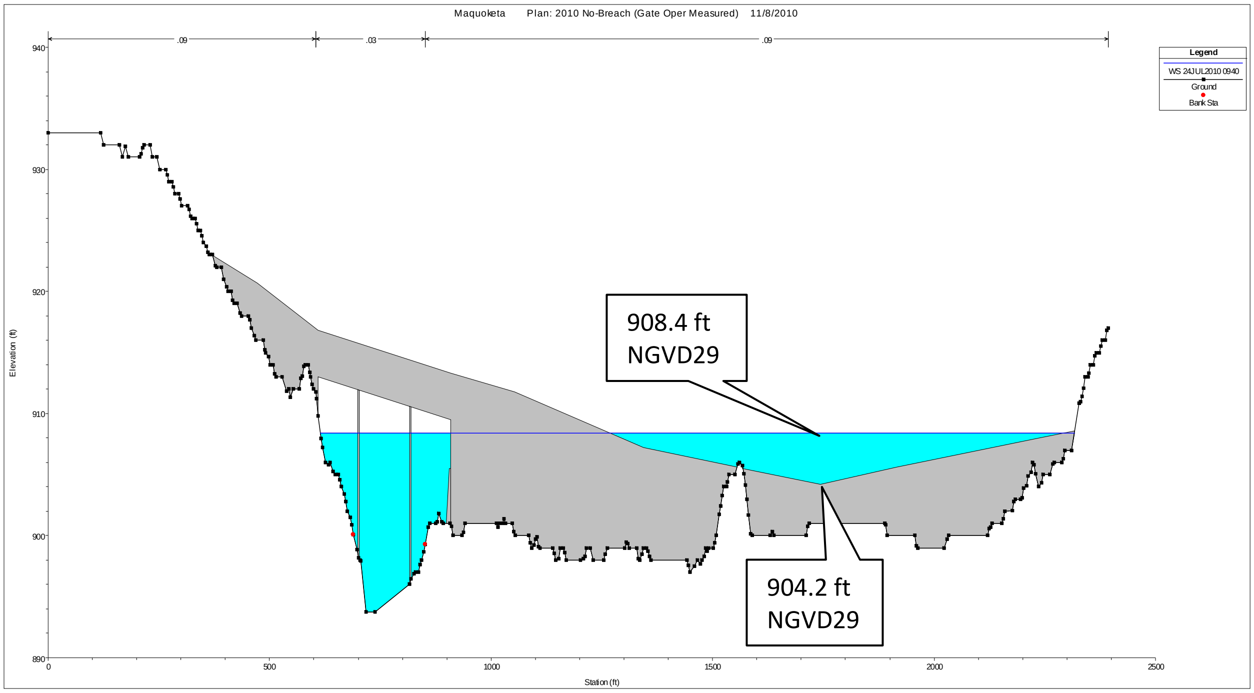
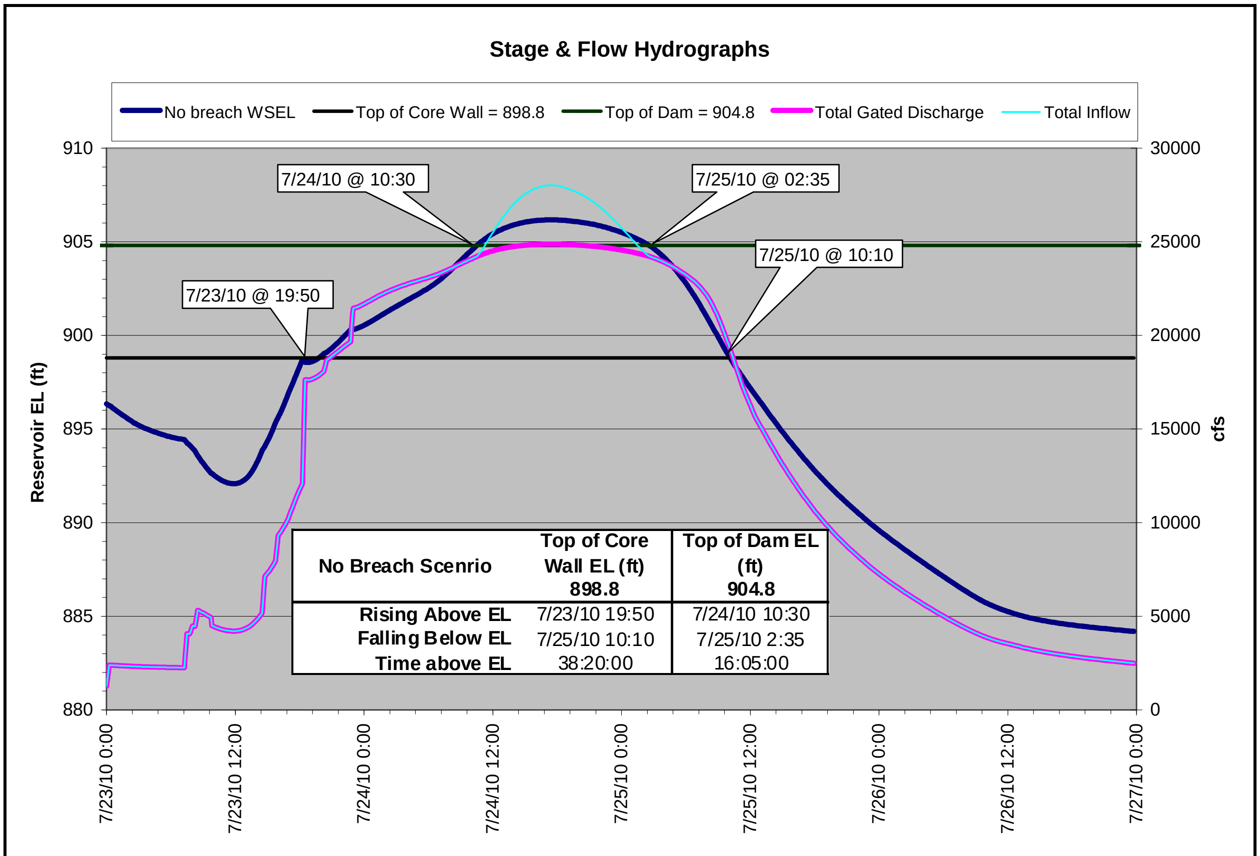


Figure 13: Photograph and Iowa DNR HEC-RAS Model Comparison

The revised model based on the operator's logbook and measured final gate openings matches the observed timing and water surface elevations well up to the point where breach progression is underway. As such, the revised model peaks after the actual time of failure.

Additional Modeling Runs Requested by the Team

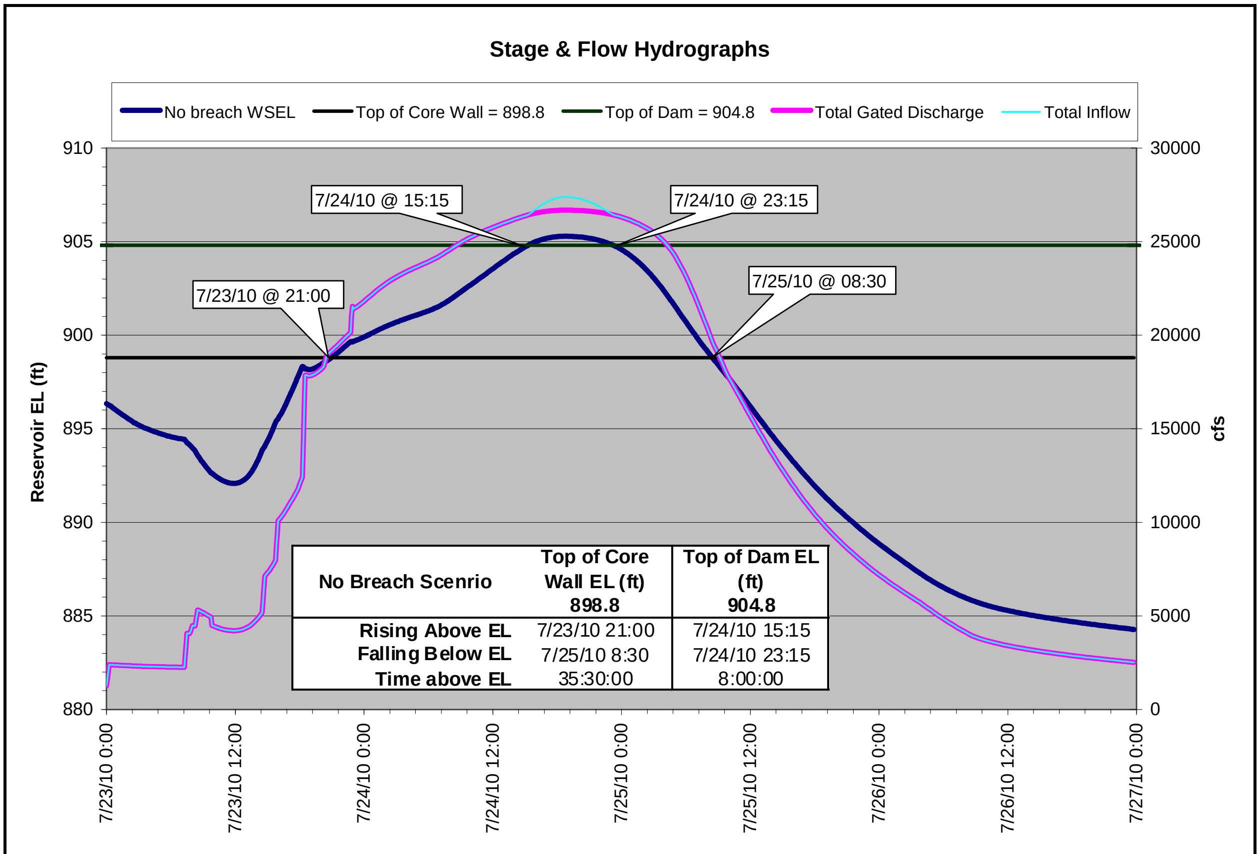
- The revised no-breach model was used to determine the potential total overtopping duration for when the reservoir elevation was above the top of core wall, 898.8 ft. This along with the stage hydrograph for the breach scenario is shown in Figure 14.



Time Series	Maximum	Time at Max	Volume (ac-ft)
HW Stage	906.17	24Jul2010 1725	
TW Stage	877.52	24Jul2010 1735	
Flow	28017.53	24Jul2010 1725	107016.99

Figure 14: Reservoir Duration above Top of Core Wall (EL 898.8 NGVD29)

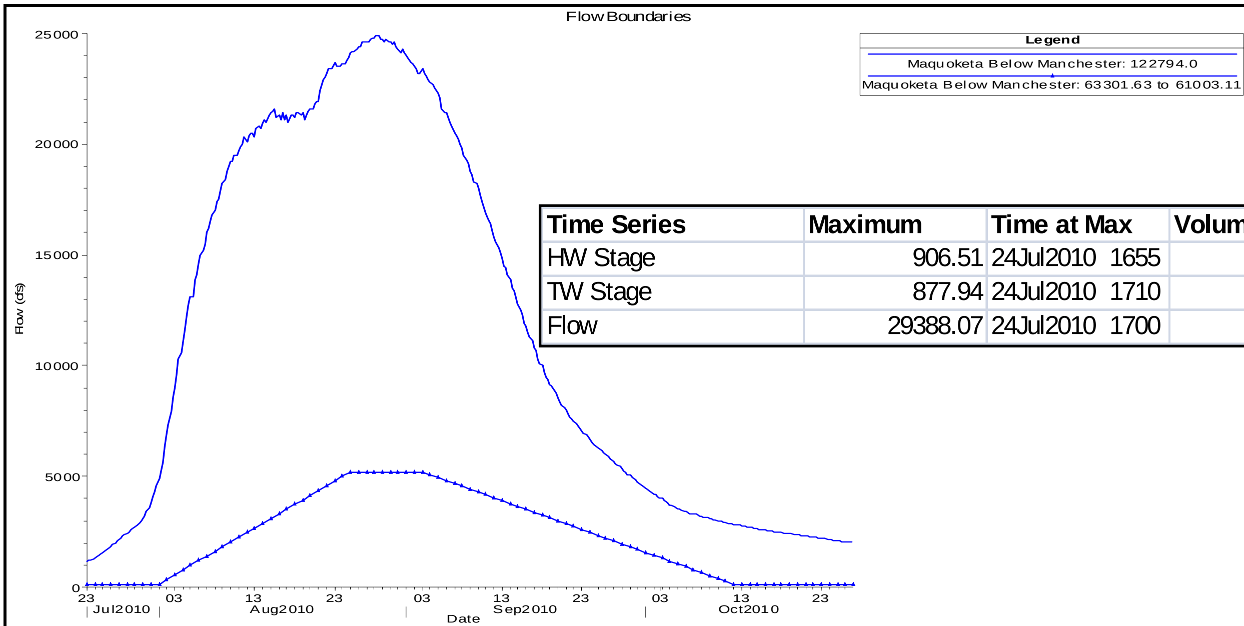
- The revised time series gate operations as reported in the operator's logbook shown in Figure 8. These results are shown in Figure 15.



Time Series	Maximum	Time at Max	Volume (ac-ft)
HW Stage	905.29	24Jul2010 1845	
TW Stage	877.32	24Jul2010 1900	
Flow	27380.18	24Jul2010 1845	107004.85

Figure 15: Reservoir Duration above Top of Core Wall (EL 898.8 NGVD 29) per Logbook Maximum Gate Openings

- The uniform lateral inflow assumption was varied by +/- 30% using the logbook gate operation sequence with the measured final openings. These results are shown in Figures 16 and 17.



Time Series	Maximum	Time at Max	Volume (ac-ft)
HW Stage	906.51	24Jul2010 1655	
TW Stage	877.94	24Jul2010 1710	
Flow	29388.07	24Jul2010 1700	111167.58

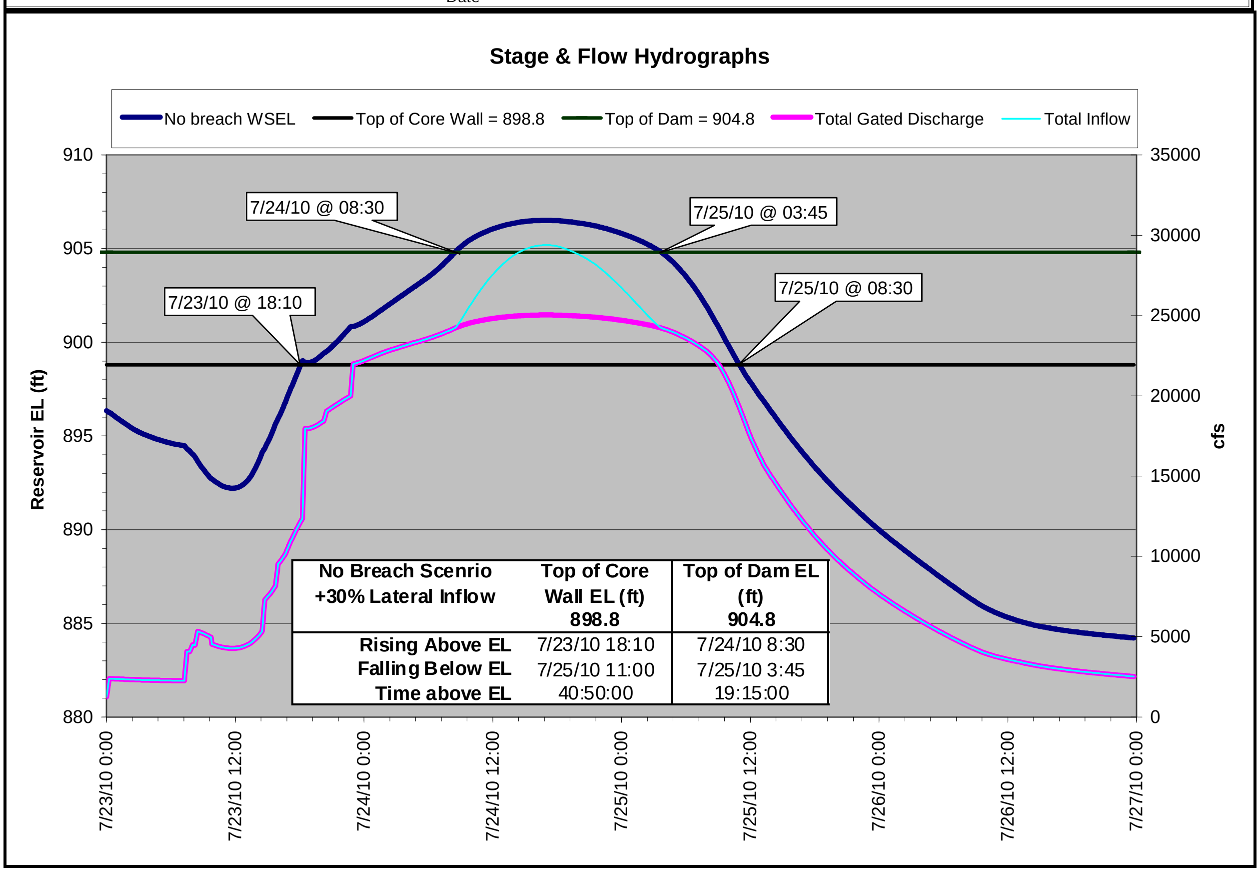
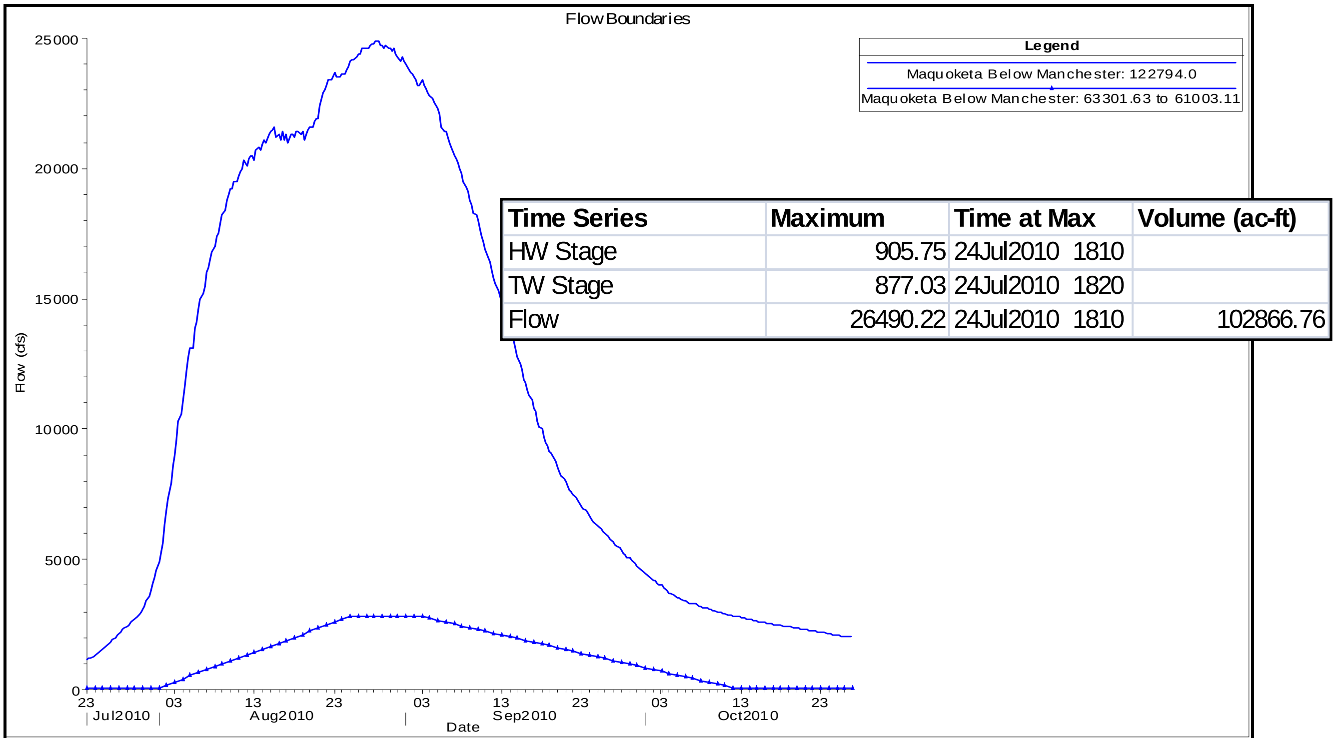
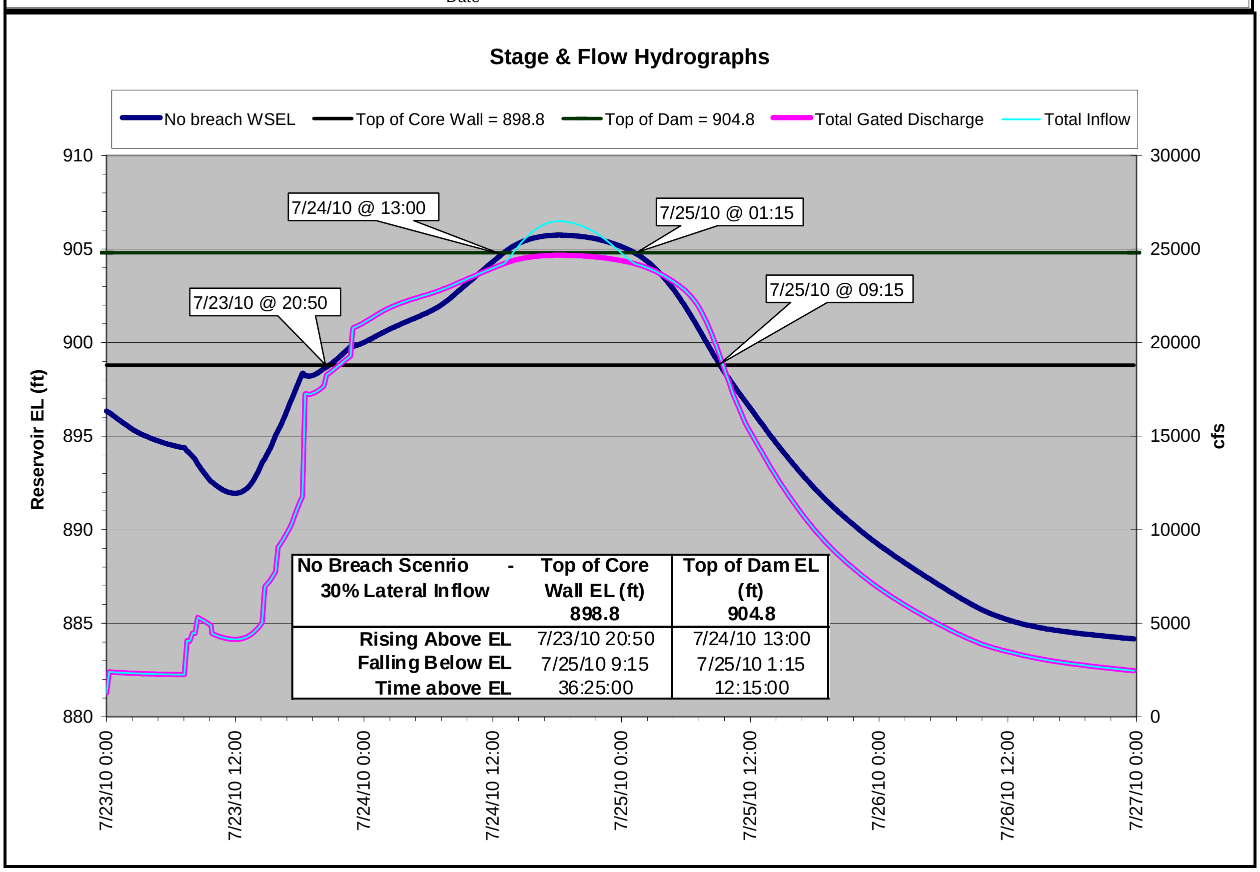


Figure 16: Lateral Inflow Assumption +30% and Stage and Flow Hydrograph Information per Measured Gate Openings.



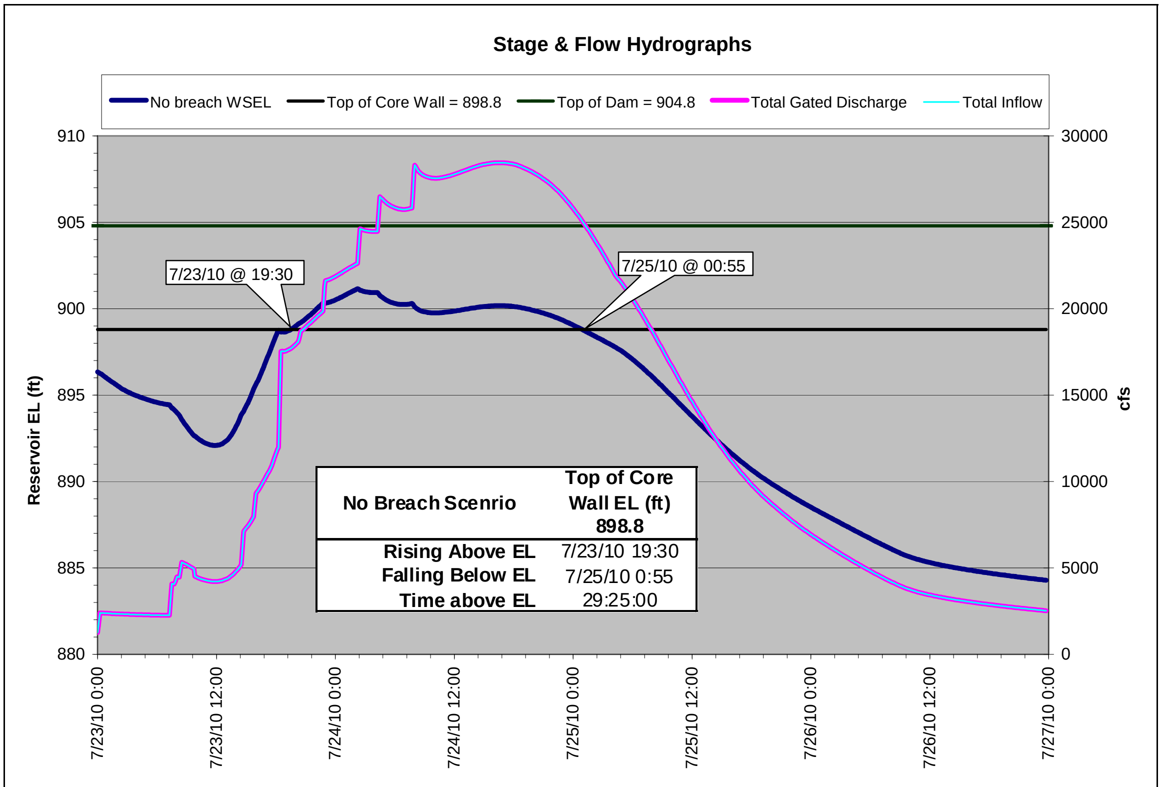
Time Series	Maximum	Time at Max	Volume (ac-ft)
HW Stage	905.75	24Jul2010 1810	
TW Stage	877.03	24Jul2010 1820	
Flow	26490.22	24Jul2010 1810	102866.76



No Breach Scenrio - 30% Lateral Inflow	Top of Core Wall EL (ft) 898.8	Top of Dam EL (ft) 904.8
Rising Above EL	7/23/10 20:50	7/24/10 13:00
Falling Below EL	7/25/10 9:15	7/25/10 1:15
Time above EL	36:25:00	12:15:00

Figure 17: Lateral Inflow Assumption -30% and Stage and Flow Hydrograph Information per Measured Gate Openings.

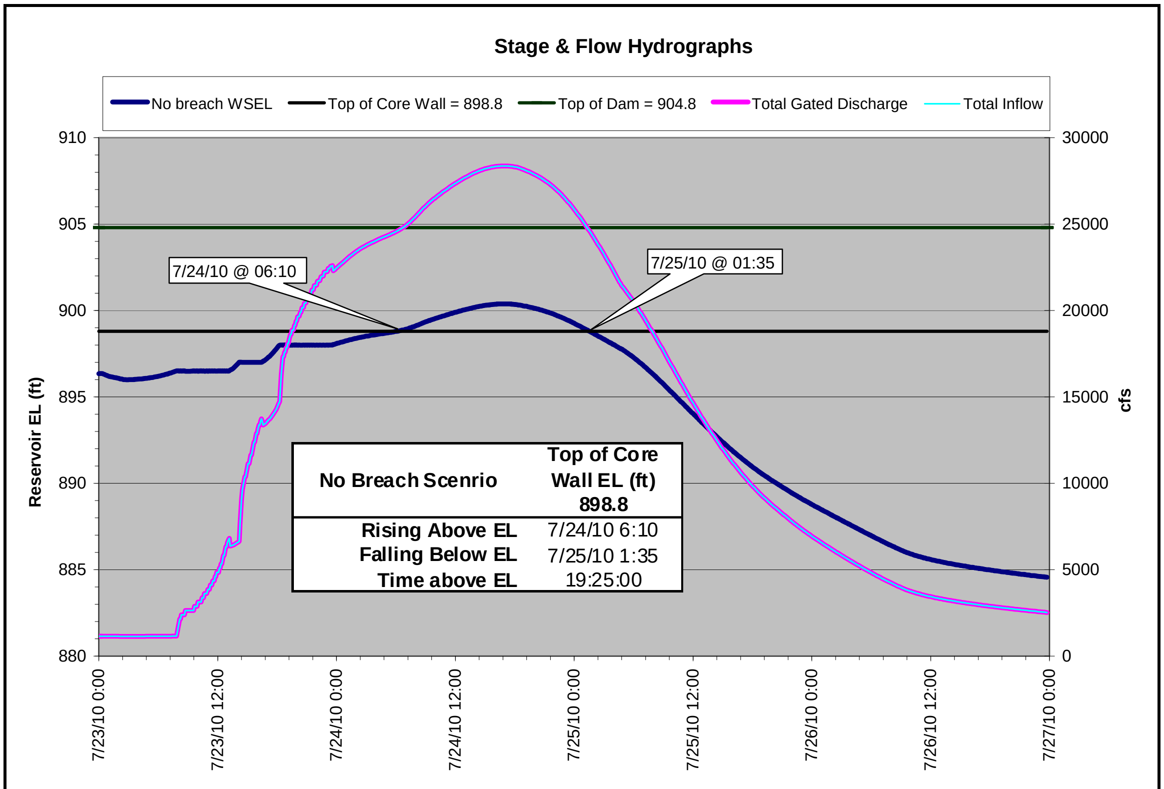
- The logbook gate operation sequence assuming that all three gates were functional and could be opened to 18 feet. These results are shown in Figures 18.



Time Series	Maximum	Time at Max	Volume (ac-ft)
HW Stage	901.16	24Jul2010 0215	
TW Stage	877.66	24Jul2010 1645	
Flow	28449.47	24Jul2010 1635	107011.16

Figure 18: Hypothetical Scenario using Logbook Controlled Gates with all 3 Gates Fully Operational to 18 feet Max Opening.

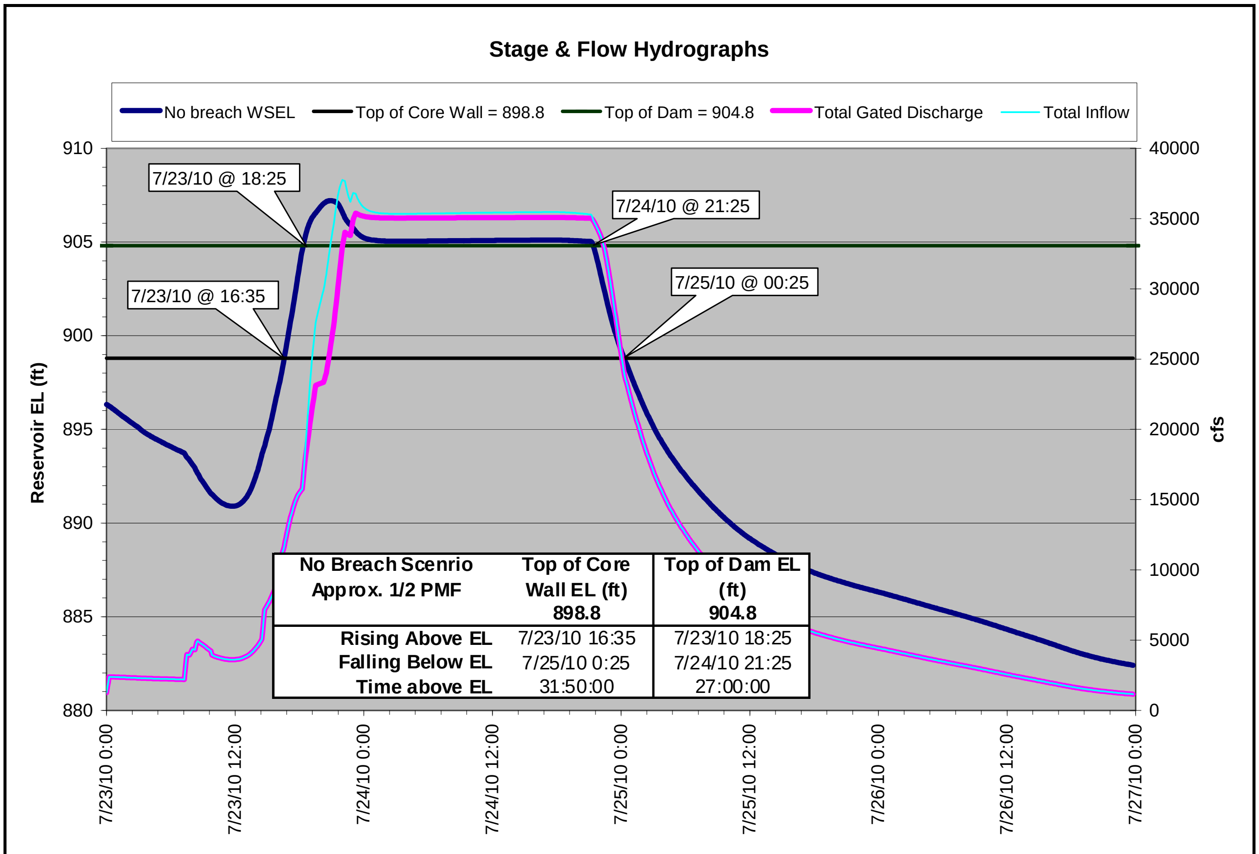
- Elevation controlled gate operation sequence assuming that all three gates were functional and could be opened to 18 feet. These results are shown in Figures 19.



Time Series	Maximum	Time at Max	Volume (ac-ft)
HW Stage	900.38	24Jul2010 1655	
TW Stage	877.63	24Jul2010 1705	
Flow	28364.64	24Jul2010 1655	106963.68

Figure 19: Hypothetical Scenario using Elevation Controlled Gates with all 3 Gates Fully Operational to 18 feet Max Opening.

• Figure 20 shows the results of routing approximately the ½ PMF hydrograph.



Time Series	Maximum	Time at Max	Volume (ac-ft)
HW Stage	907.21	23Jul2010 2055	
TW Stage	880.22	23Jul2010 2215	
Flow	37763.21	23Jul2010 2200	116147.52

Figure 20: No-Breach Scenario Routing the ½ PMF Inflow.

APPENDIX G

Report on Mechanical Equipment

Lake Delhi Dam
Mechanical and Electrical Observations and Report

PURPOSE: To provide supplemental information to the Lake Delhi dam failure report concerning the mechanical and electrical components of the dam. In particular, this supplement will provide observations and analysis of the gated spillway and hydropower features of the dam.

A. Mechanical and Electrical Observations

Lake Delhi Dam is located near Delhi, Iowa (between Dubuque and Waterloo). The dam overtopped and failed during a flood event on July 24th, 2010. The hydraulic and hydrology details for this flood event are provided in the main report. An inspection of the mechanical and electrical components took place on September 8th, 2010. The inspection included discussions with the dam operators including Mr. Dave Fink. Previous operators as well as other individuals familiar with the operation of the dam were also interviewed by the IPE. Their input is summarized elsewhere in the IPE report. Mr. Fink is in charge of the dam operations. The dam is owned and operated by the Lake Delhi Recreation Association. This report provides observations and analysis of the mechanical and electrical dam components. An additional phone conference took place with Mr. Dave Fink on September 27th, 2010, to clarify additional questions in regards to mechanical and electrical components.

Since the dam overtopped and failed, the lake behind the dam was released. Thus, the areas in front of the spillway gates are accessible as shown in the pictures below. A significant amount of debris is currently sitting in front of the gates.

At the September 8th inspection, it was noted that the spillway gates had not been operated since the flood event and were in the same position as when the dam failed. Gates 1 and 2 were fully opened. Gate 3 was only partially opened and could not be raised completely during the flood event. This is discussed further below.

The project was constructed between 1922 and 1929. The mechanical and electrical machinery went into service in 1928. The project includes three gated spillways and a hydropower facility. The hydropower facility is discussed further below. This structure is designed to pass river flow through the dam. During a design flood event, the structure is capable of passing approximately 25,000 cubic feet per second (cfs) through a combination of 3 spillway vertical lift gates and wicket gates in the hydropower facility. (This information was provided by Mr.

Fink.) Each vertical lift gate is 17 feet tall by 25 feet wide. The gates are operated using wire ropes and electric driven machinery.

View of the three spillway gates or vertical lift gates. Gate 1 is the north gate to the left in the picture. Gate 3 is the south gate to the right in the picture.

Note the debris in front of the gates. The debris in front of Gate 2 consists of several boats and pontoons.

The lift gate machinery includes an electric motor, gear box (gear reducer), pinion gear, spur gear, and cable drums. All components are in series. There is one set of machinery for each lift gate.

This picture shows gates 1 and 2 in the up position. These gates were raised prior to the flood event and are fully opened. Gate 3 is in the background. Again, the gates were left in the same position and not moved since the dam failure.

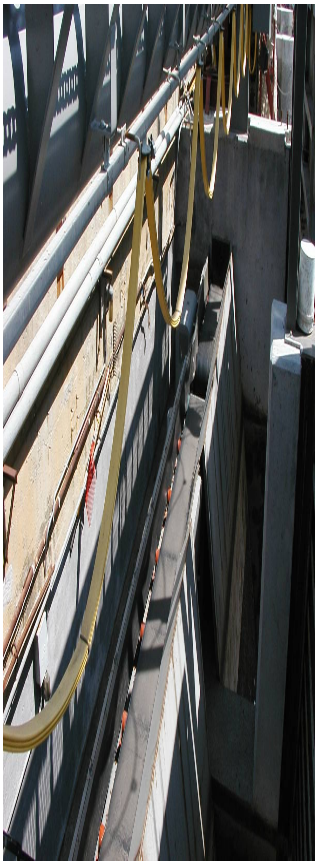
B. Hydropower Facility.

The hydropower at the site was deactivated in 1973. The dam was originally built by Interstate Power for electric power generation. The hydropower at the site has two turbines with each turbine having 16 foot diameter openings in the dam. The turbines are currently deactivated and no flow passes through them. Additional details on the turbines were not obtained for this report. There have been recent discussions, reports, and plans for bringing hydropower back on line and rehabilitating the turbines. Some of this design work has already been completed according to Mr. Fink.

There are also two wicket gates as part of the hydropower facility. The site uses the wicket gates extensively to also pass flow through the dam (in addition to the spillway gates). Each wicket gate is 5 foot in diameter. Mr. Fink stated the wicket gates could each pass around 250 cfs through the dam (500 cfs total).

Mr. Fink said the wicket gates are used primarily for passing flow in lieu of the spillway gates. He noted that even with a 1 -1/2 inch rainfall event, flow could usually still be passed through the wicket gates and not the spillway gates. Mr. Fink thought that 95% to 99% of flow is typically passed through the wicket gates, during small floods.

This picture shows the inside of the hydropower facility and the two turbines. Both of the turbines are currently deactivated. The new hydropower work being proposed would add new draft tubes for the turbines and provide up to 60% increase in flow (above the existing flow through the dam) according to Mr. Fink.



This picture shows the hydropower area of the dam on the upstream side (or the pool side). The hydropower area of the dam has a trash screen in front of it. This overhead trash screen system was added in 2009.

This picture shows the opening into the hydropower facility on the left side of the picture and adjacent to the three spillway gates. The hydropower facility is at the left(north) end of the dam and adjacent to the number 1 spillway gate.

Again, note all the debris in front of the gates including the hydropower facility. Since the lake is drained, there is full access to the entire area in front of the dam.

The hydropower facility was also flooded during the July flood event. Considerable clean-up effort was done by the staff since the flood event. The electrical systems in the hydropower facility were also flooded.

This is a previous inspection photo taken 2009 on the downstream end or tailwater side of the dam. The spillway gates are to the left. The hydropower facility is to the right in the photo.

As noted above, the site staff uses the wicket gates to pass flow through the dam in addition to the three spillway gates. However, the flow capacity of the wicket gates is considerably less. The turbines are deactivated.

The site staff maintains a very tight pool level in the summer. If the pool is 2/10 foot high or low, adjustments are made to the pool level through the wicket gates. During the winter, flow is passed through the wicket gates to maintain a minimal flow through the dam (250 cfs per wicket gate).

The electrical system in the hydropower facility was recently upgraded (for operating the wicket gates). The hydro facility was converted over to 480 volt electrical system from the original 208 volt system. A programmable logic controller (PLC) was also recently added along with a transducer for keeping track of water and pool levels.

C. Gate 3 Operating Issues. The vertical lift gate number 3 failed to open ***completely*** during the July 2010 flood event. Mr. Fink said the gate was able to be opened approximately 6 feet. However, measurements taken during the September 8th inspection showed the gate open only 4.3 feet. Whether the gate was open 4 feet or 6 feet, this then reduced the overall spillway capacity. However, according to Mr. Fink, each spillway gate has approximately 8000 cfs capacity and most of the capacity through the gate occurs in the initial opening of the gate. He thought probably 6000 cfs out of the total of 8000 cfs was able to be passed through the Gate 3

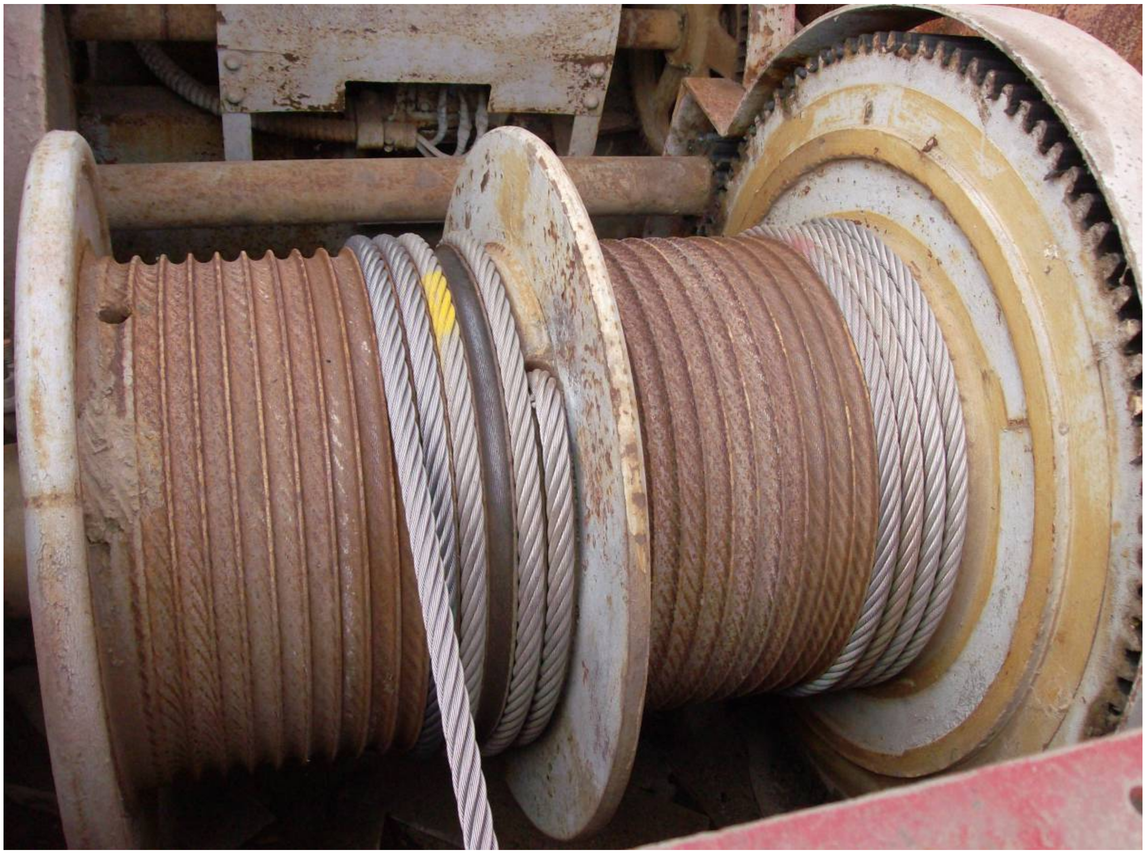
The issues with the Gate 3 have been prevalent since 2008 according to Mr. Fink. There is an embedded rail at the bottom of the gate (north side) that was damaged. The rail is shifted and binds the gate as it opens and closes.

Mr. Fink said the first 1 foot opening of the gate is the most critical and usually the gate will fully open after that. During the July flood event, the Gate 3 was unable to be opened any further then the 4 or 6 foot travel. One of the sheave blocks was actually pulled out of the concrete and broke off trying to open the gate according to Mr. Fink.

View of Number 3 Spillway Gate in the position during the September 8th, 2010, inspection

D. Machinery Details. The gate operating equipment for all three spillway gates is original as constructed machinery that went into service in 1929. The machinery was manufactured by Phillips and Davis and is noted as boom type hoist machinery. The notation on the machinery is “P&D Hoist” manufactured in Kenton, Ohio. There have been some electrical upgrades and wire rope has been replaced. The wire rope was replaced recently on Gate 3. As the age of the equipment increases, the reliability of the equipment decreases thereby increasing the probability of unsatisfactory performance. The consequences of a machinery failure with the gates in the closed position would result in the project losing or reducing the ability to make spillway releases during a flood.

This picture shows the three gate machinery housings. The housing covers the motor, gate controls, and cable drums.



This picture shows the cable drums inside the machinery housing. There are two lift points on each spillway gate.

The cables from each drum extend out from opposite sides of the machinery housing.

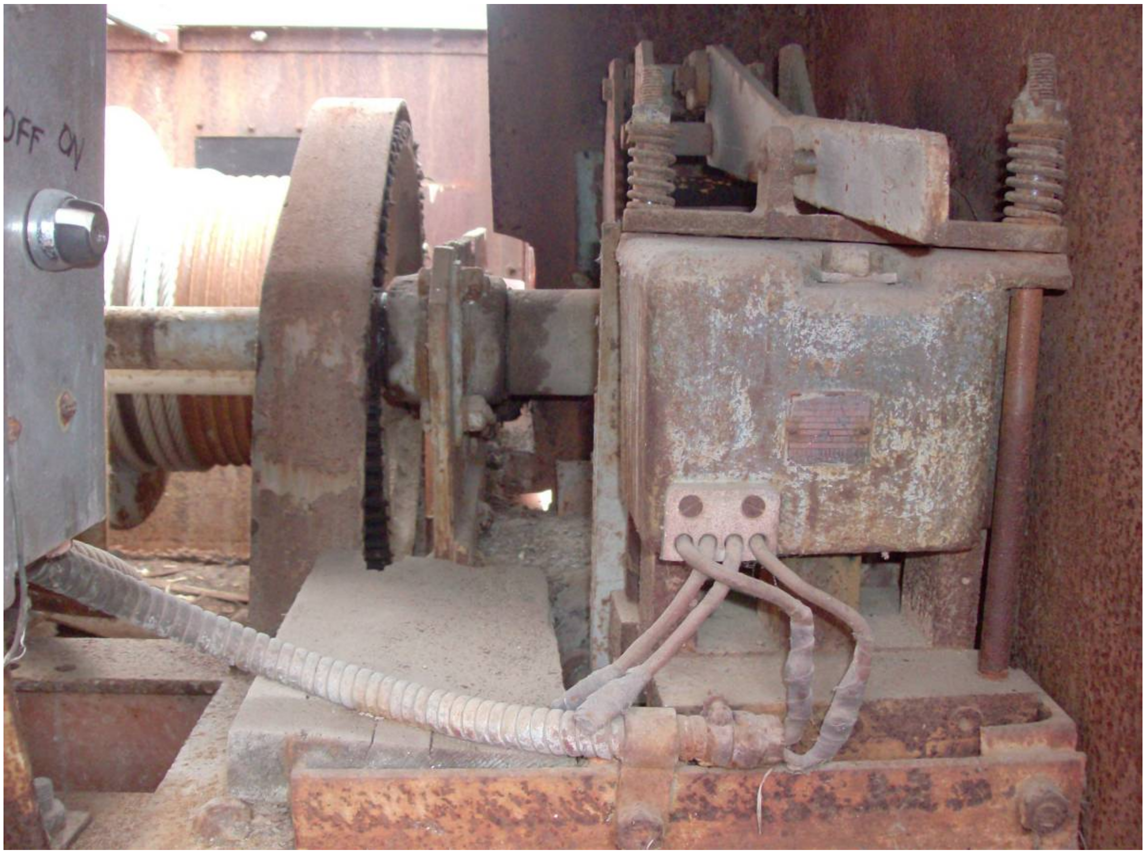
This picture shows the wire rope extending from the cable drum and the housing.

This picture shows the reeving details for the wire rope and the multiple wire rope sheaves.

Another picture showing the wire rope extending from the machinery housing to the wire rope sheaves.

View of Gate 1 wire rope looking from top and showing the connection to the vertical lift gate.

Again, each vertical lift gate has two lifting points or pick points. The reeving details and connections details are identical between the two sides of the gates.



This picture shows the inside of the machinery housing. Note the cable drum, motor, brake, and controls.

The electric motor is designed to run on 3-phase, 208 volts. The motor is original and has not been rehabilitated. Data on the motor as follows:

Manufacturer: General Electric
Horsepower: 10hp
3-phase, 60 hz, 208 volt

Mr. Fink thought the brushes in the motors may have been replaced otherwise they are original.

Since the hydropower facility got submerged during the July flood event, this also submerged the circuit breakers and electrical feeders to the spillway gates and disabled the 208 volt system to the gates. Mr. Fink stated that the 208 volt system cannot be turned back on. The dam staff has instead installed new breakers in the dam 480 volt system and installed transformers to feed the spillway gate machinery controls and motors.

This picture shows the push button station for operating the gate machinery.

This is all locally controlled at the machinery. The push buttons themselves have been replaced otherwise all the control equipment is original.

This picture shows a better detail of the gearing for the machinery units.

The cable drum gear and pinion gear are shown in this photo.

There is also another rack and pinion gear in the background of the photo.

Mr. Fink reported that the lifting speed of the gates is approximately 2 or 3 feet per minute.

E. Vertical Lift Gates.

There are a total of 3 vertical lift gates through the spillway. The gates include rollers (installed on each end of the gate) which allow the gate to be opened and closed under flow conditions. In that regard, the gates are very similar to tractor gates installed at various Corps dams. All the gates are identical and are 17 feet tall and 25 feet wide and have a total travel of 20 feet. Mr. Fink did indicate that the center gate (Gate 2) does work the best out of the 3 spillway gates. He reported no issues with this gate.

The last extensive inspection of the three spillway gates was in 2008 according to Mr. Fink.

Mr. Fink said the hydropower wicket gates are primarily used to control flow through the dam. However, for larger rain events, the spillway gates are operated. He thought the gates are generally moved once or twice a month during the summer.

Mr. Fink said the spillway gates are generally kept closed during the winter and the wicket gates used to pass flow. He said the winter shut down usually occurs in late December or early January. For the spring start-up, Mr. Fink said he usually tries to have the spillway gates deiced and ready for operation by mid-March or early April. He said the site usually tries to keep one spillway gate deiced throughout the entire winter if possible. This is done by using agitators, mixers, and heaters. He said they used to use fly ash from a nearby power plant but that cannot be done anymore.

Debris building up against the front side of the dam was reported to be a major issue. The site staff remove and deal with debris on a continual basis. This includes trees and vegetation. As part of the new rehab work proposed (see the discussion below), the gates were going to be designed to raise 3 feet to 4 feet higher than current design. Mr. Fink indicated that in 2008 five pontoon boats got stuck in the spillway gates. The bottom seals constantly have debris hitting them according to Mr. Fink.

Mr. Fink indicated that rehabilitation work on the gates and machinery was just about to commence as the flood happened. All the machinery was going to be replaced. This was a contract with Steel-Fab.

The rehab work was going to completely revise the machinery design. New screw type actuators were to be installed. These actuators would lift and lower the gates from the top instead of the bottom. The top of the gates were to be structurally modified and strengthened.

Other work included in the rehab was replacing the side seals and bottom seals on the gates. A completely new 480 volt electrical system was to be installed and a new control system installed for operating the new hoist units. The new work included fabrication of new bulkheads for installing in front of the spillway gates. This would allow the individual gate bays to be dewatered.

This picture shows the vertical lift gates and the hydraulic cylinders installed on top of the gates. The dam operators use the hydraulic cylinders to help seat the gate in place after they are closed by the machinery. The I-beam is set on the underside of the concrete walkway and the cylinder is then extended. This forces the gate down into the seating position on the sill.

This picture shows the rollers on the end of the lift gates. The rollers on both gates 1 and 2 were inoperable and severely corroded. It was reported that the rollers on gate 3 were recently replaced. The site had one set of rollers in the hydro facility ready to be installed on one of the spillway gates. New rollers are stainless steel. Note all the zebra mussels on the gate and rollers. Even with the rollers severely corroded on gates 1 and 2, the site is still able to move the gates up and down. The rollers assist in opening and closing the gate under flow conditions.

This picture is also looking towards the pool side (lake side) of the dam. Note all the docks in the background.

F. Electrical Power

Commercial power is the primary power to the site. The site operations staff indicated the commercial power is fairly reliable. Outages were noted to be very brief.

It was reported the current electrical system utilizes 480 volt, 3 phase power. The circuit breakers are located in the hydropower house. The 208 volt system has been disabled. The 208 volt system was damaged and submerged during the July flood event and was not restored. The 480 volt system was repaired.

The site currently has a LP generator to back up the 480 volt electrical system. This generator was not observed. Mr. Fink said the site used to have a diesel generator.

Power line and transformers to dam structure

G. Probability of Failure of Mechanical/Electrical Components and System

Calculations of probability of failure of the mechanical and electrical systems to operate satisfactorily can be developed. This can be done using fault tree analysis software which uses Dormant-Weibull formulas and tools developed by the Corps of Engineers Risk Management Center mechanical/electrical methodology team (Excel Spreadsheet). The probability of failure to open various combinations of gates can be calculated using the software. The software only analyzes the mechanical and electrical equipment. The issues with Gate 3 appear to be structural and not mechanical or electrical.

The fault tree analysis method uses a combination of gates arranged in a tree format. Each components probability of failure is calculated in the fault tree and the software provides the overall probability of failure of the system. Input data used to determine probability of failure include the components characteristic life, actual age, component condition, environment, inspection/operation intervals, stress and temperature.

Characteristic life for various components was determined from a panel of experts from around the Corps in an expert elicitation.

Condition scenarios can be developed using the fault tree analysis. All 3 sets of machinery are generally in the same condition. The only significant differences are the replacement of wire rope and gate rollers has varied. The usage between all 3 sets of machinery varies slightly.

The fault tree program can provide the probability of failure to open 1gate, 2 gates, or 3 gates or any combination.

APPENDIX I

WINDAM ANALYSIS

WinDAM[1] Analysis of Delhi Dam Overtopping Event 7/24/2010

Overview:

The purpose of this analysis is to determine if the dam embankment would have likely failed as a result of the overtopping flow without the contributing effects of the piping. The analysis was completed by using a beta test version of WinDAMb developed by the USDA Agricultural Research Service in cooperation with the USDA Natural Resources Conservation Service and Kansas State University. The headcut migration model within WinDAMb is based on laboratory and dam breach physical model studies.

Model Development:

The first step in the analysis was to develop a WinDAM model that simulated the overtopping flows of 7/24/2010. The geometric and hydrologic data input sources are summarized below:

- The inflow hydrograph, spillway rating curves, starting reservoir water surface elevation and tailwater rating curves were obtained from the HEC-RAS Maquoketa River Model from US 20 to the City of Hopkington, developed by J. Garton at the Iowa Department of Natural Resources.

- The stage-storage data was obtained from the Ashton Engineering Inspection Report, 2002

- The geometry of the embankment was obtained from historical plan and cross-section drawings of the project

- The overtopping section of the embankment was estimated at 120 feet from a comparison of the drawings and the photographs of the event.

The resulting WinDAM model was run without considering the breach potential of the embankment. The WinDAM model results were compared to the HEC-RAS model to match the maximum overtopping flow depth and duration. The differences in the models are due to the routing limitations of WinDAM. WinDAM will not model opening and closing of gates or the dynamic reservoir routing included in the HEC-RAS model. The WinDAM model was calibrated by making slight adjustments to the elevation of the embankment. After calibration, the WinDAM model overtopped for 13.5 hours with a

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[1] WinDAMb Integrated Development Environment (Beta Version 2009.11.16), Developed by USDA and USACE in cooperation with Kansas State University (KSU), Copyright (2009) by USDA, USACE, KSU, and SNL

maximum flow depth of 1.36 feet, while the HEC-RAS model overtopped for 16.0 hours with a maximum flow depth of 1.37 feet.

The next step in model development was to select the embankment strength and erodibility characteristics. A summary of how these characteristics were selected is shown below:

- Soil Test Results – PI is 8.7, percent clay is 15%, representative diameter (D75): 0.14 mm = 0.006 inches
- A range of values for Kd and τc were selected from the tables shown below (Tables obtained from ARS WinDAM Beta Testing Workshop, September 2009)

Table for Estimating Kd (Erodibility Factor):

	Modified Compaction (56,000 ft-lb/ft^3)		Standard Compaction (12,000 ft-lb/ft^3)		Low Compaction (4,000 ft-lb/ft^3)	
% Clay	≥ Opt	Dry	≥ Opt	Dry	≥ Opt	Dry
> 25	0.02	0.5	0.05	1	0.1	5
10-25	0.2	1	0.5	5	1	10
5-10	2	10	5	50	10	100
0-5	80	200	80	200	200	200

- Kd estimated range between 1 and 10

Table for estimating τc (Critical Shear Stress):

	Modified Compaction (56,000 ft-lb/ft^3)		Standard Compaction (12,000 ft-lb/ft^3)		Low Compaction (4,000 ft-lb/ft^3)	
% Clay	≥ Opt	Dry	≥ Opt	Dry	≥ Opt	Dry
> 25	1	0.002	0.2	0.004	0.04	0.0
10-25	0.01	0.0	0.002	0.0	0.004	0
5-10	0.001	0	0	0	0	0
0-5	0	0	0	0	0	0

- τc estimated range between 0.004 and 0 psf. Since $\tau c < 0.1$ assume $\tau c = 0$ since model results will be the same.

- Total Unit Weight – estimated @ 115 lb/ft^3 (erosion model is not sensitive to this parameter)

- The undrained shear strength is defined as one half the unconfined compressive strength. Estimated unconfined compressive strength is 1000 psf (threshold between soft and firm soils – Terzaghi & Peck), so undrained shear strength is 500 psf. A range between 375 and 625 psf was used in the model

- Vegetal Inputs

  - Upstream slope is grass covered, assign SCS Retardance Class C, Curve Index = 5.60
  - Downstream Slope covered with trees and undesirable vegetation assign Maintenance Code of 3, Manning's n=0.08, and Vegetal Cover Factor = 0
  - Crest is pavement, assign Manning's n=0.025

Results:

Nine model runs were completed using the range of Kd and undrained shear strengths discussed above. In all cases, the models predicted breach of the dam, with varying breach formation times. A summary of the results is shown below:

WinDAM Model Results			
Kd (erodibility factor)	Undrained Shear Strength	Maximum Overtopping Depth	Breach Formation Time
	(psf)	(feet)	(hours)
1	375	1.35	5.75
1	500	1.35	8.25
1	625	1.36	8.75
5	375	0.96	0.50
5	500	1.01	0.50
5	625	1.05	1.00
10	375	0.83	0.25
10	500	0.82	0.25
10	625	0.84	0.25

These results suggest that the dam would breach due to the overtopping flow of 7/24/2010 without the contributing effects of piping. Additional runs were completed to determine the threshold soil properties where the observed overtopping flow would not cause failure of the embankment. The threshold occurs approximately where Kd = 0.70

and the Undrained Shear Strength = 1000 psf. These strength parameters exceed the probable values of the failed embankment section. These higher strength erodibility parameters would only be expected in an embankment with significantly higher clay content compacted to 95% Standard Proctor.