Independent Panel of Engineers

Report on Breach of Delhi Dam

December 1, 2010

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I.
Purpose and Scope

Delhi Dam breached on July 24, 2010 after several days of intense rain in the watershed above the dam. In response to this event, the Governor of the State of Iowa requested assistance from the National Dam Safety Review Board in providing an Independent Panel of Engineers to evaluate the cause of the overtopping and breach of Delhi Dam. This request was made to the Administrator of the Federal Emergency Management Agency (FEMA) dated August 6, 2010. The National Dam Safety Review Board includes representatives from federal and state agencies as well as a member from the private sector and operates under the direction of FEMA. The National Dam Safety Review Board is statutorily established under the Dam Safety Act of 2006 (Public Law 109-460). The National Dam Safety Review Board provides the Director of FEMA with advice in setting national dam safety priorities and considers the effects of national safety policies affecting dam safety.

In the August 6, 2010 letter, the state of Iowa identified the scope of the Independent Panel of Engineers review as follows:

- Review the operational characteristics of the project leading up to the breach of the upper reservoir.
- Perform an evaluation of the breach of the dam to determine the specific failure mode.
- Submit a final report documenting the results of their findings on the cause of the breach of the upper reservoir and the important lessons learned from

In a letter from the Deputy Administrator of FEMA to the Director of the Iowa Department of Natural Resources dated August 27, 2010, a commitment was made to convene a three member Independent Panel of Engineers (IPE) under the auspices of the National Dam Safety Review Board. The three members represent federal agencies with extensive experience in dam safety and include:

William Fiedler, Bureau of Reclamation
Wayne King, Federal Energy Regulatory Commission
Neil Schwanz, U.S. Army Corps of Engineers

In addition to the IPE, the Lake Delhi Recover and Rebuild Task Force was created by the Governor of the State of Iowa by Executive Order Number Twenty-Five on August 6, 2010. The Task Force was created "to assist in the collaboration of citizens and businesses with local, county, state and federal agencies, to develop strategies for both the recovery and rebuilding of the Lake Delhi area, including, most specifically, whether and under what conditions the

Delhi Dam should be rebuilt. While operating under parallel schedules the IPE and the Task Force have different objectives and separate and independent reports have been produced by the two groups.

In order to fulfill its mission, the IPE initially collected and reviewed key information. The IPE operated independently and access to individuals and to any requested information was freely granted. Areas of focus included: the design and construction of Delhi Dam, subsequent modifications to the dam, the operational and performance history of the dam, past examinations and reviews of the dam, the timeline of events leading up to and including the breach of Delhi Dam and the emergency response to the dam breach. A key activity for the IPE was convening in Iowa during the week of September 6th, 2010. On September 7, 2010 the IPE reviewed records at the Iowa Department of Natural Resources Offices in Des Moines, Iowa and conducted interviews with personnel from the Department of Natural Resources, dam operators, owner's representatives and local residents. On September 8 and 9, 2010, the team inspected the damsite and the upstream and downstream areas and conducted additional interviews with personnel from local government agencies and from the Lake Delhi Recreation Association. The team spent September 10, 2010 in Des Moines at the Department of Natural Resources Office and reviewed additional records and conducted additional interviews.

The findings of the IPE are included in this report. Supporting the findings are report sections that provide a summary of the key information collected by the team and evaluations performed by the team and others. The IPE was supported by a number of individuals in performing their work. These contributions were critical to the overall report and the IPE is grateful to the following individuals:

Angela Damron - Civil Engineer, FERC, Chicago Office – travelled with IPE during site visit and assisted team with note taking, interviews and hydraulic analyses.

Scott Airato FERC, Chicago Office performed hydraulic analyses of the flood event at Delhi Dam at the request of the IPE.

William Brown FERC-Atlanta Office performed the embankment overtopping erosion analysis.

Tim Paulus – Mechanical Engineer, U.S. Army Corps of Engineers, St Paul District – inspected mechanical equipment at Delhi Dam and authored the findings on the mechanical equipment.

Lori McDaniel – Flood Plain Management and Dam Safety Manager, Iowa DNR – coordinated visit by IPE to Iowa, set up interviews, collected information requested by the team.
Jonathan Garton – Civil Engineer, Iowa DNR – provided additional coordination during IPE visit to Iowa, served as a follow-up contact after site visit for collecting additional information.

Peer Reviewers
Charles Wagner FERC Regional Engineer Atlanta Regional Office Division of Dam safety and Inspections
William Engemoen Senior Technical Specialist, Bureau of Reclamation
Joseph P. Koester, PhD, PE, US Army Corps of Engineers, Geotechnical and Materials Community of Practice Lead
Duane Stagg PE, US Army Corps of Engineers, Mississippi Valley Division Dam Safety Program Manager

In the reading of this report all elevations are referenced to the 1929 National Geodetic Vertical Datum. Conversion to local datum is as follows:

NGVD29 = Local Datum + 774.8

This yields the following elevations.

Top of dam:	EL 904.8 NGVD29 = EL 130.0 ft local datum
Normal Reservoir:	EL 896.3 NGVD29 = EL 121.5 ft local datum
Top of core wall:	EL 898.8 NGVD29 = EL 124.0 ft local datum

II.
Description of Dam and Operations

A. Location and General Description

1. Location

Delhi Dam is located on the Maquoketa River about 1.4 miles south of the town of Delhi, Iowa (Figure II-1). The Maquoketa River, located in northeastern Iowa, is a tributary of the Mississippi River. The dam was constructed between 1922 and 1929 by the Interstate Power Company for hydroelectric power generation. Generation of power was eventually terminated at the dam in 1968 [Allen, 2009]. The dam is currently owned and operated by the Lake Delhi Recreation Association (LDRA).

2. General Description

Delhi Dam, also known as Hartwick Dam, was designed as a concrete dam and earthen embankment. The 704-foot long structure consists of (from left to right looking downstream): a 60-foot long concrete reinforced earthfill section abutting the left limestone abutment ; a 61-foot long conventional reinforced concrete powerhouse containing two S. Morgan Smith turbines with two Westinghouse generators (each rated at 750 kW); an 86-foot long gated concrete ogee spillway, with three 25-foot x 17-foot vertical lift gates; and, a 495-foot long embankment section that was originally constructed with 1V:3H upstream slopes and 1V:2H downstream slopes, that extends to the right abutment of the dam (in this report when right and left is used in reference to the dam, the convention is that this is while looking downstream; also the right abutment of the dam is the south abutment and the left abutment is the north abutment**)**. The crest of the south embankment section of the dam is 25 ft wide and the dam crest is at elevation 904.8 ft NGVD29. A general plan of the site is shown on Figure II-2.

The maximum section of the concrete portion of the dam has a height of about 59 ft and the embankment section has an estimated maximum height of 43 ft. Lake Delhi, the reservoir behind Delhi Dam has an area of approximately 440 acres and a storage volume of 3790 acre-ft at normal reservoir (elevation 896 ft) and a reservoir volume of about 9920 acre-ft at the crest of the dam (elevation 904.8 ft) [Allen, 2009]. The spillway crest is at elevation 879.8 and the hollow inside of the spillway crest structure is filled with rock.

The concrete reinforced earthfill section of the dam at the left abutment was originally constructed with two parallel concrete retaining walls, founded on rock and spaced 20 ft apart. Rock fill was placed between the walls. In 1967, a concrete crib wall and additional fill was placed upstream of the original walls. The area downstream of this section serves as a parking and staging area for performing maintenance in the powerhouse [FERC 2002 Prelicense Inspection]

3. Hazard Classification

The hazard classification for Delhi Dam is uncertain and is reported differently in separate documents. Delhi Dam was classified as a moderate hazard structure in the last dam safety inspection report [Allen, 2009], based on its importance as a private recreational structure. In the FERC 2002 Prelicense Inspection Report the inspector classified the dam as having a high hazard potential due to the downstream population at risk [FERC 2002 Prelicense Inspection Report]. In an earlier inspection report it was concluded that the gated spillway can just handle the 100-yr flood but cannot handle the 0.5 PMF flood [Ashton, 1998]. The Ashton Engineering Report [Ashton, 1998] concluded that the Delhi Dam was a low hazard(potential) structure

B. Embankment Dam

1. Embankment

Little information is available regarding the original design, construction (materials and procedures) and foundation information of the Delhi Dam embankment section. As indicated previously, the original cross section of the embankment dam was constructed with an upstream side slope of 1V on 3H, a downstream side slope of 1V:2H and a 25 ft top width. A significant portion of the embankment dam was later widened. The general cross section presented in Figures II-2 and II-3 suggest that the embankment dam was originally constructed using one fill material while incorporating a concrete core wall upstream of the dam centerline for seepage control. A surface sample of remaining embankment material exposed by the breach was obtained during a post breach inspection. A mechanical analysis, including hydrometer, and Atterberg limits testing were performed on this sample and testing classified the sample as a sandy lean clay (CL). About 30% of the sample was sand with 100% passing the #4 sieve and about 70% passing the #200 sieve. The liquid limit was 25.9 with a plasticity index of 8.7 (See Section IV Geotechnical Considerations).

2. Concrete Core Wall

The concrete core wall is shown to be founded directly on bedrock for a distance of 20 ft from the right abutment of the gated spillway. To the south (right looking downstream) of that point the core wall is founded on steel sheet pile driven to varying depths. For some distance it is expected that the sheet pile was driven to the top of bedrock and it is stated in an inspection report that the embankment has a concrete cutoff wall founded on steel sheet pile driven to rock [Ashton, 1998]. Lack of foundation information prevents direct determination of the top of rock location, verification that the sheet piles were driven to rock and determination of the distance from the gated structure to the point where the sheet pile fully penetrated.

The concrete core wall is located about 12 ft upstream of the upstream embankment dam crest shoulder and extends to the base of the embankment where it is founded on bedrock adjacent to the gated spillway or on steel sheet pile 20 ft south of that point. Photos II-1 and II-2 show the core wall during construction and photo II-3 shows the post breach condition of the core wall at the sheetpile contact. A remnant of the core wall remains attached to the gated structure (likely connected with reinforcing) while sheet pile remains in place where the concrete monolithic stem has broken away (embedded only, without through reinforcement or shear studs). The top of the concrete core wall is at elevation 898.8 ft, 6 ft below the crest of the dam. Project drawings show the upper 12 inches of sheetpile embedded into the concrete core wall. The sheetpile was placed in three sections of different pile lengths. Project drawings show the start of the sheetpile at a distance of 20 ft south of the southern end of the concrete spillway. From that point to 127 ft south of the spillway, the sheetpile is described in older reports as 30 ft long, then 87 ft of sheet pile with 25 ft of penetration, and lastly 127 ft of sheet pile with 20 ft of penetration. From original construction photos, the source of the embankment soils where less than a half mile south of the right (south) abutment.

3. Erosion Protection

Erosion protection was provided using riprap and vegetative cover. Riprap was placed in a narrow band on the upstream slope likely protecting the embankment within the anticipated range of reservoir elevations and wave action. Additional rock was placed by the in 2009[Mohn,2009]. Riprap was also placed on the downstream slope near the gated spillway. Photo II-4 identifies the existence of this original riprap as an emergency spillway. Existing ground surveys from the 1967 Delaware County road project indicate a dip in the crest of the dam estimated to be about 1 ft lower than the crest elevation with a bottom width of about 50 ft and 1V on 50H side slopes. The photo shows the placement or riprap erosion protection but the riprap is not shown to extend to the toe of the dam. This spillway was subsequently filled during the 1967 project and it is unknown whether the riprap was removed during this work. A berm, approximately 60 ft wide, was also constructed at the downstream face of the embankment in 1967 when the county road across the top of the dam was realigned as shown in Figure II-2. The top of the berm was several feet below the crest of the dam and is used as a parking lot [Allen, 2009]. Figure II-4 presents the existing ground contours prior to work by the LDRA in 2009. The 1967 embankment widening did not extend completely to the gated spillway and the downstream slope adjacent to the spillway remains at about 1V on 2.2H (estimated from Mohn 2009 plans).

4. 2009 Embankment and Berm Modifications

LDRA construction in 2009 included adding fill on the upstream embankment slope to create an access road and working pad at EL 898.0 ft. Work was also completed on the downstream slope area that included constructing an access road to a working platform at EL 864.0ft near the downstream right wingwall of the gated spillway (see Figure II-4). This work included placement of rock and likely removal of woody vegetation on the downstream slope. Photo II-5 shows the trees and vegetation on downstream embankment slope. Significant trees and overgrowth remained on the slope between the gated spillway and the 2009 LDRA project.

C. Operations

1. Hydropower

The hydropower at the site was deactivated in 1968. Hydropower at the site consists of two turbines with each turbine fed by 16-foot diameter penstocks. In addition to the turbines, there are two wicket gates within the powerhouse. The wicket gates can pass smaller flood inflows through the dam. The wicket gates are 5 ft in diameter and can each pass about 250 ft^3/s, at reservoir water surface elevation 896 ft.

2. Spillway Gates

The spillway gates at Delhi Dam are operated to control the reservoir water surface at elevation 896 ft, the normal reservoir elevation, for as long as possible during a flood. The spillway gates are numbered from left to right looking downstream. Gate 1 is the left most gate (also the north most gate). The typical sequence of opening the gates is to open Gate 3 first up to full opening, then Gate 2 to full open and finally Gate 1 to full open. The spillway gates are opened in response to increases in inflows into the reservoir. The spillway has a capacity of about 32,000 ft^3/s, with the reservoir at the crest of the dam, elevation 904.8 ft. The spillway gates are typically operated every year to pass the spring runoff [FERC 2002 Prelicense Inspection Report]. The primary power to the site is supplied by commercial power. This source is generally reliable. The current electrical power at the site is 480-volt, 3-phase power. The 208 volt system at the site is no longer functional. An LP generator is also provided as backup to the 480 volt incoming power

The spillway gates are generally kept closed during the winter months. The winter shutdown typically occurs in late December or early January.

Attempts are made by the Lake Delhi Recreation District personnel to keep the gates deiced during the winter. This is accomplished through the use of agitators, mixers and heaters. The gates are deiced and ready for spring operation by mid-March or early April.

Debris at the spillway control structure has been an issue and is something operating personnel deal with on a regular basis. During the July 2010 flood event, several boats passed through the spillway structure with at least one boat becoming trapped underneath Gate 2.

The modified permit that was issued by the Iowa Natural Resources Council after the dam was transferred to the Lake Delhi Recreation Association included some special requirements for regular assessments of the dam:

> "Be it further ordered that the permittee designated by this order, or its heirs, assigns or successors in interest to said dam and appurtenances location in Sections 29 and 30, T88N, R4W, Deleware County, Iowa, shall cause a structural and operational assessment by a qualified professional engineer, registered in the State of Iowa, to be made of such dam and appurtenances and filed with the Iowa Natural Resources Council on or before July 1, 1977, and at five year intervals thereafter."

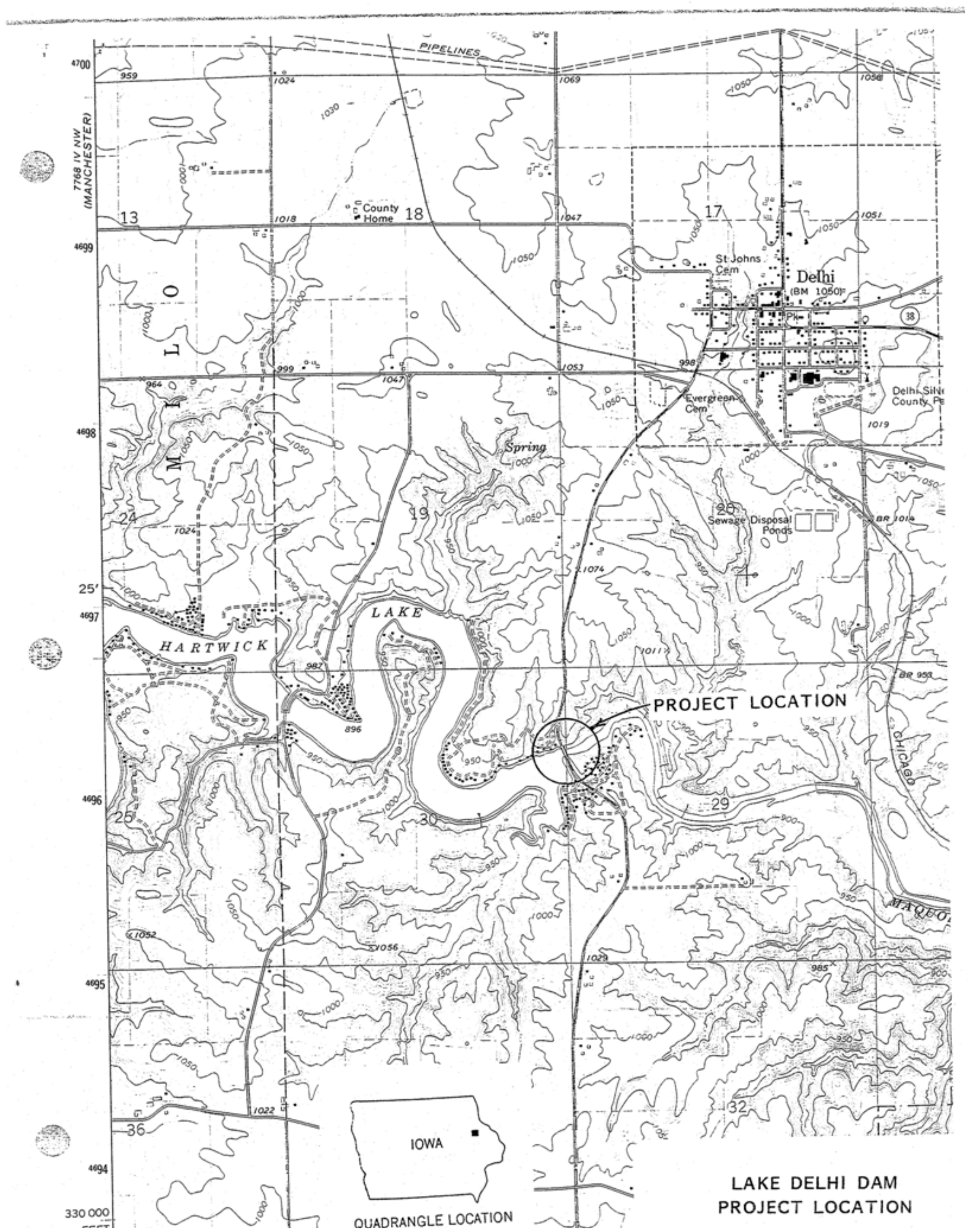


Figure II-1.
Project location

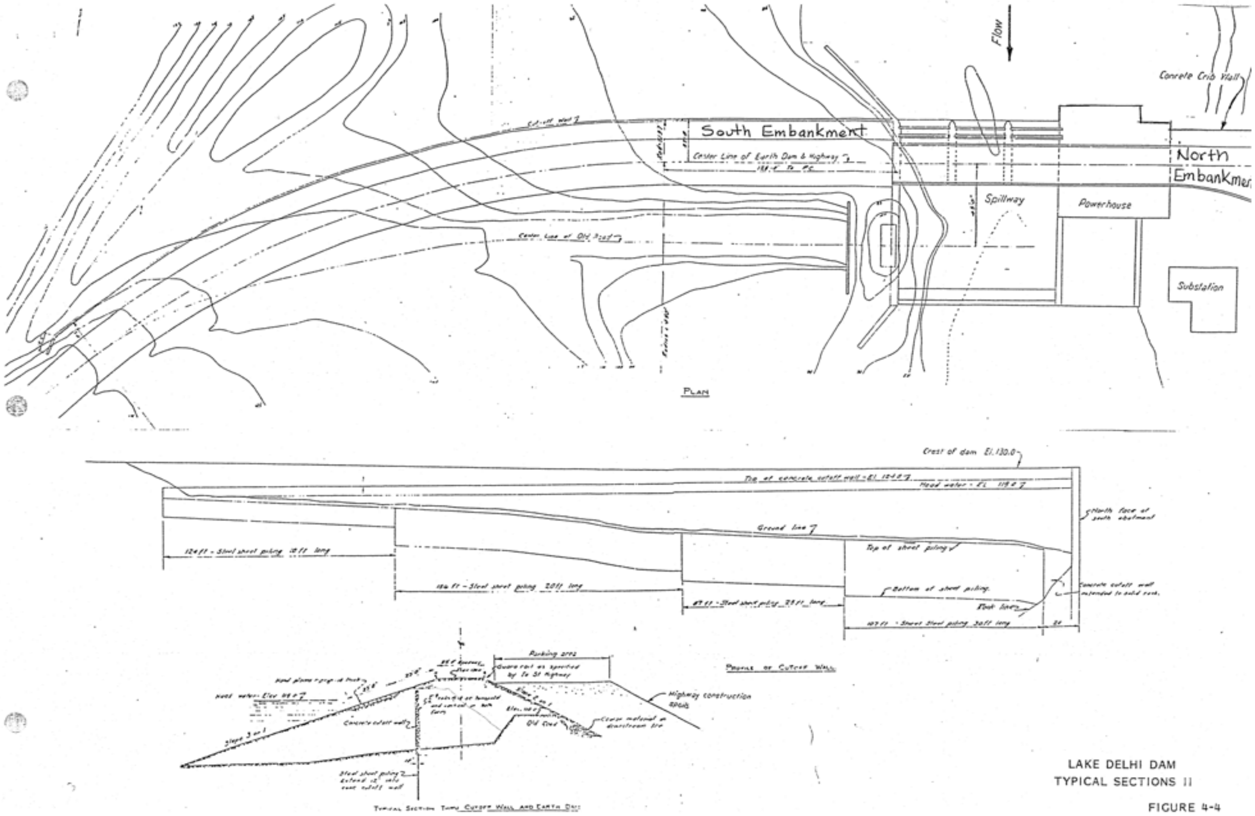


Figure II-2.
General plan and typical embankment section.

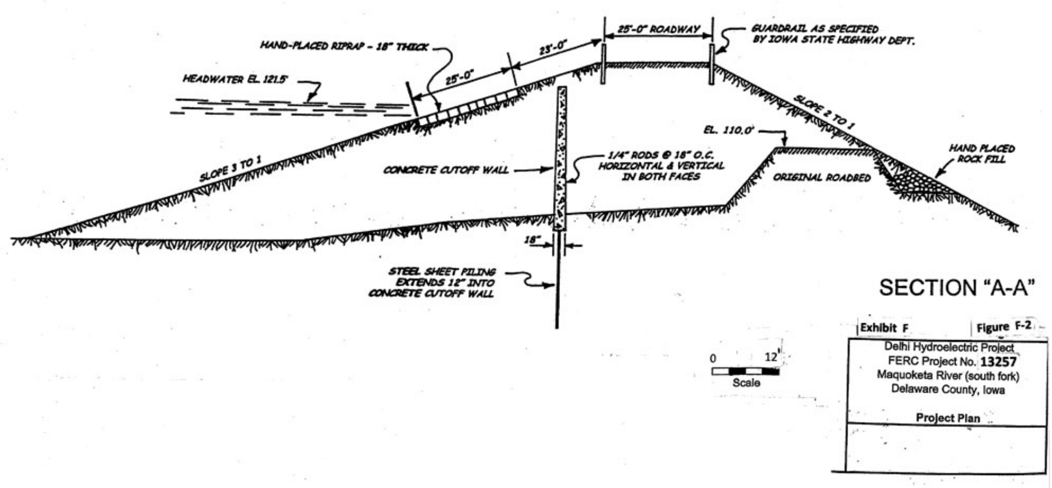


Figure II-3.
Embankment dam section.

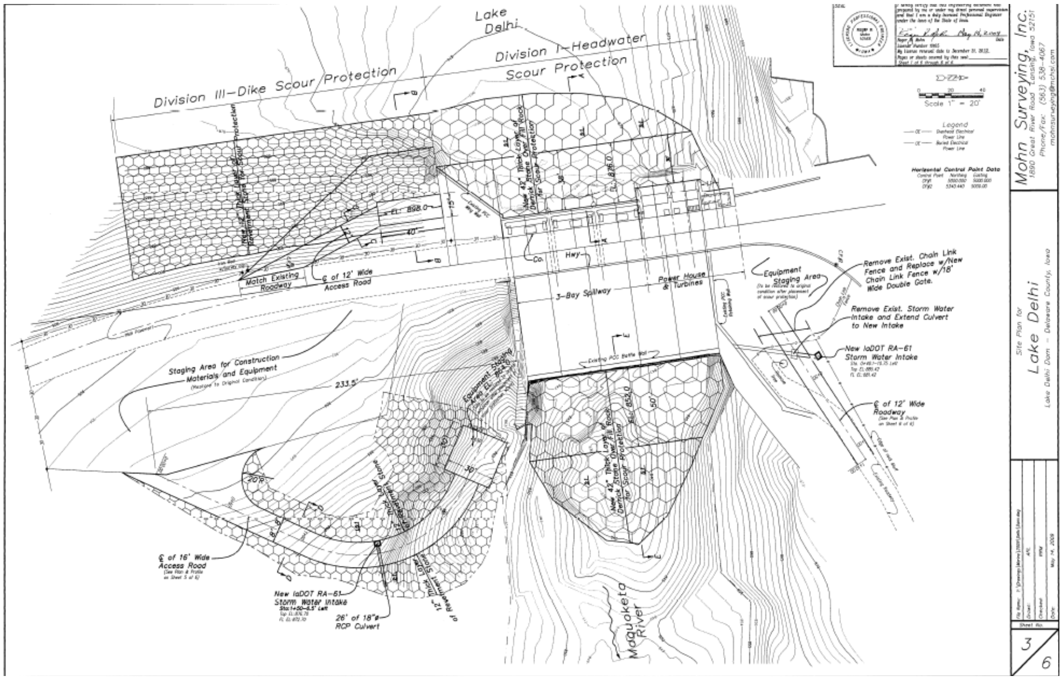


Figure II-4.
Plan showing widened embankment section right of gated spillway.

Photographs

Photo II-1
Upstream side of dam during construction
Note corewall and right embankment construction.

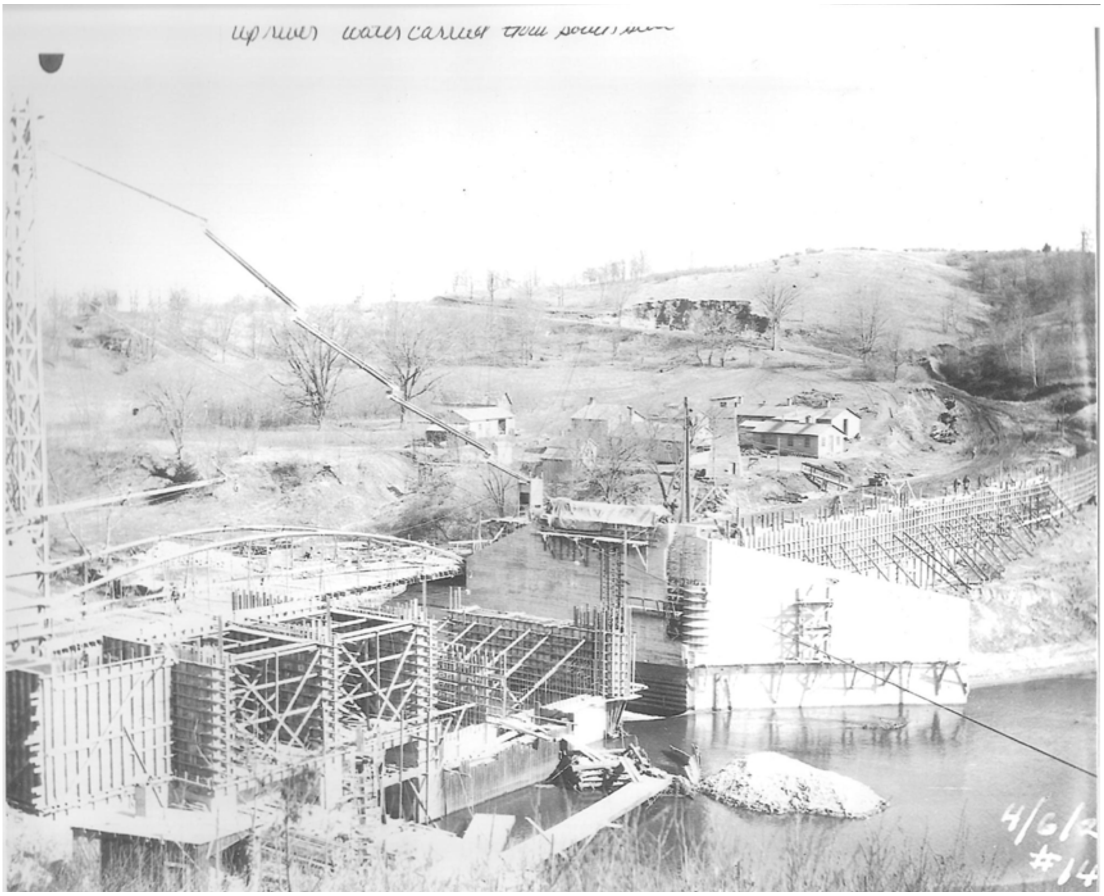
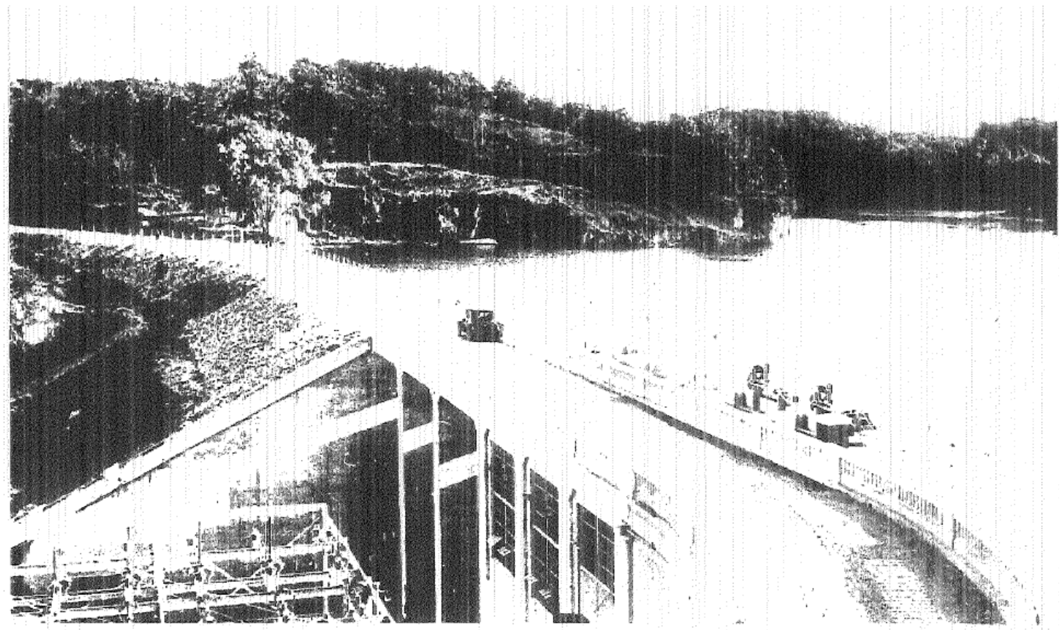


Photo II-2
Embankment core wall construction
Note corewall and right embankment construction.

Photo II-3.
Breach area
Note Embankment core wall remnant at gated spillway and sheet pile.



Power dam across the Maquoketa River south of Delhi. *(Photo courtesy of Delaware County Historical Society)*

When built

Photo II-4.
Riprap lined emergency spillway adjacent to gated spillway.

Photo II-5.
Vegetation on the downstream embankment slope

III

Reservoir Operations History

A. Previous Flood Events

The maximum recorded river flow is reported as 28,905 ft^3/s and occurred on 6/15/1925 [FERC 2002 Prelicense Inspection]. There have been several large flood events at Delhi Dam prior to the July 2010 event. These events as well as the July 2010 event are highlighted below.
2002 Flood Event

1. 2002 Flood Event

The peak inflow into Lake Delhi, as measured at the USGS(United States Geological Survey) gaging station (05416900) on the Maquoketa River at Manchester, Iowa was 10,800 ft^3/s on June 4, 2002 [USGS Website]. The 2002 flood had the greatest impact downstream of Delhi Dam. Peak discharges at the Maquoketa River at Monticello of 45,000 ft^3/s were recorded, which has been estimated to be represent a recurrence interval greater than 500 years [USGS 2004]. The maximum water reservoir water surface for this flood event is unknown."

2. 2004 Flood Event

The peak inflow into Lake Delhi, as measured at the USGS gaging station (05416900) on the Maquoketa River at Manchester, Iowa was 26,000 ft^3/s on May 23, 2004 [USGS Website]. At the time, the inflow at the Manchester gaging station was estimated to have a recurrence interval of about 100 years and was the largest known flood in the upper part of the Maquoketa River Basin at the time. The maximum reservoir water surface at Delhi Dam was elevation 898.3 NGVD29 which is 2 ft above the normal reservoir elevation and 6 inches below the top of the core wall.

3. 2008 Flood Event

The peak inflow into Lake Delhi, as measured at the USGS gaging station (05416900) on the Maquoketa River at Manchester, Iowa was 22,100 ft^3/s on May 26, 2008 [USGS Website]. In 2008, five pontoon boats got stuck in the spillway gates, according to Mr David Fink of the Lake Delhi Recreation Association. The maximum water reservoir water surface for this flood event is unknown."

B. 2010 Flood Event

The peak inflow into Lake Delhi, as measured at the USGS gaging station (05416900) on the Maquoketa River at Manchester, Iowa was about 25,000 ft^3/s on July 24, 2010 [USGS Website]. A detailed analysis of the July 2010 flood is presented in Section V of this report

One of the considerations during the July 22-24 flood event on the Maquoketa River is the breach of the rock dike at Quaker Mill Dam. Quaker Mill Dam is located on the Maquoketa River about 3 miles upstream of the USGS gaging station at Manchester, Iowa (see Photo III-1). The dike at Quaker Mill Dam had breached previously on April 25, 2008. The dike was rebuilt in 2010 and the top of the structure was placed 5.5 ft below the original elevation of the dike. The new crest elevation allowed for flow over the dike when 2 ft of water was flowing over Quaker Mill Dam. The dike had a 6:1 downstream slope and a 2:1 upstream slope. The top of the dike was 8 ft wide. Engineering fabric was placed on top of the dam remnant and then 2 ft of riprap was added on top.

The Quaker Mill Dam Rock Dike breached Friday, July 23. Photo III-2, according to Anthony Bardgett Delaware County Engineer;, shows, the dike in the process of breaching and the photo was taken at about 7:00 pm on Friday evening. The rock dike breach was complete sometime Friday. There was speculation that the failure of the Quaker Mill Rock Dike contributed significantly to inflows at Delhi Dam and may have been a factor in the overtopping breach at Delhi Dam. This is not believed to be likely for several reasons. A review of the flood hydrograph at the Manchester USGS gaging station reveals no significant spike in flow on Friday through Saturday. Secondly, the travel time from the Quaker Mill Rock Dike to Delhi Dam is estimated to be about 6-8 hours, so if there was incremental increase in flood inflow, it would have past Delhi Dam well before the peak inflow occurred at Delhi Dam. Thirdly, it is believed that some of the flow that passed through the breach of the dike at Quaker Mill would have been passed over Quaker Mill Dam instead, offsetting an incremental effect of the dike breach.

C. General Spillway Gate Operations

The spillway gates at Delhi Dam were difficult to open and close. According to Mike Russell, a former operator at the dam a small crane had been used previously to sometimes initiate opening of the gates. A jacking device was installed on the top of the gates to force the gates down to their fully closed position (see Photo III-3). During the 2010 flood event, a crane was onsite and was considered to be utilized to open Gate 3, which was stuck at a 4.25 foot opening (Based on field measurement taken during the IPE inspection which differs from the operators log book which shows the opening to be 6 ft) The crane was located at the right abutment of the dam and once settlement of the dam was observed on Saturday morning July 24, the decision was made that it was too dangerous to take the crane across the crest of the embankment dam. Damaged concrete behind the left gate guide for Gate 3 is a likely cause of Gate 3 not operating to full opening during the July 2010 flood. This area was identified in a 2009 inspection of the dam and does not appear to have been repaired before the July 2010 flood (see photos III-4 and III-5, taken from an inspection report from Stanley Consultants dated October 13, 2010.

Another factor that can affect the release capacity through the spillway gates is the potential for debris plugging the spillway gates. Debris, in the form of woody vegetation, has been reported to be a common occurrence at the spillway control structure. Boats on Lake Delhi have also become unanchored during at least two floods and have been passed through the spillway gates. Video of the July 2010 flood show several boats, Liquid Petroleum Gas tanks and other debris passed easily through Spillway Gates 1 and 2, creating additional potential for gate blockage.

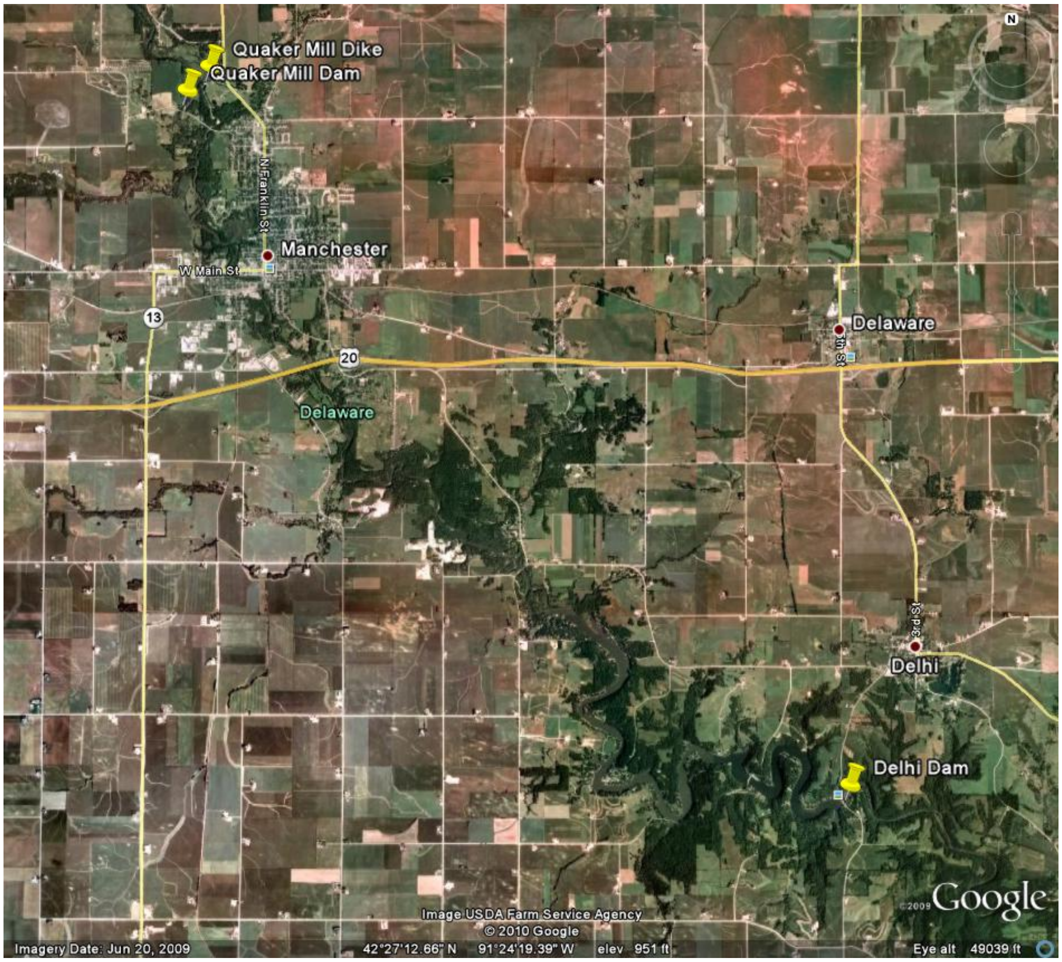


Photo III-1
Aerial View Showing Relationship of Quaker Mill Dam and Dike to Delhi Dam
(Google)

Photo III-2
Quaker Mill Rock Dike in Process of Breaching on 7/23/2010

Photo III-3
Jacking device on top of each spillway gate to force gate into closed position Beam would be place under structural concrete to force the gate down the last few inches

Photo III-4
Damage to spillway pier adjacent to Gate 3 left side guide[Stanley 2010]

Photo III-5
Damage to spillway pier adjacent to Gate 3 left side guide
[Stanley 2010]

IV

Geotechnical Considerations

A. Design Considerations

Construction of the Delhi embankment dam in the latter 1920's pre-dates several critical design considerations that became known with increased research and dam construction projects and with expanded periods of record of constructed dams. As dams are aging and owners are moving into dam rehabilitation there are geotechnical design aspects that are currently followed that were not in place at Delhi Dam. These are discussed throughout the following sections.

1. Embankment and Foundation

1.a.. Foundation Stratigraphy.

There is little recorded information regarding embankment and foundation conditions. Original construction drawings identify bedrock beneath the gated spillway and partway beneath the embankment dam (see the rock line shown on Figure II-2) but no information is provided on the location of bedrock following the embankment dam alignment or information describing the overburden, embankment fill or borrow materials. Bedrock is described in an inspection report as Silurian age dolomite of the Kankakee formation where the Kankakee Fm is mostly yellowish grey dolomite with occasional layers of chert [Ashton, 1998]. Bedrock outcroppings are exposed on the left valley wall and on the valley floor to the south of the right abutment of the gated spillway (Photo IV-1).

Two shallow hand auger borings were taken during a post breach site visit within the exposed foundation material of the breach area (beneath the embankment dam footprint). Visual sample classifications of the first hand auger, located 2 ft upstream of the remaining sheetpile and 20 ft north of the intact core wall, identified about 2.8 ft of silty, clayey fine sand (SM/SC) transitioning to a poorly graded medium to coarse sand (SP). Underlying this was 0.5 ft of soft, black high plasticity clay (CH) that mantled coarse sand with some black fines (SP-SM). An obstruction at a depth of 3.5ft prevented further sampling. The second hand auger, located about 60 downstream of the core wall, identified about 2ft of soft silt/clay (ML/CL) overlying 0.5ft of hard ML/CL. Beneath this was black silty sand (SM) to a depth of 3 ft. Sloughing of the hole beneath the water level prevented further advancing of the hole. The soft CH material and overlying sand identified in Hole #1 may have been deposited following the breach but it was thought that the underlying black sand existed prior to construction. The hard clay layer and underlying sand in Hole #2 would also have existed prior to construction of the dam.

1.b. Embankment Soils

The exposed embankment dam material was significantly covered with rock and debris upstream of the core wall and some pavement surfacing and aggregate downstream (Photo IV-2). A close up view of the exposed material is shown in Photo IV-3. As described previously, a sample of the embankment material was classified as sandy lean clay (CL). 100% of the tested sample passed the #4 sieve with about 70% of that passing the #200 sieve. The liquid limit was 25.9 with a plasticity index of 8.7 (see Figures IV-1a/b). At the exposed face, the embankment material was soft but was also discomposed through sloughing and weathering. The density of the intact material is unknown but is expected to be lower than current design standards (note that the standard Proctor testing wasn't developed until 1933). Photo 2-2 shows the state of the upstream embankment material during construction of the core wall. Placement and compaction of the fill adjacent to the slender core wall was likely tenuous with care taken to bring material up evenly on both sides, possibly without significant compaction, to avoid wall displacement. It is unknown how much foundation preparation or embankment compaction was performed during construction.

1.c. Filters.

The embankment cross section shown in Figure 2-3 shows the inclusion of the original roadbed that existed prior to construction of the dam. The roadbed is a continuation of the old river bridge which is seen in Photo 2-1 just downstream of the dam. Again, no information is available regarding the material used in construction of the original roadbed. Figure 2-3 also depicts the placement of rockfill downstream of the roadbed without the use of filter material between any contacts.

2. Structural Elements

a. Left Abutment. The left abutment of the embankment dam ties into the right abutment of the gated spillway including the upstream and downstream wingwalls. The spillway walls are founded on bedrock and were constructed using several counterforts (Photo IV-4) spaced relatively close together. Also seen in Photo IV-4 is a remaining part of the old bridge stonework and the portion of the fishway that was integrally constructed with the downstream wingwall. A close up of the bridge pier and fishway are shown in Photos IV-5 and IV-6. The fishway seen in the photos diverged from the downstream wingwall, through the embankment fill, in a direction towards the upstream wingwall as shown on Figure IV-2. The fishway was abandoned by plugging the inlet with concrete as seen in Photo IV-7 (it is unknown if additional abandonment measures were used such as completely filling the conduit with grout). The plan presented in Figure II-2 also indicates the presence of an old bridge abutment/wall near the old bridge pier. It is unknown whether this was left in place upon construction of the dam.

The ability to compact embankment fill materials between and around the structural counterforts and against and around the old bridge pier is difficult and would not be accepted practice today without additional defensive design features. Minimal compaction at the structure was likely, creating a potential seepage path once the core wall was overtopped. The fishway also created a penetration through the embankment and the forming on the downstream wingwall created an overhang allowing a potential void or low stress area if the embankment fill were to settle.

b. Concrete Core Wall The core wall is shown in Figure II-3 as an 18-inch thick monolithic concrete stem atop steel sheet piling and located upstream of the embankment crest. The core wall appears to have performed well in controlling seepage up to the July 2010 event. Past inspections did not report any evidence of visible seepage, sinkholes or movement of material. There are, however, reasons to suspect the core wall was a contributor to the embankment dam breach. Although the embankment performed well up to the recent event, it is very possible that prior loadings did not achieve a water surface elevation that exceeded the top of the core wall (EL 898.8ft) or have a sufficient duration to develop internal erosion. It was mentioned to the IPE team that the embankment adjacent to the spillway was used as an emergency spillway and may have been used in 1947. This would have loaded the embankment above the top of the core wall, but this could not be verified. If the reservoir conditions did exceed the top of the core wall there was no record of performance issues with the embankment.

There are two critical concerns with the core wall as designed at Delhi Dam; the elevation top of core wall and the inclusion of a relatively rigid element within the earthen embankment. The top of core wall was constructed to an elevation 6ft lower than the top of the embankment. From a seepage aspect the concrete stem of the core wall is essentially impervious and would be expected to perform well in limiting seepage through the embankment. The hot rolled sheetpile beneath the concrete stem would leak but would be expected to have permeability less than the alluvial sand materials found in the hand auger borings. Thus the core wall would be expected to provide significant head reduction thereby decreasing seepage and lowering the zero pressure line within the embankment. The fact that there was no reinforced connection at the sheetpile and concrete stem is not a concern in terms of seepage performance. There is evidence that internal erosion (piping) was a significant factor in the developing breach and that seepage concerns became evident when reservoir elevations exceeded the top of core wall. Had the core wall been constructed nearer the crest of the dam, where the elevation of the core wall would be higher, it is possible that seepage erosion may not have been an issue and the dam would have only been vulnerable to overtopping flows.

The original designers of the dam may have thought that loading events greater than the top of core wall would have a relatively short duration and that steady state seepage conditions would not have time to develop through the CL embankment material. This seems reasonable; however, inclusion of the core wall within the embankment dam creates a soil/structure interaction problem involving a wall that is relatively rigid in bending and very rigid vertically compared to the embankment and overburden foundation materials. Where the concrete stem is founded directly atop bedrock and where the sheetpile was driven to bedrock the core wall can be considered non-yielding vertically. As foundation and embankment materials settle this creates non-uniform stress states near the wall and potential voids beneath the concrete stem where the concrete overhangs the steel sheetpile. As the core wall is located upstream of the embankment centerline the lateral loads on the wall differ from one side to the other. A complete soil/structure analysis was not performed but it is expected that areas of high and low stress exist such that seepage overtopping the core wall could migrate along the wall, especially when considering the likelihood of minimal compaction against the vertically formed concrete (not battered).

3. Erosion Protection and Surfacing

a. *Riprap.* Photographs and video taken during the 2010 event show that wave action was not an issue and the riprap erosion protection placed on the upstream embankment slope minimized the potential for erosion from flow along the embankment towards the gated spillway. The rock placed on the downstream slope by the LDRA in 2009 is expected to have slowed surface erosion during overtopping but did not control under seepage. Photos IV-8 and IV-9 show discolored discharge emerging adjacent to the rock covered geotextile. The discharge appears to be emerging from the area of hand placed rock or original roadbed as shown on Figure 2-3. It is unknown whether the riprap placed for the emergency spillway, described in Section II Description of Dam and Operations was removed prior to placing fill. There is insufficient evidence to suggest this was a factor in the breach.

b. *Vegetation.* Mature trees existed on the downstream slope as seen in Photos 2-5 and IV-9. Amateur video taken during the breach event shows these trees toppling as the breach developed. Discussion with Mr David Fink described the breach occurring initially adjacent to the gated structure with a downstream to upstream progression. The description is typical to what could be described as head cutting from surface flow but could also be envisioned as sloughing due to loss of material from internal erosion. The trees hindered inspection, both prior to and during the July flood event. Previous inspection recommendations included removing the trees and keeping the slope well vegetated [Allen, 2009].

c. *Road Paving.* The road pavement across the embankment dam consisted of asphalt surface underlain by concrete. The concrete underlayment supported a greater degree of undermining than asphalt surfacing alone and would be capable of supporting a roof that might form if piping occurred at shallow depths. The County road alignment project completed in 1967 included placement of fill materials. Photos IV-10 and IV-11 show lenses of coarser fill material placed above the top of the core wall either during original construction or during the 1967 work. Inconsistent fill material may have contributed to additional seepage to the downstream side of the core wall and embankment.

B Embankment Dam Performance

The performance of the embankment dam during prior historic high water levels did not reveal problems associated with foundation or embankment seepage or problems were not severe enough to be observed and documented. The 2010 event loaded the structure to reservoir levels not previously experienced so this event represents a first occurrence for Delhi Dam. Problems with the embankment dam appear to be related to loading in excess of the top of the core wall so discussion of the performance of the embankment focuses on that initiating event and higher reservoir stages.

1. Loading event

The load on the embankment is induced by the reservoir elevation also termed stage in this report (the reservoir gage is set to NGVD29 datum so stage and reservoir elevation are the same). Figure IV-3 presents the stage hydrograph from modeling of the 2010 event. As seen on the figure loading above the top of the core wall began on 23 July around 7:50 pm and continued until the core wall eventually collapsed on 24 July after 1:00 pm. The period of reservoir levels above core wall loading is estimated to be about 17 hours. Overtopping of the embankment dam is believed to have begun around 10:00 am on 24 July and continued until erosion breached the road shortly after 12:00 am. If the embankment did not breach, the duration of loading above the core wall would have been about 38 hours and the period of overtopping the crest would have been about 13 hours.

2. Internal erosion (Piping)

Interviews of several individuals describe events that suggest internal erosion was a significant factor in the dam breach. Two whirlpools in the reservoir were observed at locations that closely align with the location of the core wall. What appears to be a third developing sinkhole remains on the upstream face of the embankment. The two whirlpool/vortex locations were estimated by Mr David Fink and are shown along with the developing sinkhole location on Photo IV-12. Photo IV-13 shows the location of the developing sinkhole in line with the core wall remnant at the gated spillway. Photo IV-14 shows a close up of the developing sinkhole with a deeper part of the hole extending towards the breach. A void in the sinkhole was not found but material dug from the bottom of hole is visually classified as an SP-SM, 85% sand, 5% fine gravel and 10% silt, loose with no plasticity and scattered root fragments (likely sloughing surface material). The onsite inspection indicated that the sinkhole was in alignment with the core wall. It did not appear to be related to a toppling tree or surface erosion and was too high up slope to be considered the site of a boat anchor device. Sloughing of grass into the hole and lack of material around the perimeter of the hole indicated water flow and subsequent erosion progressing downward, apparently in the direction of the breach.

Whirlpools in the reservoir were noted by several observers with the first evidence seen around 3:30am on 24 July by Mr David Fink when he stopped to investigate a sag in the chain link fence at a distance of 40 to 50ft from the gated spillway. At 6:00 am clear seepage was observed emerging through the 2009 LDRA placed riprap on the downstream slope and around 9:00am dirty discharge was first observed. Information from interviews included in the timeline discribe areas where the road settled prior to the breach, indicative of loss of embankment material. Testimony by Mr. David Fink and Mr. Wruck identify dirty water discharging at what is referred to as the short section of embankment (that part of the dam not widened by the County construction in 1967). This location was noted by Mr. David Fink on Photo IV-12. This agrees with Photos IV-8 and IV-9 that show discolored discharge in this area

3. Overtopping

The embankment dam was overtopped about 10:00am and a complete breach through the embankment occurred about 1:00PM July 24. The progression to breach was rapid once overtopping initiated. Photo IV-9 shows overtopping flow over the rock placed by the LDRA in an opening between the trees but also shows surface flow emerging from the trees in the short section of the embankment. Overtopping flows would exacerbate the conditions already started by internal erosion.

Eyewitness accounts indicated overtopping started at the gated structure at about 10:00am on 24 July with breaching occurring within about 3 hours. Photos IV-15 and IV-16 were obtained from CBS video and show the breach at a short distance from the gated spillway. It appears from Photo IV-15 that two erosion areas developed together, possibly from seepage through the road base or other coarse layers near the embankment surface and eventually widened together. The depth of scour in Photo IV-15 appears to be fairly significant in this area. The breach area widened to the gated spillway and towards the south with increasing discharge and velocity through the breach section.

Photo IV-16 shows a fully developed breach where the reservoir drawdown appears to be controlled by the top of concrete core wall. In Photo IV-17 the drawdown has extended farther upstream as it appears the core wall has failed near the structure.

C. Conclusions

Eyewitness accounts, with description of whirlpool conditions in the reservoir near the core wall alignment, indicates that internal erosion initiated prior to overtopping. Additionally, Photos IV-8 and IV-9 show discolored discharge emerging near the embankment toe (believed to be seepage related rather than related to the surface flow). The seepage from the reservoir to the tailwater may have taken several paths but the description of an orange colored discharge suggests that embankment material was entrained within the flow. The sag in the fence and whirlpool seen observed at 3:30 am on 24 July indicates that a significant amount of flow was entering the embankment. Although a corresponding amount of flow was not observed exiting the embankment, the darkness and the vegetation on downstream slope of that section of the embankment could conceal a discharge point. At 6:00 am July 24, clear flow was seen discharging above the toe of the dam and also seen discharging in the riprap above the berm/staging area shown in Figure II-4. At around 9:00 am, about 5½ hours after the first whirlpool was discovered, dirty or discolored discharge was seen emerging near the embankment toe.

The time of hydraulic loading is judged to be too short in duration for steady state seepage to develop suggesting that seepage was following a weakness within the embankment. There were several areas where seepage could be tracking, including:

- Along the core wall where compaction or stress conditions may be low;
- Along the fish way penetrating the embankment and then along the portion where it overhangs the downstream wall of the gated spillway;
- Along/through remaining emergency spillway riprap that may not have been removed when the road was re-aligned;
- Along a contact of fill/bedrock/foundation material and then discharging through the original road fill and hand placed rock;
- Through coarser material in the upper part of the embankment and then on the embankment surface or into “emergency spillway” riprap.

With the overtopping event beginning about 10:00 am, the combinations of overtopping and internal erosion led to a full breach. As flow velocity increased with the expanding breach (see Photos IV-15 through IV-20) sufficient embankment material was removed, leading to full collapse of the core wall starting first near the gated spillway where the wall was founded directly on bedrock and then southward. Photo IV-19 indicates the core wall remained in place where erosion hadn't down cut sufficiently to lose support. The down-cutting action of the flow through the breach suggests that either the piping was occurring at a shallower depth (upper coarse material, fishway or emergency spillway location) creating a preferential surface erosion path or deeper where overflow combined with piping at the exit point. The existence of the two whirlpools/vortices, the alignments of narrow erosion gullies that appeared as deterioration progressed and the path of these two gullies toward the two whirlpool locations show active piping was occurring. Overtopping likely caused accelerated erosion along the piping channels

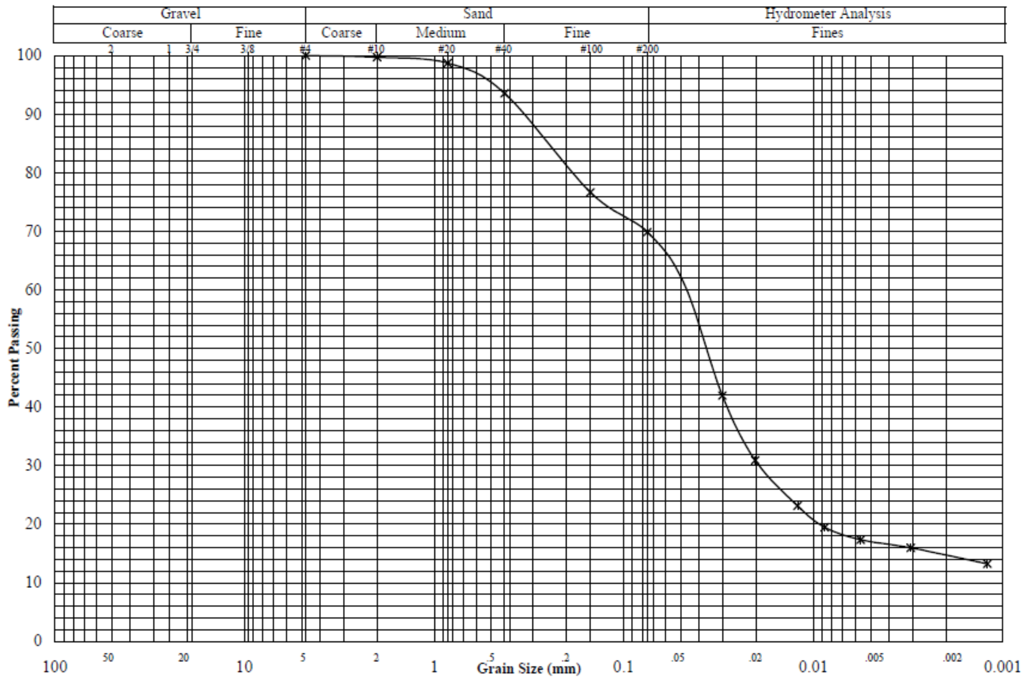
It is believed that the embankment breach was a combination of overtopping and piping. The downstream slope was not completely armored and the embankment materials are considered low plasticity and erodible. Piping discharge has been described as dirty and occurring prior to overtopping. The quantity of discharge shown in photos is significant and had the embankment not overtopped, the duration of loading would likely have been sufficiently long that internal erosion would have increased to a point where lateral support of the core wall would be reduced to a point of collapse.

Figures

Grain Size Distribution ASTM D422

Project:	Lake Delhi	Job No. :	**7638**
		Test Date:	9/14/10
Reported To:	USACE -Geotech. & Geology Section	Report Date:	9/16/10

	Location / Boring No.	Sample No.	Depth (ft)	Sample Type	Soil Classification
✶		1		Bag	Sandy Lean Clay (CL)
●					
◇					



Other Tests

	✶	●	◇
Liquid Limit	25.9		
Plastic Limit	17.2		
Plasticity Index	8.7		
Water Content	18.3		
Dry Density (pcf)			
Specific Gravity	2.69*		
Porosity			
Organic Content			
pH			
Shrinkage Limit			
Penetrometer			
Qu (psf)			

(* = assumed)

Percent Passing

	✶	●	◇
Mass (g)	292.1		
2"			
1.5"			
1"			
3/4"			
3/8"			
#4	100.0		
#10	99.8		
#20	98.7		
#40	93.6		
#100	76.7		
#200	69.8		

	✶	●	◇
D_{60}			
D_{30}			
D_{10}			
C_U			
C_C			

Remarks:

9301 Bryant Ave. South, Suite 107 — SOIL ENGINEERING TESTING, INC. — Bloomington, Minnesota 55420-3436

Figure IV-1a.
Embankment material laboratory test results.

Grain Size Distribution ASTM D422

Job No. : **7638**

Project: Lake Delhi

Test Date: 9/14/10

Reported To: USACE -Geotech. & Geology Section

Report Date: 9/16/10

	Location / Boring No.	Sample No.	Depth (ft)	Sample Type	Soil Classification
Spec 1		1		Bag	Sandy Lean Clay (CL)
Spec 2					
Spec 3					

Sieve Data

Specimen 1		Specimen 2		Specimen 3	
Sieve	% Passing	Sieve	% Passing	Sieve	% Passing
2"		2"		2"	
1.5"		1.5"		1.5"	
1"		1"		1"	
3/4"		3/4"		3/4"	
3/8"		3/8"		3/8"	
#4	100.0	#4		#4	
#10	99.8	#10		#10	
#20	98.7	#20		#20	
#40	93.6	#40		#40	
#100	76.7	#100		#100	
#200	69.8	#200		#200	

Hydrometer Data

Specimen 1		Specimen 2		Specimen 3	
Diameter (mm)	% Passing	Diameter	% Passing	Diameter	% Passing
0.030	42.0				
0.020	30.9				
0.012	23.1				
0.009	19.5				
0.006	17.4				
0.003	15.5				
0.001	13.5				

Remarks

Specimen 1	Specimen 2	Specimen 3

9301 Bryant Ave. South, Suite 107

SOIL ENGINEERING TESTING, INC.

Bloomington, Minnesota 55420-3436

Figure IV-1b
Embankment material laboratory test results.

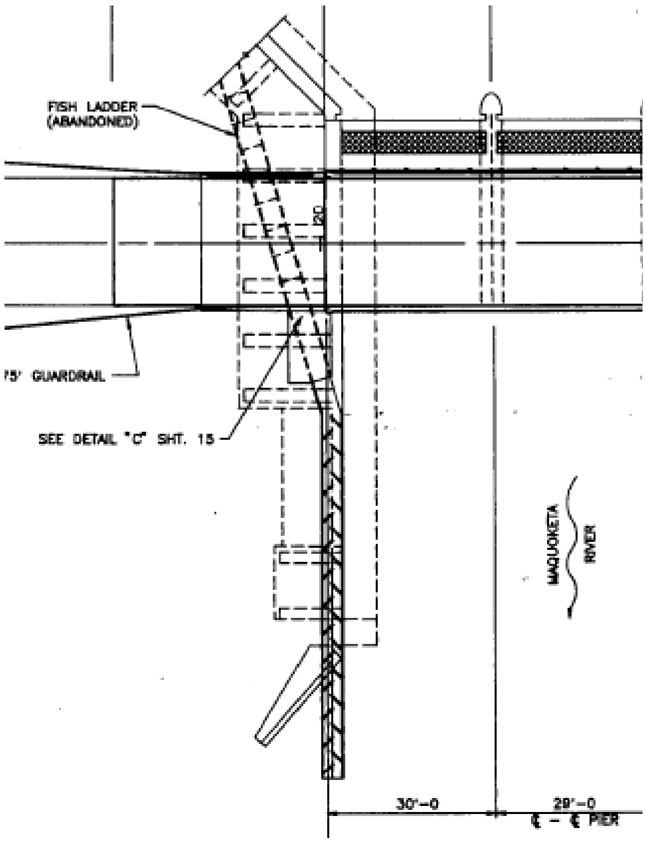


Figure IV-2.
Abandoned fishway location.

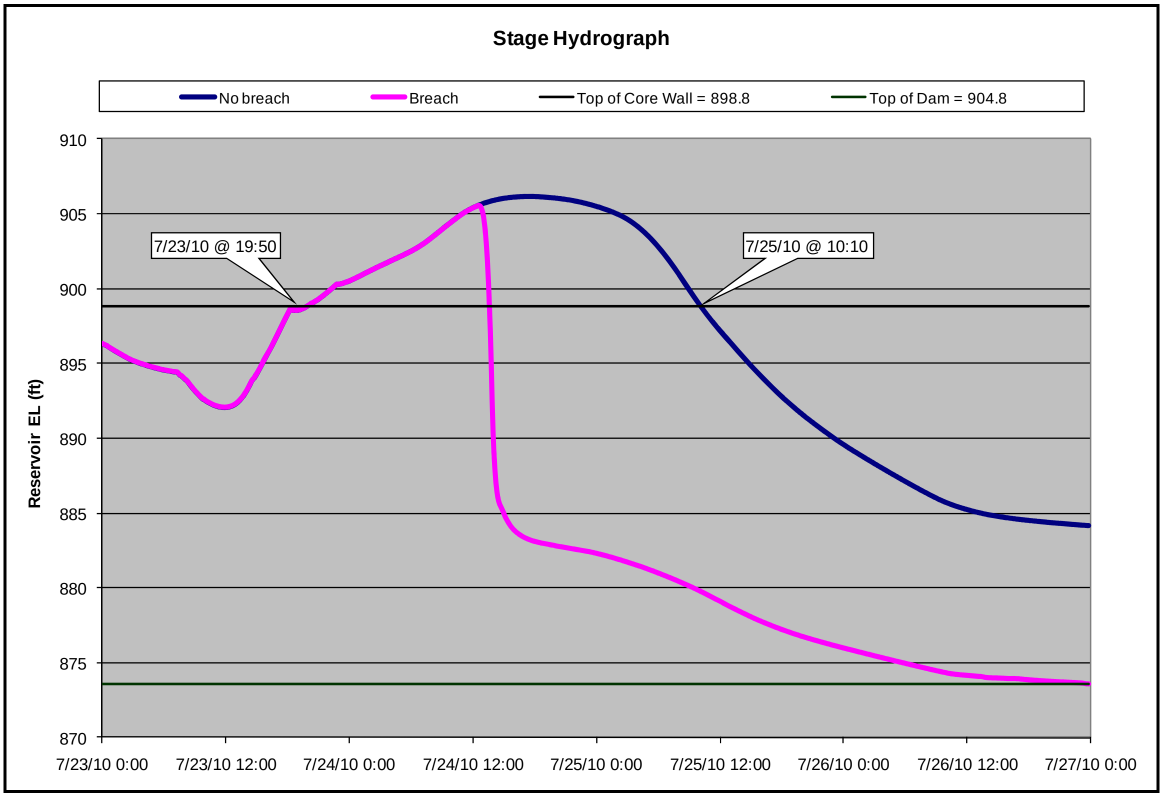


Figure IV-3
Estimated time of reservoir higher than the Top of core wall
(shown at EL 898.8ft NGVD)

Photographs

Photo IV-1.
Bedrock outcroppings on left valley wall and in current channel.

Photo IV-2.
Exposed embankment material post-breach.

Photo IV-3
Close up of exposed embankment material.

Photo IV-4
South wall of the gated spillway.

Photo IV-5
Close up of the old bridge pier and fishway remnant.

Photo IV-6
Close up of remaining fishway.

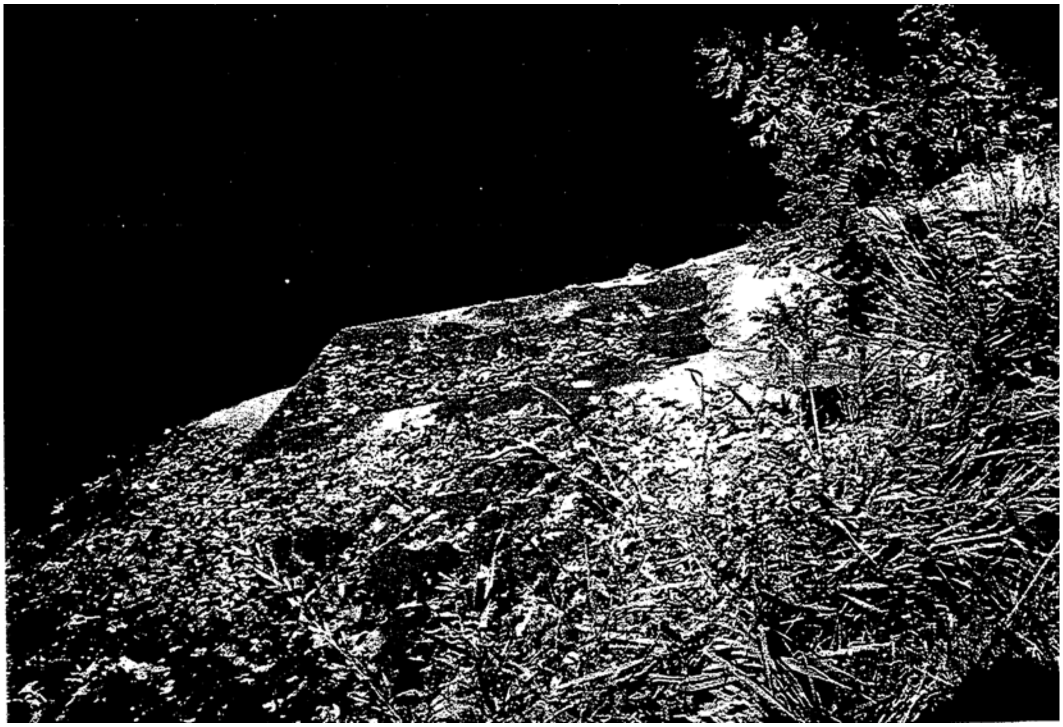


Photo IV-7
Concrete sealed fishway inlet [Ashton, 1998].

Photo IV-8
Overtopping flow and discolored discharge from beneath geotextile (still photo captured from amateur video).

Photo IV-9
Discolored discharge emerging near toe (DesMoinesregister.com).

Photo IV-10
Fill material downstream of core wall at breach. Note coarser fill material above top of core wall.

Photo IV-11
Coarse material observed above the top of core wall at the remaining sinkhole

Photo IV-12
Approximate whirlpool/vortex locations by Mr David Fink. Approximate developing sinkhole location from post breach inspection shown by red star.

Photo IV-13
Developing sinkhole location in line with remnant core wall.

Photo IV-14
Developing sinkhole with deeper area between notepad and shovel. Note surface vegetation above this area was in the process of sloughing into this hole. No void was found at the bottom of the hole but a larger rock limited the depth of excavation.

Photo IV-15
Developing breach along two fronts near gated spillway (photo from CBS news video).

Photo IV-16
Complete breach through embankment with drawdown water surface in reservoir that appears to be controlled by the concrete core wall (photo from CBS news video).

Photo IV-17
Photo taken following the complete breach through the roadway. Breach does not appear to be full depth (photo by T. McCarthy).

Photo IV-18
Continued breach development (photo by T. McCarthy).

Photo IV-19
Continued breach development showing expected core wall location
Note advancing erosion in original embankment section
(photo by T. McCarthy).

Photo IV-20. Breach expanding in width with loss of core wall (photo from DesMoinesRegister.com).

V
Hydrology and Hydraulic Considerations

A. July 2010 Inflows

The drainage basin for Delhi Dam has an area of 347 square miles [FERC, 2002]. The inflows into Delhi Dam have been derived from a number of sources. The base flow information is inflows measured at the USGS Gaging Station (No. 05416900) at Manchester, Iowa. This information is judged to be reliable and accurate and resulted in a peak flood flow of about 25,000 ft^3/s that occurred at 12:15 pm on July 24, 2010. Inflows from that location were modeled in a HEC-RAS model (created by the Iowa Department of Natural Resources) of the Maquoketa River that extended from the Manchester gaging station to the town of Hopkinton, Iowa, located about 9 miles downstream of Delhi Dam. Figure V-1 provides a layout of the HEC-RAS model. The HEC-RAS model accounted for bridges along the Maquoketa River and within Lake Delhi, which constricted the river channel and controlled the flow to a degree (see Photo V-1). Intervening flows between the Manchester gaging station and Delhi Dam were not captured at a gaging station, but were added to the model at a location about 1/2 mile upstream of Delhi Dam.

1. Previous Flood Studies.

The most current Probable Maximum Flood (PMF) for Delhi Dam was developed in 1997, and has a peak inflow of 133,600 ft^3/s [Ashton, 1998]. A flood routing of this flood results in a maximum reservoir water surface of elevation 919, which is 14.2 ft above the crest of the dam (elevation 904.8) [Ashton, 1998].

2. Estimated Inflows on July 22-24, 2010.

The inflows modeled in the flood routings through Delhi Dam consisted of two components – the flows measured at the Manchester gaging station and the intervening flows downstream of the gaging station that were estimated.

The flows at the USGS Gaging Station (No. 05416900) at Manchester, Iowa are represented by the hydrograph shown on Figure V-2. These flows are judged to be accurate. Flows downstream of the gaging station are somewhat uncertain. They are not based on stream gage data and 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 an estimated hydrograph to approximate the intervening flows. This hydrograph has a peak of 4000 ft^3/s and is represented in the hydrograph shown in Figure V-3. This is presented as the best estimate for the intervening flows but it is also estimated that the peak inflow could vary by up 30 percent (resulting in an intervening peak inflow that ranges from 2800 to 5200 ft^3/s).

3. Possible Variability in July 22-24 Inflows

The total flow that was routed through Delhi Dam was the combination of the flow measured at the USGS Manchester gage that was routed through the Maquoketa river channel and through the upper reaches of Lake Delhi (incorporating the effect of bridges that crossed the river channel and reservoir) and the intervening flow that was estimated for the drainage area downstream of the USGS Manchester gage. There is uncertainty in the flows at Delhi Dam but it is judged that estimated flows are reasonably close to the inflows experienced from July 22-24, 2010.

B. Spillway Hydraulics

Spillway discharges through the gated control structure at Delhi Dam were modeled in the HEC-RAS program. The flow through the spillway can occur under two conditions – free flow where the gates are out of the water and do not control the releases through the spillway and orifice flow, where the bottom of the gate is below the reservoir water surface and the gate opening controls spillway releases. For the Delhi Dam spillway during the July 22-24 flood, the primary condition was orifice flow. The spillway discharges for a given gate opening were calculated within the HEC-RAS program. As a check, discharges were calculated using information provided in Design of Small Dams [Reclamation, 1987]. Figure V-4 provides the results of these calculations and Figure V-5 provides the curve that was used to determine the discharge coefficient for orifice flow. The discharges calculated by the above method are generally close to what was calculated in the HEC-RAS program.

C. Flood Routing Results

Flood routings were performed using the HEC-RAS model. More detailed results of the flood routings that are summarized in this section of the report are included in Appendix G. Inflows into the reservoir were generated as described above. The volume of the reservoir was accounted for through the cross-sections along the reservoir input into the HEC-RAS model. The cross sections were obtained from a LIDAR survey model. Cross sections geometry below the normal reservoir surface that could not be detected by LIDAR was estimated based on past lake soundings and bank slopes. Spillway discharges were defined in the model and were a function of the gate opening for each spillway gate, the reservoir level and a discharge coefficient. Discharges were calculated using the following equation:

$Q = CWB(2gH)^{1/2}$, where

$C = 0.6$ (orifice discharge coefficient)
$W = 25$ ft (gate width)
B = vertical gate opening, ft
$g = 32.16$ ft/sec^2,acceleration of gravity
H = head from reservoir water surface to spillway crest (El. 879.8)

A number of routings were performed. Initial flood routings were performed with the intent of matching the reservoir operations levels as closely as possible with adjustments made to achieve the same water levels at the dam and at upstream and downstream locations in the HEC-RAS model as those that actually occurred during the flood. After these initial runs were made, sensitivity studies were made to evaluate the effect of different reservoir operations on reservoir levels and durations at Delhi Dam. A specific area of interest was evaluating the effect of the third gate operating to its full opening. Key questions were whether the dam would have overtopped with all three gates operating as intended and also the depth and duration of reservoir levels above the top of the core wall if all three gates had been fully functional.

Additional sensitivity flood routings were performed to evaluate the effect of varying some of the key input parameters in the flood routings.

1. July 22-24, 2010 Flood Event Modeling

Flood routings were performed for the conditions experienced during the July 22-24 flood event. The initial assumption on gate opening is that the gates would be opened at a rate of 0.5 ft/min, and the gate opening was triggered at reservoir water surface (RWS) elevation 896.5 for Gate 1, RWS El. 897 for Gate 2 and RWS El. 898 for Gate 3. The maximum gate openings established for the gates were 18 ft for Gate 1, 17.1 ft for Gate 2 and 4.25 ft for Gate 3. The restricted opening assumed for Gate 3 reflected the condition that was experienced during the flood (there was some discrepancy on this – the dam operators indicated that the gate was opened to a maximum opening of 6 ft during the flood; during the site exam on 9/8/10, the IPE measured the gate opening at 4.25 ft and the IPE was told that the gate positions were not changed after the flood). Based on the assumptions described above and the inflows that were initially assumed, the initial HEC-RAS runs matched very well with the water levels recorded at locations upstream of, downstream of and at Delhi Dam when the same time periods were evaluated.

One of the other assumptions in the model was that the dam would breach if overtopping flows were sufficient. Parameters were established in the HEC-RAS model that controlled the rate of the breach once overtopping flows initiated. The breach outflows and downstream water levels during the flood matched well with the actual levels experienced by downstream communities. If the dam had not failed during the flood event, overtopping depths and reservoir levels above the top of the core wall would have been experienced for a longer period of time. Figure V-6 shows the flood routing results for the case where the dam was not allowed to breach compared to the breach scenario.

2. *Sensitivity Flood Routings*

The assumptions made in the baseline routing were varied in a subsequent phase of flood routings. These sensitivity studies were an attempt to determine if a better calibration of the model to the actual flood conditions could be achieved and to evaluate the effect that different spillway operations would have had during the flood event.

a. Variable Spillway Gate Operations The baseline routing limited the spillway gate openings to those measured by the IPE during the site examination on 9/6/2010 (Gate 1 = 18 feet, Gate 2 = 17.1 feet and Gate 3 = 4.25 feet). The dam operator's log of gate openings is shown below and indicates different maximum gate openings. An additional routing was performed which used the maximum openings reflected in Table V-1.

Table V-1 - Dam Operator's Log of Gate Openings

Date	Time	Gate # 1 Opening	Gate # 2 Opening	Gate # 3 Opening
7/22	8:30 am	Closed	0.67ft	Closed
7/22	9:15 pm	Closed	1.67 ft	Closed
7/23	12:15 am	Closed	4.00 ft	Closed
7/23	7:30 am	Closed	8.00 ft	Closed
7/23	8:00 am	Closed	9.00 ft	Closed
7/23	8:30 am	Closed	11.00 ft	Closed
7/23	10:00 am	Closed	18.00 ft	Closed
7/23	2:45 pm	4.00 ft	18.00 ft	Closed
7/23	4:00 pm	3.17 ft	18.00 ft	5.00 ft
7/23	6:30 pm	14.00 ft	18.00 ft	5.00 ft
7/23	8:30 pm	19.00 ft	18.00 ft	6.00 ft
7/23	11:00 pm	19.00 ft	18.00 ft	6.00 ft
7/24	1:00 pm	19.00 ft	18.00 ft	6.00 ft

The results of the two routings matched reasonably well as shown in Table V-2. There was also a difference in the rate of gate opening used in the HEC-RAS model and as indicated in Table V-1 (see Figure V-7). This difference was explored in additional routings, but was not significant.

Table V-2 – Comparison of Gate Opening Criteria		
Gate Opening	Maximum Reservoir Water Surface El.	Duration of Core Wall Overtopping
Determined by Measured Gate Openings on 9/8/10	906.17	38.20
Matched to Dam Operator's Logbook	905.29	35.30

b. Variable Starting Reservoir Water Surface Elevation One of the comments from downstream residents interviewed by IPE was that the Lake Delhi Recreation Association should have lowered Lake Delhi during the July 22-24 flood in order to store more of the flood and minimize the downstream impacts. Flood routings were performed in which the starting reservoir water surface elevation was lowered from El. 896.2 by 2, 4 and 10 ft. Two scenarios were evaluated, which considered different spillway gate operations with the additional reservoir storage space. The first scenario, shown on Figure V-8, assumed that the spillway gate operations according to the dam operator's logbook were maintained and for this case, the lowered elevation had no effect on the ultimate flood levels. For the second scenario, shown on Figure V-9, the gates were all fully opened at the beginning of the flood, which created additional storage space to the spillway crest elevation. This case maximized the reservoir space available as well as the spillway releases (discharges were free flow and not orifice controlled). For this case, the drawdown scenarios did not overtop the dam but came close to doing so. Neither set of routings are judged to be totally realistic and the expected operation would have likely have fallen somewhere between these scenarios. Overall, the routings indicate that the reservoir storage volume is small in comparison to the volume of the flood, which minimizes the impact of additional reservoir storage.

c. Variable Intervening Flows As mentioned earlier in this section of the report, the intervening flow that was assumed in the baseline HEC-RAS run has some uncertainty associated with it. It was judged that the peak inflows could vary by +/- 30 percent. The inflow values for the intervening hydrograph were adjusted by these amounts and the flood routings were repeated. A comparison of the results of these flood routings is shown in TableV-3.

Table V-3 – Comparison of Varying Intervening Flow		
Gate Opening	Maximum Reservoir Water Surface El.	Duration of Core Wall Overtopping
Baseline Assumption for Intervening Flow	906.17	38:20
Intervening Flow Increased by 30%	906.51	40:50
Intervening Flow Decreased by 30%	905.75	36:25

Photo V-1 – Aerial View of Breached Delhi Dam in Foreground and Upstream Bridge in Reservoir

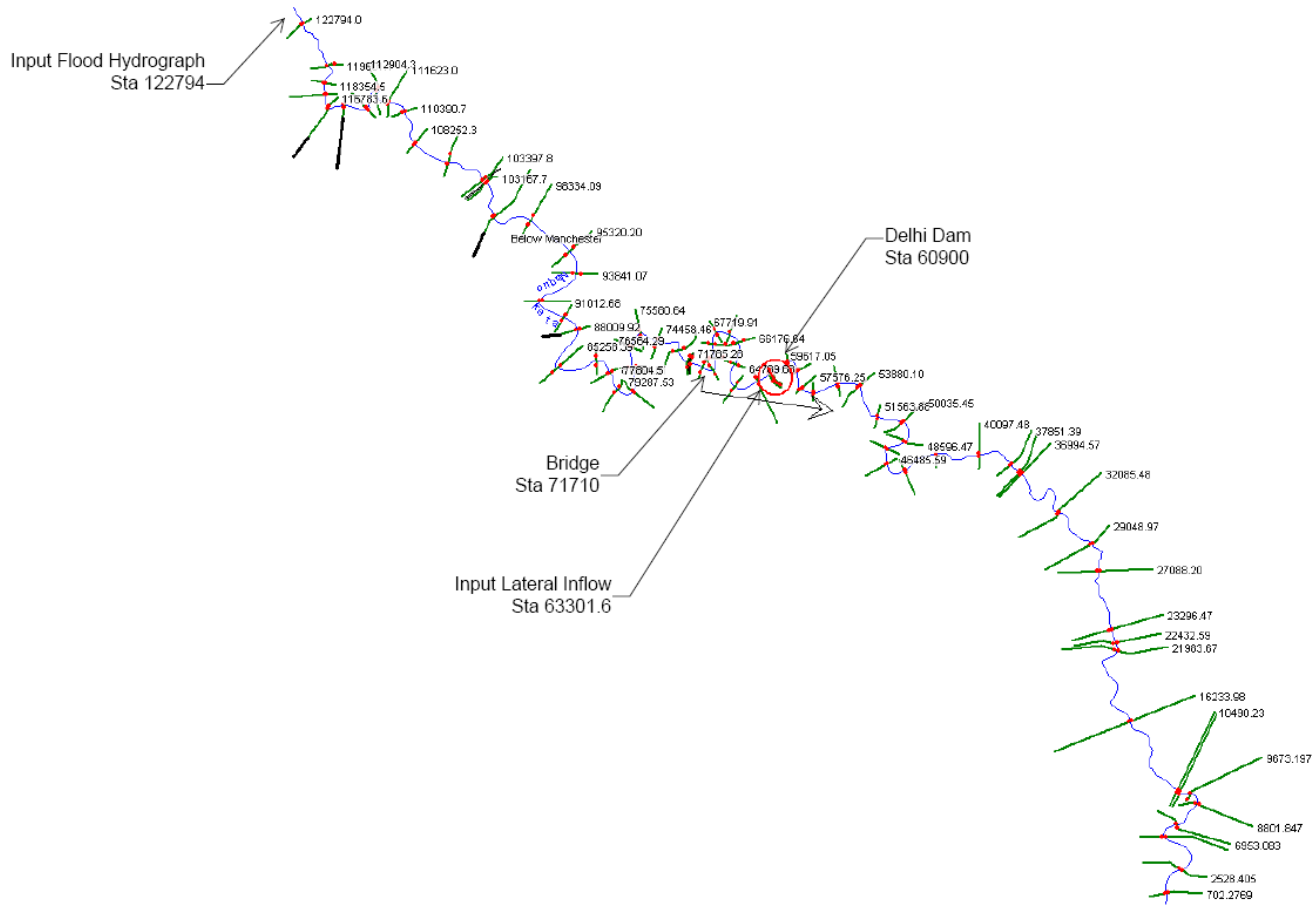


Figure V-1 - Iowa DNR HEC-RAS Model, Maquoketa River Model from US20 to the City of Hopkinton, IA

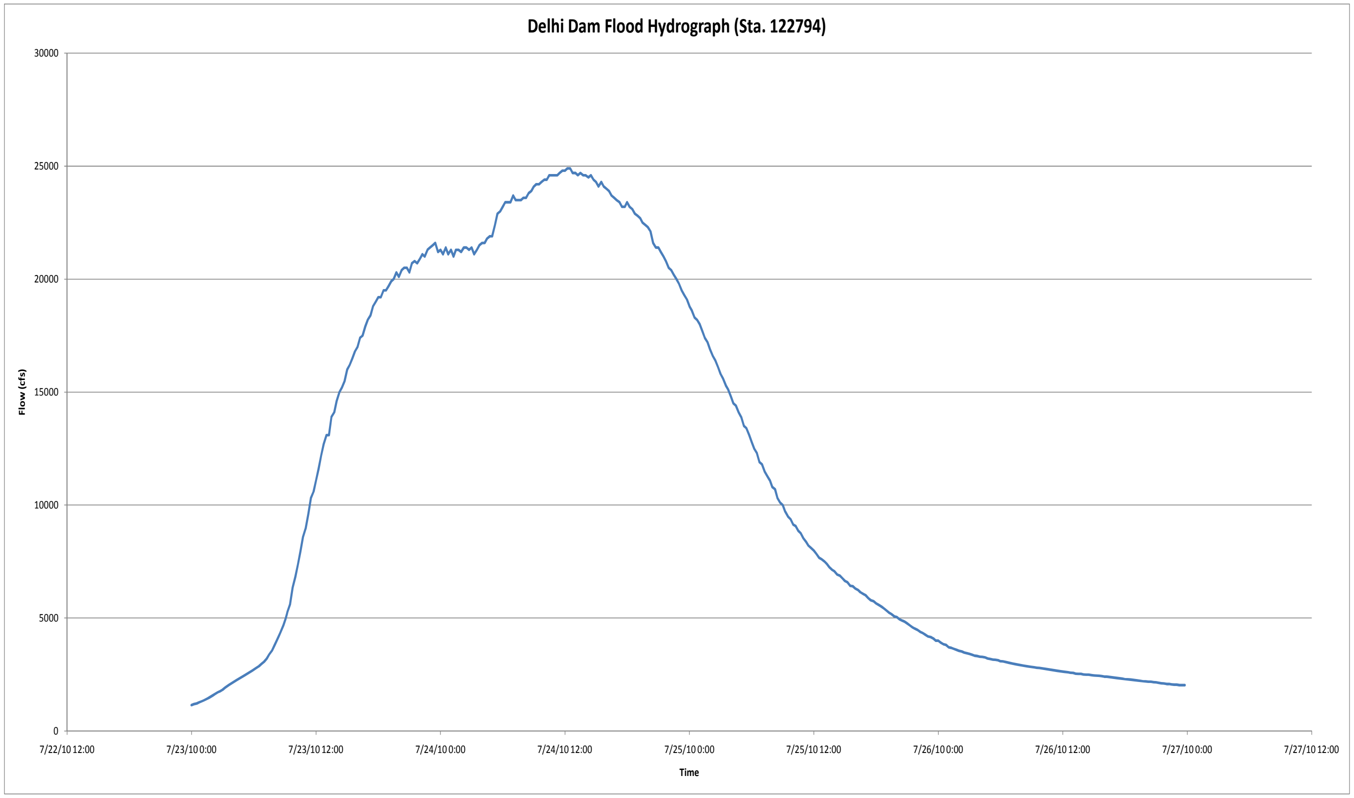


Figure V-2 - Flood Hydrograph at USGS Gaging Station 05416900, Maquoketa River at Manchester, IA

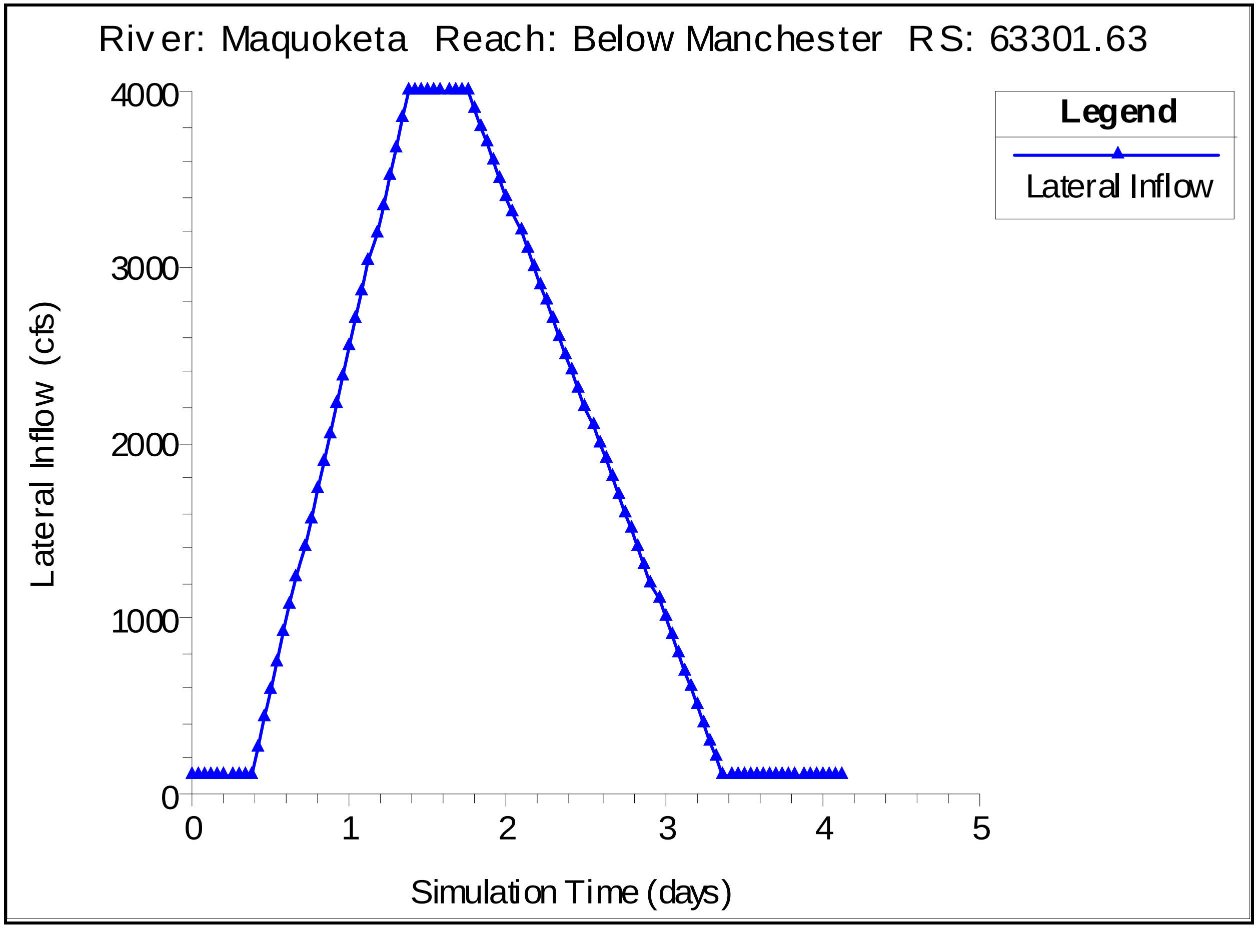


Figure V-3 - Inflow Hydrograph Representing Intervening Flow Between USGS Gage at Manchester IA and Delhi Dam

Spillway Discharges, Orifice Flow Through Spillway Gate Openings

	894			897			900			903			905.55		
d	H_1	H_2	Q (cfs)	H_1	H_2	Q (cfs)	H_1	H_2	Q (cfs)	H_1	H_2	Q (cfs)	H_1	H_2	Q (cfs)
1	14.2	13.7	519	17.2	16.7	574	20.2	19.7	623	23.2	22.7	669	25.75	25.25	705
1.5	14.2	13.45	772	17.2	16.45	854	20.2	19.45	928	23.2	22.45	997	25.75	25	1053
2	14.2	13.2	1020	17.2	16.2	1130	20.2	19.2	1230	23.2	22.2	1323	25.75	24.75	1396
4	14.2	12.2	1961	17.2	15.2	2189	20.2	18.2	2395	23.2	21.2	2585	25.75	23.75	2736
4.25	14.2	12.08	2073	17.2	15.08	2316	20.2	18.08	2536	23.2	21.08	2738	25.75	23.63	2899
6	14.2	11.2	2818	17.2	14.2	3173	20.2	17.2	3492	23.2	20.2	3785	25.75	22.75	4017
8	14.2	10.2	3586	17.2	13.2	4079	20.2	16.2	4519	23.2	19.2	4920	25.75	21.75	5236
10	14.2	9.2	4257	17.2	12.2	4902	20.2	15.2	5472	23.2	18.2	5988	25.75	20.75	6393
12	14.2	8.2	4823	17.2	11.2	5636	20.2	14.2	6347	23.2	17.2	6985	25.75	19.75	7485
14	14.2	7.2	5272	17.2	10.2	6275	20.2	13.2	7139	23.2	16.2	7909	25.75	18.75	8508
16				17.2	9.2	6811	20.2	12.2	7844	23.2	15.2	8755	25.75	17.75	9461
17.08				17.2	8.66	7054	20.2	11.66	8186	23.2	14.66	9178	25.75	17.21	9945
18							20.2	11.2	8455	23.2	14.2	9520	25.75	16.75	10339

d = gate opening in feet
H_1 = Reservoir water surface elevation – Spillway crest elevation, feet
H_2 = Reservoir water surface elevation – Centerline of spillway gate opening elevation, feet

Figure V-4 – Spillway Discharges Calculated for Orifice Flow, Using Discharge Coefficient from Design of Small Dams, Fig 9-31 [Reclamation, 1987]

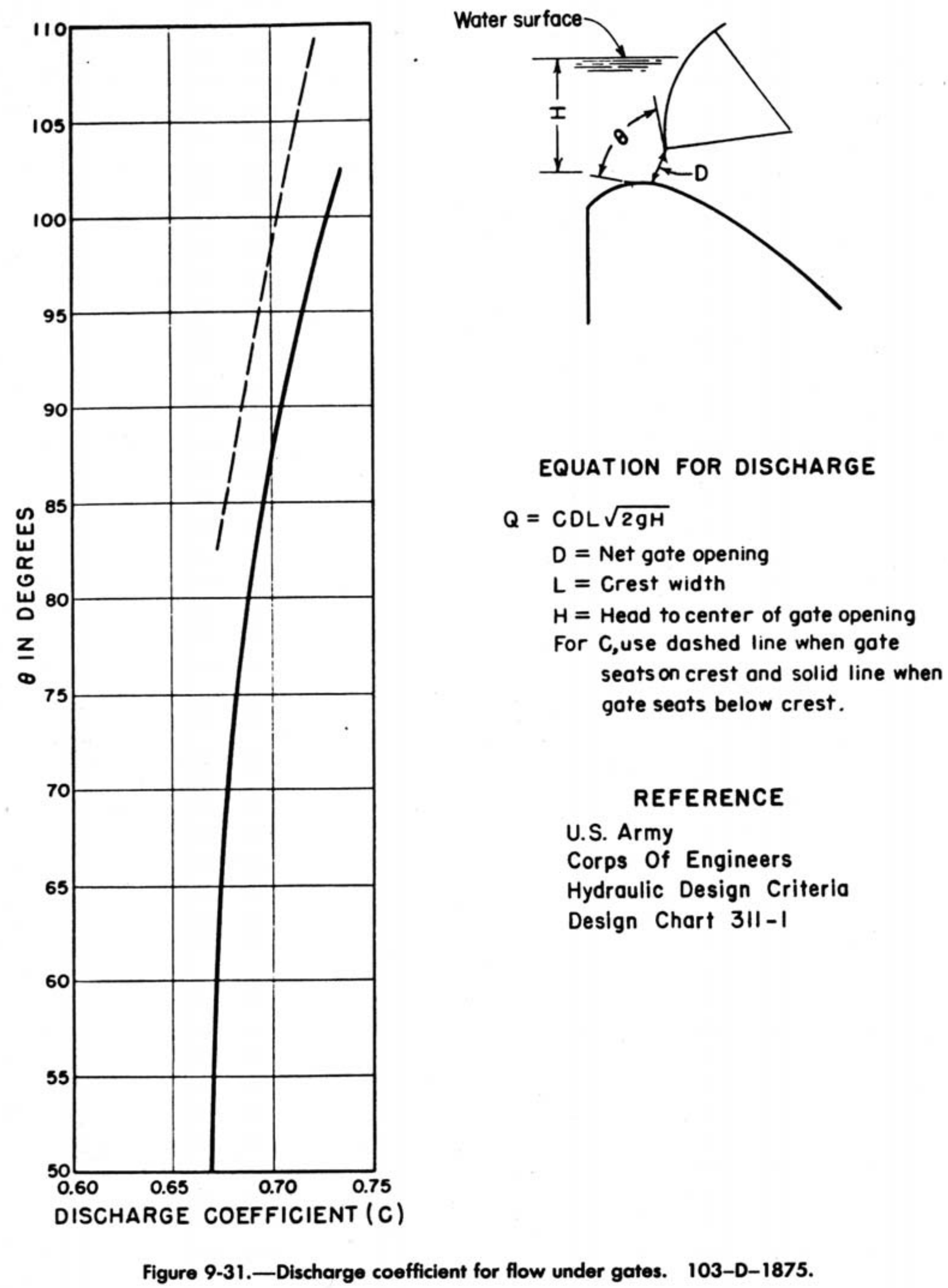
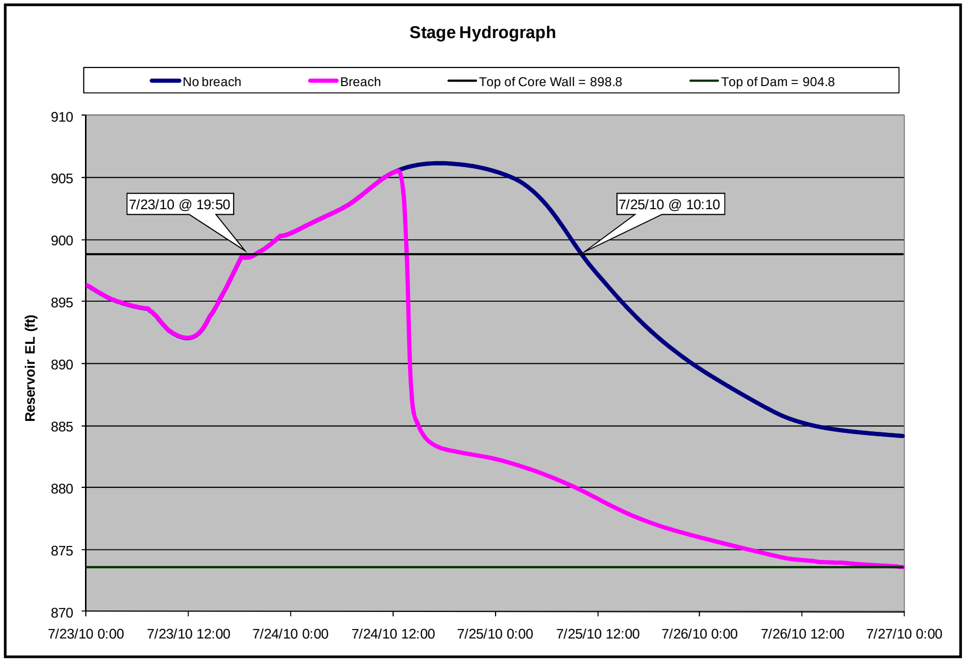


Figure 9-31.—Discharge coefficient for flow under gates. 103–D–1875.

Figure V-5 - Discharge Coefficient from Design of Small Dams, Fig 9-31
[Reclamation, 1987]

Breach Scenario Compared to the No-Breach Final Measured Gate Operations



Time Series	Maximum	Time at Max	Volume (ac-ft)
HW Stage	905.58	24Jul2010 1115	
TW Stage	886.36	24Jul2010 1235	
Flow	69292.32	24Jul2010 1235	107760.64

The breach scenario was triggered once the reservoir elevation reached 905.55 ft. Time to full breach formation was 1.5 hrs.

Figure V-6 - Comparison of Reservoir Water Levels with and without Dam Breach

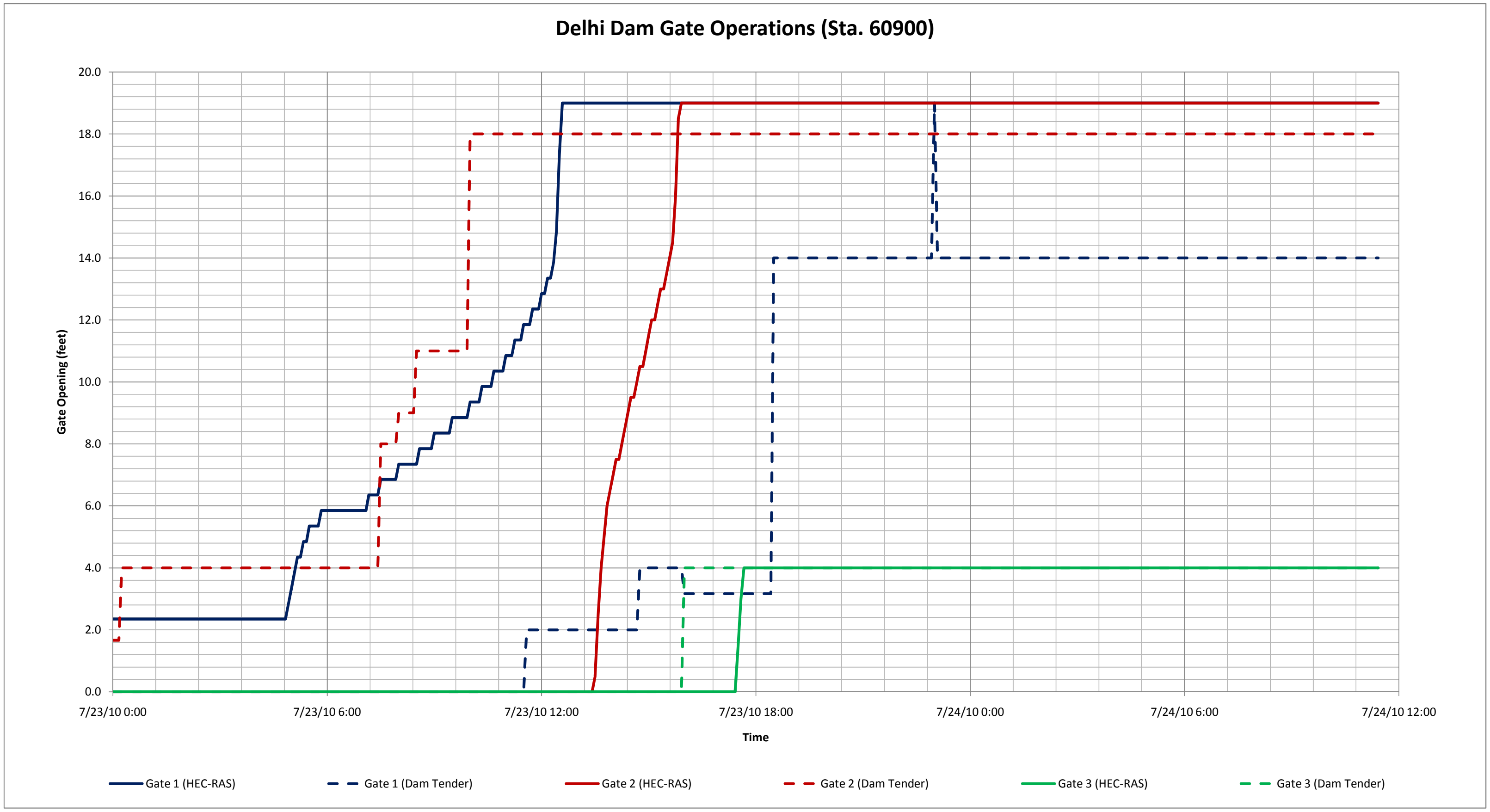


Figure V-7 – Gate Operations in HEC-RAS compared to Actual Gate Operations

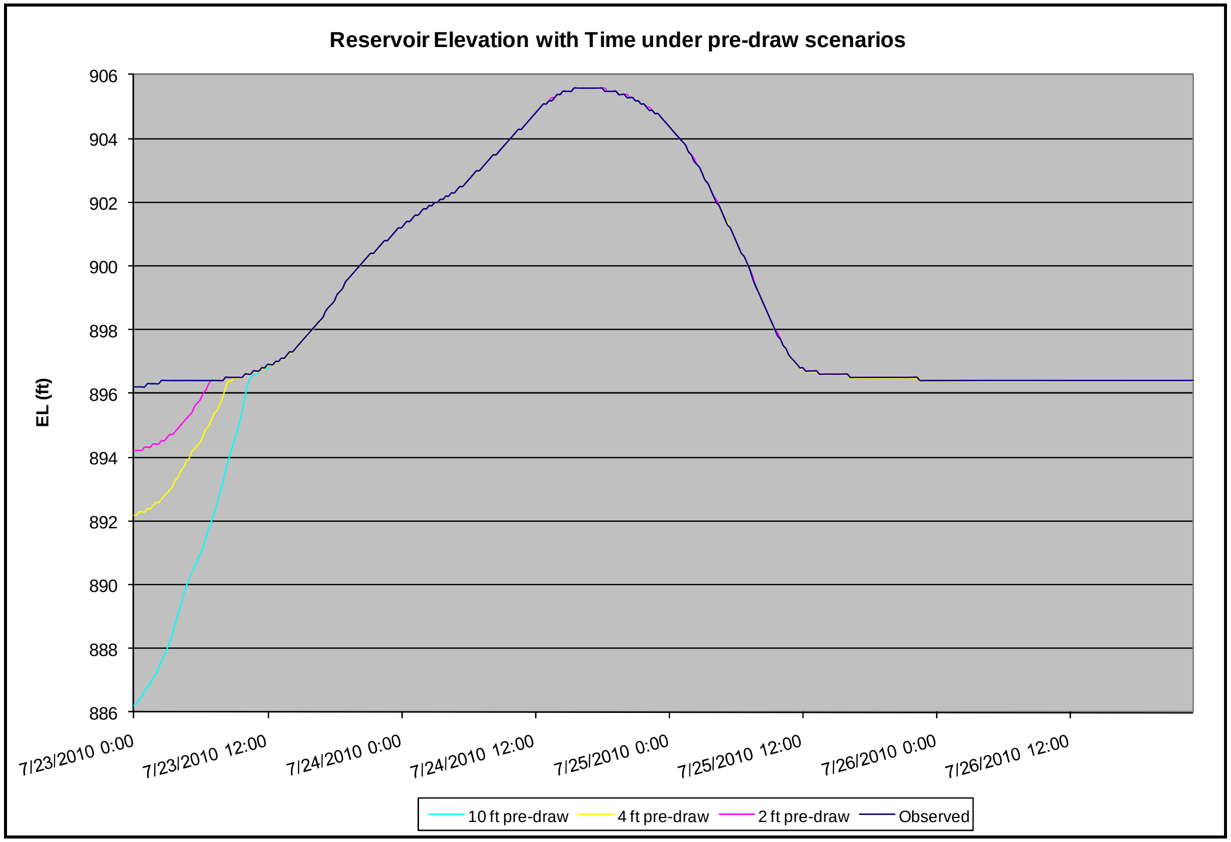


Figure V-8 - Effect on Lower Initial Reservoir Water Surface Elevation on Routing Results, Gate Opening Triggers Maintained

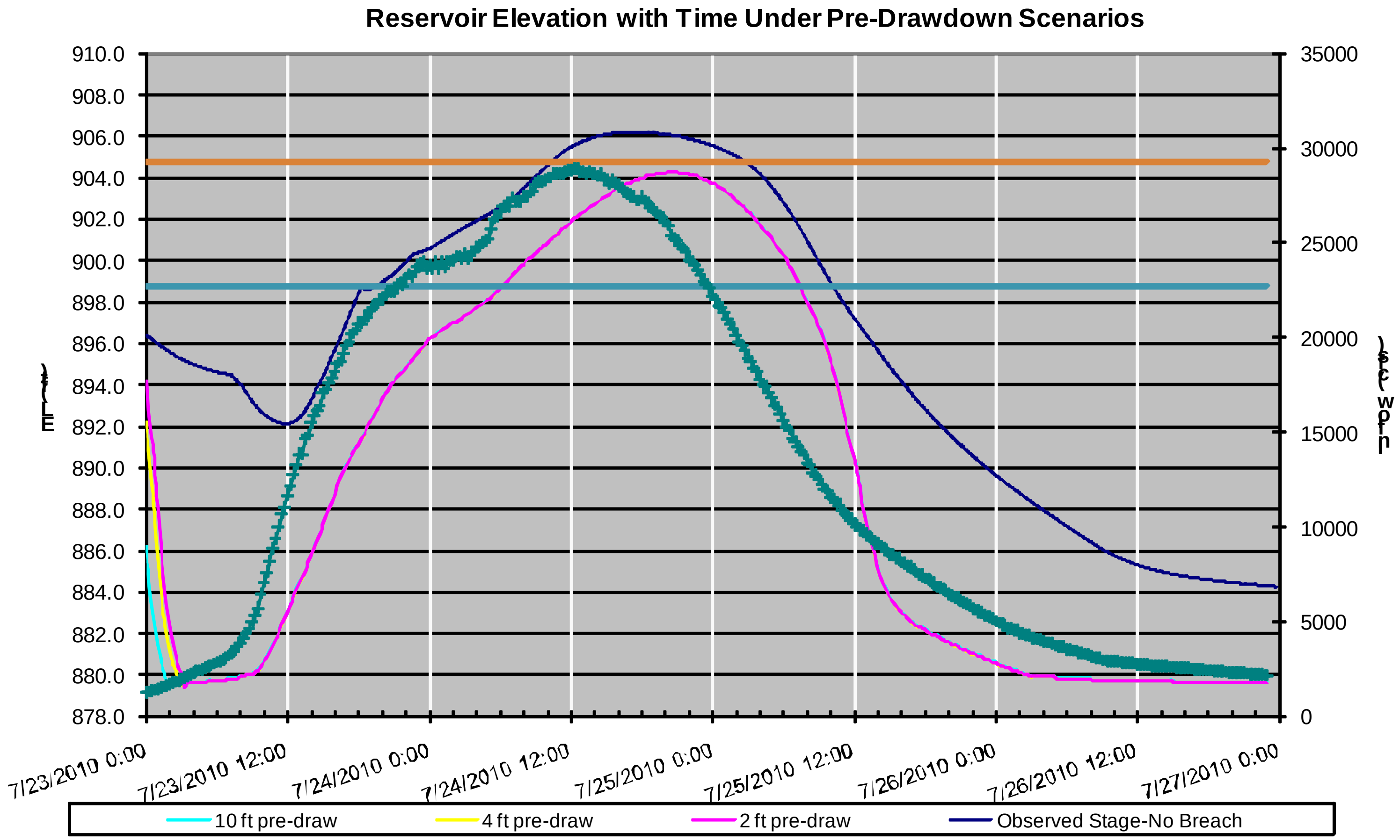


Figure V-9 - Effect on Lower Initial Reservoir Water Surface Elevation on Routing Results, Gate Wide Open

VI

Emergency Response

The breach of Delhi Dam on July 24, 2010 resulted in a catastrophic release of the reservoir. The peak breach outflow from the dam is estimated to have been about 69,000 ft^3/s. The breach outflow resulted in significant flooding damage to downstream property and property along Lake Delhi, but no loss of life occurred. The fact that no loss of human life occurred can be attributed to several factors. The concrete core wall of the dam likely slowed down the rate of the full dam breach. Without the core wall in place, the dam would have breached much faster and peak breach outflows would have likely been higher. The breach outflows were dissipated by farmland downstream of the dam and reduced the impact to communities downstream of the dam (Hopkinton and Monticello). The national Weather Service predicted crest of the breach flows was never reached. Another factor was that local officials were effective in warning and evacuating residents in Hopkinton and Monticello.

C. Emergency Action Plan for Delhi Dam

At the time of the breach of Delhi Dam, a formal written emergency action plan (EAP) was not in place. EAPs are not required for dams (including high hazard dams) by the Iowa Department of Natural Resources. An EAP had been drafted by the Lake Delhi Recreation Association, but the EAP focused mostly on residents within the reservoir area and not on downstream communities. A dam breach analysis had not been performed for Delhi Dam [Allen, 2009], prior to the dam breach. Despite this, over the years the operators of Delhi Dam and the downstream emergency management officials had developed informal protocols of notifications of changes in spillway gate openings during flood releases and those were translated into flooding levels along the downstream river channel. An informal emergency action plan appeared to be in place at Delhi Dam in the event of an emergency. The Lake Delhi Recreation Association would make calls to the dam operators, members of the Association Board, the Sheriff of Delaware County, the Manchester Fire Department Emergency Response Team, the Jones County Emergency manager, the Communities of Hopkinton (9 miles downstream) and Monticello (15 miles downstream) and the Delaware County Engineer [FERC 2002 Prelicense Inspection].

D. Warnings and Evacuation – July 24, 2010

The weather situation and potential for flooding at the dam was closely monitored and evaluated in the days leading up to the breach on July 24, 2010. Changes in spillway gate openings were conveyed to the downstream emergency management officials as were warnings issued about the dam's condition On the morning of July 24th it became apparent that the dam was in danger of failing for internal erosion. At 3:30 am, Mr David Fink sent out an initial warning of developing problems with the dam after observing the first sinkhole . At 9:00 am Mr David Fink upon finding the cloudy/dirty discharge at the toe of the embankment released the dam failure warning. to M. Ryan Delaware County Emergency manager and Ms. Brenda Leonard, Jones County Emergency Manager. The National Weather Service updated their river model downstream of the dam and predicted a stage level of elevation 805 at Monticello if the dam breached. This information was used along with GPS devices to identify homes that needed to be evacuated. Figure 1 provides a map of a portion of the town of Monticello and shows spot elevations and the homes that were evacuated. Firemen went door to door in this area and told residents to evacuate. Other downstream residents evacuated on their own, without official warning, as they became aware of the increasing flooding potential in their immediate location..

A complete assessment of the dams condition at 3:30 am July 24, 2010 was hindered by the trees and brush on the downstream slope of the embankment. No loss of life downstream was reported due to the dam breach but flooding affected numerous structures and people. Although the procedures were successful in preventing loss of life during the 2010 event, that may not be the case if a repeated situation were to occur. The identification of the whirlpool and vortex with a sagging fence in the embankment is a very serious concern and recognition of the seriousness of the situation would expectedly lead to evacuation notice of the downstream population at risk. Since this was first observed about 3:30am on July 24 either a recommendation to evacuate should have been made at that time by the dam owner or if the seriousness was not fully recognized or understood, then an emergency action response should quickly involve notification of organizations/persons with dam safety experience to help assess the situation. Dam safety training and an emergency action plan (EAP) are key to a rapid response. The Delaware and Jones County Emergency Coordinators were notified of a developing problem shortly after the whirlpool was first observed but the dam alert to the public didn't happen until about 9:00am. A detailed EAP is needed upon reconstruction of Delhi Dam.

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Photo VI-1
View of Flooding in Lake Delhi Reservoir Area, showing Bridge Upstream of Delhi Dam in Background
(Photo Taken by Tom McCarthy, Iowa DNR)

Photo VI-2
Damage and erosion immediately downstream of the dam after the breach
07/24/10

Photo by Justin Hayworth

Photo VI-3
Water levels are checked as flood waters of the Maquoketa River and Lake Delhi continue to flow over Broadway in Hopkinton on Saturday, July 24, 2010.
(Julie Koehn/SourceMedia Group News)

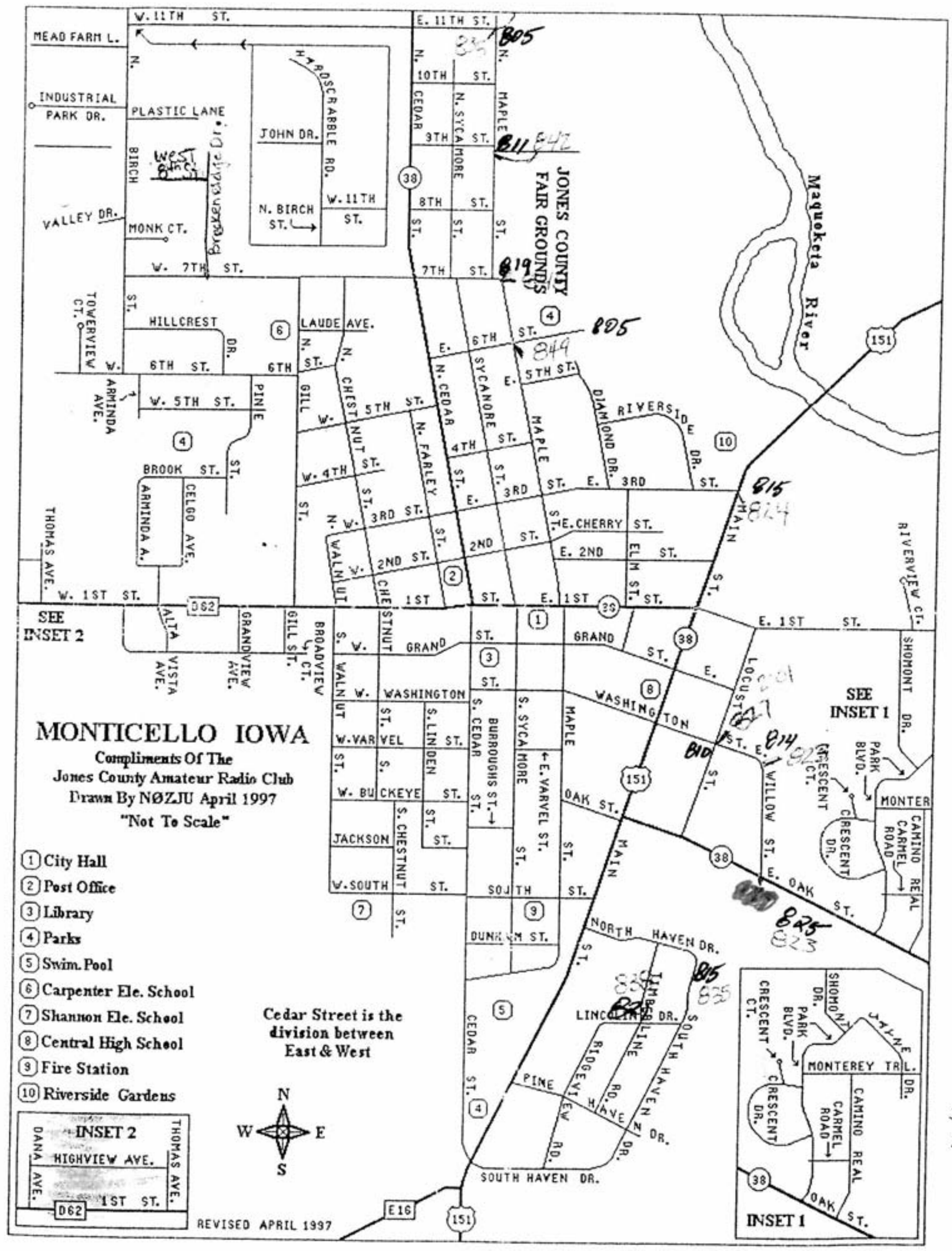

Figure VI-1
Map of Monticello, Iowa with spot elevations.

VII

Timeline of Events

The information the IPE gathered during the week of September 6, 2010 included review of records available at the state and Delhi, Iowa; interviews of state and local officials, dam operators and several local residents and a examination of photographs and videos taken of the site before, during and after the flood of July 22-24, 2010. From this a timeline of events and actions was developed in order to document the event sequence and actions of the various personnel involved in decisions and management and also assist in the computer analysis of the flood. The complete summary of interviews and composite timeline is presented herein are not verbatim nor sworn testimonies but rather a summary of the interviews and conversations between the IPE and each individual. Actual data presented may or may not be accurate since there were conflicts in the estimated details such as the eye witness descriptions of the reservoir elevations in relation to the crest of the dam and similar words used to describe different conditions such as the overtopping and progression of the breach. The presentation of the events the IPE judges as key events and actions is as follows:

Thursday
July 22, 2010

04:45am Delaware County Emergency is aware of heavy rain since midnight

Friday
July 23, 2010

00:15am hours Mr David Fink and Mike Russell discussed lowering Lake Delhi

01:00am Mr David Fink , dam operator notifies Brenda Leonard, Jones County Emergency Manager of the first gate opening at 7% (meaning 7% of the total opening of all three spillway)

07:00am Mr David Fink notifies Brenda Leonard gate opening changed to 15%

08:30am Mr David Fink notifies Brenda Leonard gate opening changed to 20%

10:00am Mr David Fink notifies Brenda Leonard gate opening changed to 33%(Gate 2 fully open)
11:40 am Mr David Fink reports reservoir at 121.0 local datum, successful in lowering the reservoir 0.8 ft.

12:45pm Mr David Fink reports reservoir at 120.7

02:30pm Mr David Fink notifies Brenda Leonard gate opening changed to 40%

03:05pm Mr David Fink notifies Brenda Leonard gate opening changed to 50%

05:30pm Mr David Fink notifies Brenda Leonard gate opening changed to 66%
(Gates 1 and 2 now fully open)
Resevoir at 122.0ft (Local datum)

08:00pm Mike Ryan receives new notice from the Weather Service 11-14 inches predicted over next 48 hours

08:30pm Mr David Fink reports reservoir at 122.17ft. Also Gate 3 wedged at 6ft opening.

10:30 pm   Mike Ryan reports heavy rain in Manchester, Mr David Fink reports 3-3.25 inches of rain at Delhi

11:00pm Mr David Fink reports the reservoir at 122.9 ft local datum

Saturday
July 24, 2010

01:00am Mike Ryan orders sandbags to Delhi

02:15am Weather Service notifies Mike Ryan of forecast for 3 more hours of heavy rain

03:30am Mr David Fink reports inspection of Delhi Dam, reports sag in upstream crest chain link fence with a whirlpool 4ft in diameter and a 5-6 inch vortex located 40-50ft south of the spillway section. No water found on the downstream side as best as he could see.
Reservoir estimated around 124-125ft (local Datum)

03:40am Mr David Fink reports to Mike Ryan of 3ft of freeboard at the dam

03:45am Mr David Fink reports to B. Leonard that dam issues were developing and could be big. Mark Stoneking, Monticello Fire Chief notified

04:00am T. Gorman, A resident on the river less than a mile downstream of the dam, call by neighbor to immediately evacuate

04:30am B. Leonard Jones County Emergency Manager got Mark Stoneking and drove to the dam arrived at 05:00am

05:30am Mr David Fink asked for riprap, Anthony Bardgett County Engineer called for rock and loader; B Leonard noted hole above the waterline. 6ft long, 3'wide at u/s slope at the upstream shoulder in line with the steep section of the embankment, fairly close to dam structure, Soil falling into hole above water line, water appeared to be coming up from bottom and all sand pieces were sliding in, bottom of hole was filled in with water, sandy material; Didn't get a look at downstream face of dam; No evidence of bump or dip or hole in road

05:45am Mike Ryan received call from Mr David Fink describing the hole in the north embankment

06:00am 6:00-6:10 Mr David Fink issues potential dam failure alert public to Mike Ryan. Road settlement but not in line with whirlpool, water discharging from riprap, 40-50 ft south of wall on downstream side above toe near some trees, flow is clear exiting 5-10ft (120-125ft local datum) down from crest, reservoir 125-126 ft. Water is also running in new riprap above berm area. No water over dam. Anthony Bardgett arrived at dam site on south side; Noticed dipping of road first in the northbound lane(Downstream side), then in the southbound lane(Upstream side), further south in roadway from whirlpool; Northbound lane road dipped (first), initially 6-8" dip, nearer to white work tent, southbound side/lane later (second); Did not want to send equipment across road for safety.
T. Gorman is at the dam at daylight, sinkhole in U/S slope- about 100ft from the concrete spillway, 15ft up from the lake 6-8 ft from the road, 6-8 ft diameter.
Road dip in area left(north) of the whirlpool

06:10am Call from Anthony Bardgett to M Ryan Riprap and loader delivery confirmed, would have equipment in place in 45 minutes. 2ft by 4ft hole with caving spotted above the water line; – Call from Mr David Fink: tells Mike Ryan to contact/alert public about dam failure; firemen asked to go door to door; call to NWS to ask to put in dam watch; calls Public Information Office(PIO) to alert media; ; riprap is coming, dangerous situation. . Delhi and Hopkinton Contacted

06:15am Per County Engineer Anthony Bardgett, a whirlpool is located in-between embankment and lake association work barge. Dip in pavement inline with whirlpool and work tent, surface drainage on berm surface. Tension cracks parallel to road and located in parking lot area between pavement and work tent, 2" at widest; Saw general swirl in water not a deep vortex, circular motion, on upstream side of embankment; Water level may have been 1-2' below road surface at this point; Berm on upstream side still above water level; Water seeping through area near water line and toe around trees/bushes; Water through part of slope, small seepage

06:20 National Weather Service confirms dam watch

06:25 Call from Mr David Fink to M Ryan: Roadbed subsidence started 1-2 inches; Did not want to risk end loader on road; Too dangerous to dump riprap in hole; M Ryan moved to EOC

06:30am Dam watch through NWS Confirmed to M Ryan. A Bardgett observations: nothing of note exiting d/s of trees, lot of water coming through; Minor seepage, downstream slope low , Per Morris Wruck water running out of riprap 40-50ft south of wall

07:30am County Engineer moved to north side. T Gorman noted 7-8am road subsidence in line with the sinkhole in the downstream direction, subsidence noticed at the time county equipment was arriving on site

08:00am Per Mr David Fink, Reservoir (approximately 3 ft below crest of dam at 126.8-127 per visual inspection); Dip in road at the same location as fence sag and whirlpool; Hole on north embankment at culvert; Immediately along fish ladder is where water started going over road and toppling trees. Scott Kinseth arrived at dam; water high but not over dam; Per Morris Wruck 3ft of freeboard at embankment section

08:15am Jon Garton notified; Report of 1.5 ft of freeboard at dam; Road settled 6-12 inches

08:30am Scott Kinseth arrived at dam, water percolating through grass at downstream edge of northern concrete non overflow section; Scour started on north side, left of powerhouse

09:00am Per Mr David Fink, dirty flow at toe of downstream slope, 5-10ft (horizontal) from bottom of fish ladder, orange in color, exiting over rock outcrop at toe of training wall, flow in riprap more diminished, Mr David Fink sent out full warning,; Delhi FD, Ryan FD, Hopkinton FD, Jones County EMA; whirlpool 50-75 ft from spillway training wall, 3-4 ft downslope from guardrail-8-9ft from edge of pavement; Noticed cloudy/dirty water coming/blowing out of bank/riprap on downstream side near bushes/trees; Seepage to left and up higher; Running at other location as well; Issued mandatory evacuation downstream; water not over roadway; Running water exit above tailwater; Hole on north side upstream face showed up; 3 ft below dam,

Jon Garton call to Chip Hughes (LDRA): Reservoir 6 inches below crest and rising

09:30am Dirty water at toe of short section, exiting over rock outcrop at toe, road dip well defined, water exiting above tailwater, hole on north embankment upstream face appeared, water 3-3.5 higher than normal(around 127FT local Datum)

09:40am Call to Chip Hughes (LDRA): Report of water 1-2 inches over the road, reservoir rising 1 foot per hour; 1.5 miles upstream Hartwick bridge deck under 4-5 ft of water Call to B Leonard from Mr David Fink: water over the road at narrow part of levee, Mr David Fink says to evacuate everybody

10:00am Downstream shoulder starting to develop a hole, widening within 10' of concrete wall/structure, not far from wall; Water is going over roadway, a minor sheet of water/flow 10' wide (not where road is dipped), stopped and started; Area being overtopped right next to concrete section, 10'; Overtopped area, minor sheet flow, south of settled area; Shoulder area eroded at downstream end of sheet flow; Water coming through north side downstream wall and bubbling up through fill; Water coming through cracks north DS wall, water bubbling up through the ground near wall at dam location near curve on downstream side at north

10:37am Chip Hughes call to Scott Kinset : Water over the road

11:30am Call from Anthony Bardgett to Mike Ryan: water leaving south side of dam, flowing through the roadbed; roadbed is going to washout

12:00 Noon Antony Bardgett observes lots of water going around edge of parking lot area, over road, and maybe under road as well (near/around dipped area); Heavy flow over top of dam and underneath road

12:10 pm Per Tom McCarthy water flowing over road since 10:30, **trenched** through road, water no more than 1.5ft over road; Arrived at Hartwick Marina

12:15pm Per Mike Ryan And Anthony Bardgett:Roadbed collapses

12:22pm Report to Jon Garton from Chip Hughes - erosion stalled around 50% road washout;100-150ft south of spillway ; from Tom Mc Carthey reports 50% Road washout

12:30pm Per Mr David Fink Water started going over roadway, washed into trees, and eroding large area of trees downstream; 15-30ft trees toppling; 25% breach; Roadway caved in starting from dam and moving wider.
Per Tom McCarthy; Arrived at Delhi Dam; Water trenched through road, asphalt surface downstream chipping off, midway within tree section

12:45pm Crest Road 75% breach per Antony Bardgett;

12:55pm Entire width of road, 8-12 ft section of road dropped gradually and then progressed to south; Asphalt/pavement section in road/embankment dropped 6-7 inches and then started to crumble and go; Seemed like first section to start was close to dam / concrete wall, but unsure exactly where; Then, each section next started and continued to other side along embankment section by section

01:00pm Tom McCarthy reports the remainder of embankment with Upstream chain link fence collapses (Editors Note First indication of complete upstream to downstream breach

01:02pm Tom McCarthy reports 60ft wide breach through roadway

At least three area of active sinkholes have been identified in the investigation of this event; Two by the eyewitnesses and a third found in the post breach inspection by the IPE

From this information, the initial embankment deterioration is internal erosion (piping) as numerous eyewitnesses describe deteriorating conditions and typical piping characteristics (whirlpools, road subsidence, and cloudy discharge) several hours prior to overtopping of the embankment. Eye witness accounts include descriptions of discharges from riprap on downstream slope and cloudy discharges at the toe before overtopping began

The close coordination of all parties involved speaks of good emergency response training. However the observation of the whirlpool with a vortex at 3:30 am July 24, 2010 should have been sufficient to start emergency measures immediately, at least in the mobilization of equipment, and materials and notification to county engineer and the state DNR. Also, more frequent inspection of the embankment with a clear unobstructed view of the downstream slope and toe may also have yielded an emergency warning at an earlier time.

VIII

Report of Findings

Delhi Dam was breached on July 24, 2010 after two days of heavy rain in the drainage basin above the dam. The dam breach initiated about 1:00 pm on July 24, 2010 and resulted in an estimated peak breach outflow of about 69,000 ft^3/s. The flood and the dam breach resulted in extensive property damage in the reservoir above the dam and in the communities downstream of the dam. No loss of life occurred as a result of the dam breach. There were a number of factors that influenced the breach of the dam. These included: the design of the dam, which included a reinforced concrete core wall as the primary impervious element in the dam; the embankment materials, which appear to have consisted of a low plasticity sandy clay; the limited ability of the dam to pass a major flood (given the spillway capacity was initially designed to about 25,000 ft^3/s); and, the binding of one of the spillway gates preventing its full opening during the flood event. These items are more fully explored below. The findings are broken down into seven categories: Dam Design and Construction; Dam Performance Prior to July 22-24 Flood; Dam Performance During July 22-24 Flood; Alternative Scenarios for Reservoir Operations During July 22-24 Flood; Regulatory Oversight, Dam Safety Reviews, and LDRA Maintenance; Cause of Dam Breach; and Recommendations.

Dam Design and Construction

- There was limited information on the dam materials in terms of gradations of the materials and density of the in place embankment. It appears that the dam embankment consisted of a homogeneous material, with a reinforced concrete core wall placed upstream of the centerline of the dam. Samples from the remnant of the embankment were tested and it was determined that the material was a sandy clay with low plasticity.

- The core wall was placed on top of a sheet pile cutoff wall that extended to rock in most areas. The sheet pile cutoff stopped about 20 ft before the right spillway wall on the left side of the embankment. The top of the cutoff wall extended to within about 6 ft of the crest of the dam.

- The concrete cutoff wall on top of a sheet pile wall created a rigid element in the dam that would not settle over time, as compared to the embankment on either side of the cutoff wall which did settle over time. This situation likely created differential settlement in the area of the cutoff wall that caused low stress that could lead to cracks in the embankment fill that could have emanated from the cutoff wall. The potential seepage path created by the cracks from the cutoff wall and the low plasticity embankment material created a situation where internal erosion of the embankment could initiate and progress quickly.

- The spillway was the primary waterway for passing flood flows at Delhi Dam. The wicket gates in the old power plant have a discharge capacity of about 500 ft^3/s but this flow is relatively small compared to the spillway capacity. The spillway is regulated by three 25-foot wide by 20-foot-high vertical lift gates. With all three gate fully opened and the reservoir at elevation 904.8 NGVD29 (130 ft local datum), the estimated spillway capacity is about 32,000 ft^3/s.

Dam Performance Prior to July 22-24, 2010 Flood

- No adverse performance of the dam was reported to the IPE. No significant seepage had been reported at the downstream toe or on the downstream face of the dam
- Although the embankment performed well up to the recent event, it is very possible that prior loadings did not achieve a water surface elevation that exceeded the top of the core wall (EL 898.8 ft) or have a sufficient duration to develop internal erosion.
- The spillway gates have been difficult to operate in the past. The gate guides are tapered at the bottom and sometimes the gates would stick in the closed position or at small gate openings. A crane had been used in previous floods to operate the spillway gates.

- The lack of maintenance of the embankment section immediately south of the spillway and the 2H:1V downstream slope made inspection of the dam for seepage flows difficult.

Dam Performance During July 22-24 Flood

- During the July 22-24 flood, Gate 3 could only be opened 6 ft during the flood. This was a significant reduction in the spillway capacity The spillway Gate 3 was never opened any more than 4.25 ft during the entire July24, 2010 flood
- Nothing out of the ordinary was observed related to the dam performance during the July 22-24, 2010 flood, until the reservoir water surface exceeded the top of the core wall, at elevation 898.8 ft. Within about 8 hours of this occurring, vortex in the reservoir and sinkholes on the upper portion of the upstream face of the dam were observed. The first vortex was noticed about 40-50 south of the concrete structure; the second noticed later was estimated about 100ft south of the concrete structure. Seepage from the downstream slope was first observed around 6:00am July 24, 40-50ft south of the spillway training wall. At 6:00am, settlement of the dam crest was observed in the areas where the vortices and

sinkholes were first observed. All of this evidence is consistent with internal erosion occurring in the portion of the embankment above the top of and downstream of the concrete core wall.

- The dam breach began to accelerate around 12:30 pm on Saturday, July 24th. The dam breach was caused by internal erosion of the embankment, flows over the embankment and structural failure of the thin concrete core wall. The breach of embankment dam occurred about 1:00pm on July 24th. The concrete core wall appeared to have failed due to differential loading caused by the flood and erosion of downstream embankment soils As erosion of embankment soils continued, sections of the core wall also toppled ceasing at a maximum breach width of 235 ft. It is likely that the concrete core wall slowed down the rate at which the embankment dam breached.

- The breach of Delhi Dam did not cause any loss of life. This is attributed to several factors: the concrete core wall likely slowed down the rate of the dam breach; warning of dam failure was sent several hours before the breach; the flood wave was dissipated in farm fields, which reduced the level of flooding in the downstream communities of Hopkinton and Monticello; door to door warnings were issued in Hopkinton and Monticello which evacuated residents whose homes would have been inundated.

Alternative Scenarios for Reservoir Operations During July 22-24 Flood

A number of scenarios were evaluated to help determine if different spillway operations would have made a difference in reservoir levels and the breach of the dam.

- One of the items explored was the gate openings for the spillway. A routing was performed in which all three gates were opened to 18 ft, which was the maximum opening achieved by Gate 1 during the July 22-24, 2010 flood. The flood routing results indicated that Delhi Dam would not have overtopped if all three gates had been fully opened. However, the reservoir would have exceeded the top of the core wall by up to 2.4 ft for about a day and it is likely that internal erosion would have initiated in the embankment. Based on the duration of seepage that likely would have occurred through the embankment, it is judged that the dam would have suffered damage and possibly a total breach.

- One of the criticisms from downstream residents is that the dam operators should have lowered the reservoir in anticipation of the peak flood inflows. Routings were performed that evaluated the effect of lowering the reservoir by 2-, 4- 8- and 10-ft at the beginning of the flood.

The results of these flood routings indicated that the maximum reservoir water surface and the duration of high reservoir water surface elevations would have been essentially unchanged. This reflects the fact that the reservoir volume is relatively small in comparison to the flood volume and any space that was created would have been filled prior to experiencing the peak flood flows.

Regulatory Oversight, Dam Safety Reviews and LDRA Maintenance

- The State of Iowa Department of Natural Resources had regulatory oversight of Delhi Dam. The state inspects high-hazard dams on a 2-year frequency and moderate and low hazard dams on a 5-year frequency. Delhi Dam was inspected in 2010, 2007 and 1999 by the DNR inspector and by an engineering firm in 2002 and 1997. Delhi Dam has been classified as a moderate-hazard dam by the DNR and the inspections have been on at least a 5-year interval. The DNR Technical Bulletin No. 16, Design Criteria and Guidelines for Iowa Dams, identifies Moderate Hazard dams as: "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 dams are identified as: "Structures located in areas 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." Given the conditions downstream of Delhi Dam, a moderate hazard dam classification is judged to be appropriate. Chapter III of Technical Bulletin No. 16 indicates that the design flood for moderate hazard dams should be one half of the probable maximum flood (PMF) and that the dam should not overtop for this flood. Delhi Dam had the capacity to pass about the 100-year flood and would have overtopped for a flood representing one half of the PMF, even with the spillway gates fully functioning.

- Two detailed dam safety inspection reports were prepared for Delhi Dam by Ashton-Barnes Engineering in 1997 [Ashton1998] and by Ashton Engineering in 2002 [Ashton 2002]. Both reports included documentation of an inspection of the dam, a discussion on stability of the dam, spillway adequacy and adequacy of maintenance and methods of operation. The reports also contained conclusions and recommendations. The 1998 report was more comprehensive and the scope was more clearly defined. The scope of the 1998 report included the following: perform analytical stability analysis of all pertinent dam features; perform stress analysis, as needed, to assess the condition of individual project elements; perform hydrological and hydraulic analysis required to assess spillway adequacy relative to current criteria; and evaluate the maintenance and current methods of operation. The 1998 inspection report stability and stress evaluation focused on the concrete portions of the dam and did not specifically address any stability issues with the embankment dam. The 1998 report concluded that the spillway could just handle the 100-year flood event (estimated at that time). The dam was classified in the report as an intermediate size, low hazard potential dam, which would require that the dam handle between the 100 year flood and one-half the Probable Maximum Flood. Both the 1998 report and the 2002 report concluded that the spillway had adequate capacity. A recommendation was made to perform a study on ways to improve the spillway gate hoist system, which was inoperable. The 2002 inspection report had a similar focus and similar conclusions as that of the 1998 report. It was stated that "The equipment used to operate the control gates for the spillway has been completely reworked since the 1997 inspection. The current methodology of operation is satisfactory."

- The Lake Delhi Recreation Association owned, operated and maintained Delhi Dam. Repairs of the gate hoisting mechanisms had been performed over the years and a complete replacement of the hoisting system was designed and was planned to be installed in 2010. The original hoist system consisted of motor operated hoists that included steel wire ropes attached to the bottom of the gates (2 cables per gate). The new hoists would have consisted of screw type actuators. The actuators would lift and lower the gates from the top instead of the bottom of the gates. It was also planned to strengthen the top of the gates. Repairs were made to the spillway gates and hoist equipment over the years, the most recent being in 2009. The DNR Inspection report dated 8/17/2009 reported that Gate 1 (the left most gate looking downstream) could only be opened 8 ft due to damage incurred during the 2008 floods. This gate was opened fully during the 2009 flood. There was another issue identified in the 8/17/2009 Inspection Report. It identified a hole in the left pier for the right spillway gate (Gate 3). The hole was located about 15 ft below the top of the spillway gate. The hole extended completely through the pier behind the gate guide. In a follow-up note to the files from Dave Allen on 1/21/2010, it was noted that the concrete repairs had not been made. The inspection report had identified a completion date for the pier repairs of 12/31/2009. An October 13, 2010 inspection report by Stanley Consultants indicated that the repairs had not been made prior to the July 22-24 flood. They report the condition of the left pier in their report:

  > "Severe concrete deterioration and damage was observed in the piers adjacent to Gate 3(Photos III-4 and III-5). The left embedded steel guide slot for Gate 3 had broken away and moved outward from the pier, spalling the adjacent concrete and exposing the underlying reinforcing. It was reported that Gate 3 seized during the flood and

could not be fully opened. The out of plumb guide slots likely caused this situation."

Cause of Dam Breach

From the eyewitness descriptions, photographic and video evidence and limited excavation investigation, the cause of the dam breach was internal erosion in the embankment coupled with overtopping flow. The internal erosion was most likely caused by a seepage path initiated along differential settlement of the embankment material adjacent to the core wall. The failure mode was triggered by reservoir levels that exceeded the top elevation of the concrete core wall which was exacerbated by the inability to open the third gate beyond the 4.25 ft measured in the post breach investigation.

The location and design of the concrete core wall and the fact that it did not extend to the crest of the dam created more favorable conditions for internal erosion of embankment materials once the reservoir reached the elevation corresponding to the top of the core wall. The IPE believes that any flood of sufficient magnitude which raised the reservoir above the top of the concrete core wall for a more than several hours would have resulted in the embankment experiencing piping/internal erosion.

If internal erosion did not occur, the duration of 16 hours and maximum depth of 1.4 ft of overtopping predicted by the flood model (with one gate malfunctioning) would have likely caused a breach via overtopping and headcutting erosion. Overtopping erosion to the point of breach was predicted with WINDAM, a NRSC erosion program. Several other factors that would add to the likelihood of overtopping erosion are the downstream slope of 1V:2H, erosion features located at the toe as described in recent inspection reports, the rock toe and inclusion of the 1920's roadbed, unknown (but likely low) insitu soil densities and the trees and vegetation on the downstream slope. Conversely, if the dam had not experienced overtopping flows above the original dam crest elevation (this would have required that Gate 3 was fully functional July 22-24, 2010 flood event), it is possible the internal erosion mechanism by itself would have lead to the breach of the dam.

Recommendations

The scope of the IPE investigation was limited. Several recommendations are made that will add to a better understanding of the breach at Delhi Dam:

1. Investigate the remaining sinkhole and the flow path from the sinkhole to its terminus.
2. Conduct a complete investigation of the remnant of the embankment, 1967's berm and foundation soils including but not limited to classification of soil and critical material properties.

Additional recommendations are made to address possible rebuilding efforts at Delhi Dam:

3. Remedial measures for the north embankment section should be developed and included in any reconstruction scenario.
4. If the owner elects to pursue a FERC license, it would be wise to delay any further investigations, demolition, reconstruction designs and analyses until coordination and procedures with FERC are established.

Finally, recommendations are made that address issues related to managing dam safety issues and a dam safety program.

5. More consistent approaches should be developed for classifying dams according to hazard and achieving compliance with the associated design standards. The DNR classified Delhi Dam as moderate hazard, but Ashton Engineering classified the dam as low hazard. This has an impact on the design flood standard that is applied to dams.
6. Dam inspectors performing inspections for the DNR and consulting engineering firms performing dam safety evaluations should have strong backgrounds in dam engineering and potential failure modes analysis. There were design weaknesses at Delhi Dam that an experienced dam engineer would recognized likely led to additional investigations.
7. The failure of Gate 3 to fully operate during the flood appears to have been caused by the failure to complete concrete repairs behind the left gate guide. Education and enforcement mechanisms are needed to clearly identify critical dam safety issues and their impacts and to ensure these issues are resolved quickly.
8. Review/ update the estimated return period for the July 2010 flood event based on historical inflows at Delhi Dam.

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