MILAN DAM (ID #713)
CONDITION ASSESSMENT AND HYDROLOGIC STUDY

Prepared For:

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Executive Summary

Milan Dam was constructed in 1935 and is currently operated by the City of Milan. The dam originally was intended to generate hydropower though never served this purpose. The City now uses the dam to impound the 22 acre Ford Lake.

This report consists of the results of detailed inspections of the principal and auxiliary spillways, the embankments, and related structures. The report also examines the hydrologic capacity of the spillway and recommends needed improvements.

Based on these inspections, the condition of the principal spillway is satisfactory, with no significant deficiencies noted. Specific concrete repairs are suggested as preventative maintenance. The concrete structures of the auxiliary spillways are in satisfactory condition, though their general condition is noted as fair due to the presence of debris and sediment that block full water passage; and the lack of an adequate means for keeping inlet screens clear.

Based on hydraulic modeling, it is confirmed that the embankment overtops at the 50, 100, and 200 year flood events. The downstream embankments have a relatively gradual slope, but the calculated overtopping velocity at the 200 year spillway design flood suggests that armoring immediately downstream of the crest should be installed to prevent incising. In addition, armoring at the steep slopes farther downstream is recommended. Also, the spillway abutment walls were found to overtop at the 200 year event. These walls should be raised to above the 200 year water elevation and flood walls added to the left and right embankments to prevent overtopping in close proximity of the principal spillway. Construction of floodwalls would cause some constriction of the overtopping area, but if the existing 48-inch drawdown valve in the spillway is utilized, the 200 year flood elevation will not increase beyond its calculated level.

If the above modifications are made to the spillway and embankment, the bypass spillways and tunnels can be closed and abandoned. In general, it is recommended to close the tunnels if possible. While the tunnels can be cleaned and modified to ensure good passage of water, their maximum potential flow still does not provide enough hydraulic capacity to prevent overtopping, and therefore the added maintenance cost and liability of the tunnels is not justified.

This report also considers alternatives for adding a bypass spillway and for removing the dam.

The results of this report should be considered together with the results of recent studies on installing a promenade along Wabash Street (January, 2012) and dredging of Ford Lake (January, 2012) in order to determine a long-term strategy for Ford Lake in view of the City's goals.

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APPENDICES

1.0 Introduction

Stantec was hired by the City of Milan to conduct a detailed condition assessment of the Milan Dam and appurtenant structures. Pursuant to the MDNRE letter dated August 4, 2010 **(Appendix B)**, this report determines spillway capacity and overflow rates, and makes recommendations for corrective actions.

The following are addressed in this report:

- Results of a detailed condition survey of the existing dam and appurtenant structures
- Topographic survey results of embankments and relevant structures
- An assessment of the spillway and embankment conditions
- Hydraulic analysis determining total hydraulic capacity of the spillway and overtopping flows
- Alternatives for improving overflow spillway capacity with probable costs

Taken together with recent studies on dredging alternatives for Ford Lake (January, 2012) and installation of a promenade along Wabash Street (January, 2012) this report is intended to assist the City in determining long-term plans for Ford Lake.

Acknowledgements

Stantec would like to thank Milan leadership and staff for their cooperation in completing this project, particularly: the Ford Lake Committee, for providing guidance on the project; Martha Churchill, for the historical information on Milan Dam; Bob Grostick, who gave us access to City facilities; and the Milan Historic Society for access to the benchmark inside the old Fire Barn.

2.0 Background

2.1 NOMENCLATURE

This report refers to conditions with respect to two general categories: 1) Dam Safety; and 2) Physical Condition. The Dam Safety category is concerned with the ability of the dam and embankments safely to impound the water of Ford Lake. Though this report does not constitute a Dam Safety Report, it nevertheless evaluates conditions based on the following Dam Safety Criteria which are accepted by the MDEQ and National Dam Safety Review Board.

2.1.1 Dam Safety Terminology

SATISFACTORY

No existing or potential dam safety deficiencies are recognized. Acceptable performance is expected under all loading conditions (static, hydrologic, seismic) in accordance with the applicable regulatory criteria or tolerable risk guidelines.

FAIR

No existing dam safety deficiencies are recognized for normal loading conditions. Rare or extreme hydrologic and/or seismic events may result in a dam safety deficiency. Risk may be in the range to take further action.

POOR

A dam safety deficiency is recognized for loading conditions which may realistically occur. Remedial action is necessary. POOR may also be used when uncertainties exist as to critical analysis parameters which identify a potential dam safety deficiency. Further investigations and studies are necessary.

UNSATISFACTORY

A dam safety deficiency is recognized that requires immediate or emergency remedial action for problem resolution.

2.1.2 General Terminology Describing Physical Condition

The Physical Condition of structures refers to specific deficiencies that may or may not be potential dam safety issues. Deficiencies that may not be dam safety issues include those that are not severe enough to reduce the satisfactory rating (see above); and those on structures that do not relate to impounding Ford Lake. Even though a certain Physical Condition may not constitute a dam safety issue, it is noted as a maintenance item that should be repaired since it could lead to an eventual dam safety issue and/or lead to more expensive repairs in the future.

Descriptive terms used for noting physical condition will include:

- Stage: early/advanced
- Severity: minor/moderate/significant

Specific conditions that may be referenced include:

Concrete

- Spalling: Material is visibly deteriorated in areas as noted. Usually suggests deeper disintegration of material.
- Delamination: Sub-surface damage, usually found by sounding tests.
- Efflorescence: White minerals from the concrete bleed to the surface. Often an early sign of deterioration and usually indicates hydrostatic water pressure on the opposite side.

Embankments

- Subsidence: A lowering of the earth surface. Usually suggests sub-surface settlement or erosion.
- Erosion: Earth material is worn away from the surface, usually due to water running over the top.
- Scarping: Earth material is worn away from the shoreline, usually due to wave action from below.
- Seepage: Water moves through the embankment soils due to the difference in hydrostatic pressure between the two sides.

RIGHT and LEFT are with respect to facing downstream.

2.1.3 Abbreviations

CFS	cubic feet per second
FPS	feet per second
FEMA	Federal Emergency Management Agency
FIRM	Flood Insurance Rate Map
FIS	Flood Insurance Study
MDEQ	Michigan Department of Environmental Quality
MDNRE	Michigan Department of Natural Resources and Environment (now MDEQ)

NAVD 88	North American Vertical Datum of 1988
NGVD 29	National Geodetic Vertical Datum of 1929
WS	water surface
WSEL	water surface elevation

2.2 DESCRIPTION OF EXISTING FACILITY

The original Milan Dam was built in the late 1800's and removed by Henry Ford in 1937. At that time, the present Milan Dam was constructed as part of a Village Industry project. The Ford project expanded the existing mill pond and re-directed the Raisin River approximately 500 feet south of its old alignment to a new gravity concrete overflow spillway. The new facility was designed to generate hydro-power, but the hydro capacity apparently was never fully utilized.

The principal spillway is a 60-foot concrete overflow structure that includes a 48-inch diameter drawdown drain valve (**Appendix C, Photo D-1**).

The existing earthen embankments were constructed at the time of the present concrete spillway and extend approximately 620 feet to the left (north), and 200 feet to the right (south), of the spillway. It is presumed that the embankments were made of native earth material. Wabash Street, a bituminous asphalt paved road, overlays the embankments and traverses the spillway via a bridge. Earth fill has been added to the downstream side of the embankments, effectively making them in excess of 100 feet wide in most locations (**Photo D-2**).

The former location of the Raisin River is near the low point of the present embankment, approximately 500 feet left (looking downstream) of the current principal spillway.

In the right earthen embankment are two (2) auxiliary spillways that used to serve as headrace inlets for two different hydro power installations. The left auxiliary spillway inlets to the Henry Ford power house, and the right auxiliary spillway leads to an old mill building approximately 600 feet downstream of the pond. Both spillways are 5 feet wide, and discharge into nominal 5 foot wide x 4.5 foot high tunnels that traverse Wabash Street.

The existing dam is rated as a Significant Potential Hazard.

2.3 CRITICAL ELEVATIONS AND DIMENSIONS

The following is based on the latest survey and as-built information (elevations given in feet, NGVD 29):

Principal spillway		
	crest elev.	686.92
	width	60 feet
	abutment walls, top of concrete elev.	693.08
Drawdown valve		
	size	48-inch diameter (nom.)
	invert elev.	677.9 +/-
Left auxiliary spillway		
	crest elev.	687.13
	tunnel width	5 feet
	tunnel height	4.5 feet
Right auxiliary spillway		
	crest elev.	687 +/-
	tunnel width	5 feet
	tunnel height	4.5 feet
Embankment crest elev.		692.4
Normal headwater elev.		687 +/-
Normal tailwater elev.		679 +/-

2.4 RECENT INSPECTIONS AND CORRESPONDENCE

2.4.1 Part 315 Dam Safety Inspections (October 2008)

The latest Part 315 Dam Safety Inspection was conducted by MDEQ in 2008. The conclusions of the report called for: investigation of cracking in the downstream retaining wall near the powerhouse; removal of shrubs around the principal spillway and head race inlets; completion of an Operations and Maintenance plan; and update of the Emergency Action Plan. The report noted that the 1992 inspection report concluded that the spillway could pass the required 4600 CFS, but with no freeboard. The 2008 report determined that, due to mitigating circumstances, the minimal freeboard was acceptable. (**Appendix B**)

2.4.2 Letter from MDNRE on Spillway Capacity (August 4, 2010)

This letter is a response to a request to close one or both of the auxiliary spillway tunnels. Although they did not include in their analysis the capacity of the auxiliary spillways, MDNRE (now MDEQ) rejected the request, stating that the principal spillway did not have adequate capacity to handle the 0.5% (200 year) flood event of 4600 CFS, and therefore, whatever additional capacity was provided by the tunnels had to be left intact. MDNRE found that, based on the latest survey data, the left embankment overtops during the 0.5% event. The letter raised concerns that such overtopping could cause erosion on the downstream bank. The letter states that the spillway needs to have adequate capacity as part of 315 regulations, and suggests that enhancement of the downstream embankment should be included with this. (**Appendix B**)

2.5 RECENT REPAIRS, MODIFICATIONS AND ACTION TAKEN

2.5.1 Fencing over Auxiliary Tunnel Entranceways

After the 2008 inspection, fencing was installed over the openings for the left and right auxiliary spillways. The fencing was intended as a safety feature and to prevent debris from entering the tunnels.

2.5.2 Reconstruction of Riverfront Retaining Wall

The powerhouse outlet and riverfront retaining wall were reconstructed in 2010/2011. This work corrected the deficiency as noted in the October 2008 MDEQ Dam Safety Report.

2.5.3 Clearing of Shrubbery around Spillways

Based on our inspection, no issues were noted with shrubbery around the spillway structures.

3.0 Survey

Stantec conducted topographic survey of the spillway, auxiliary spill structures, earthen embankments and downstream overflow areas. The goals were to: establish accurate embankment and spillway crest elevations; determine the path of flow for overtopping events; and characterize the downstream overflow areas with regard to slope and type of ground cover.

Results of the survey are in **Appendix A** and were used as a basis for the hydraulic study.

Initial surveys of Ford Lake and the downstream areas used a GPS-derived datum that was based on NAVD 88 due to no USGS benchmarks located in the project area. Subsequent to these initial surveys, a USGS benchmark that had been enclosed within the old Milan Fire Barn was found, and the previous datum was corrected on this basis. The correction factor between the two data is as follows: add 1.01 feet to a GPS-datum elevation to obtain the elevation in NGVD 29. The principal spillway crest elevation under the GPS datum was 685.91. In NGVD 29, the spillway crest is 686.92, which is the elevation reported above in **Section 2.2**.

All elevations in this report are on the basis of NGVD 29, which is the same datum that FEMA used for their Flood Insurance Study.

4.0 Existing Conditions

Stantec conducted inspections of the existing facilities on various dates, including June 26; June 29 and August 17, 2011.

4.1 VISUAL INSPECTIONS

4.1.1 Embankments

Stantec inspected the embankments both from the shore and from a boat in Ford Lake.

The left upstream embankment was in fair condition. Erosion and scarping along the shore extended several feet above the water line, posing a threat to the stability of Wabash Street **(Appendix C, Photos E2 and E3)**. Though the loss of embankment was comparatively minor with respect to the remaining width of the embankment, the erosion could increase the chance of the embankment developing an incision in the event of overtopping.

The right upstream embankment was in satisfactory condition. Minor to moderate scarping existed along the shoreline **(Photos E4 and E5)** which should be repaired, but did not constitute a dam safety issue.

The crest of the embankment is overlain by Wabash Street **(Photo E6)**. No settlement in the road surface that might suggest subsidence or loss of material in the embankment was noted. The crest was in satisfactory condition.

The downstream embankment on both the left and right sides was in satisfactory condition, although modifications are recommended to protect the embankment in cases of overtopping (see **Sections 5.0** and **6.0** below). On the left side, the embankment gradually slopes down as part of a park area. The numerous trees were not a significant risk due to the width of the embankment **(Photo E7)**. The downstream right embankment aligns with the generally flat yard of the City Hall **(Photo E8)** and Neckel Court, which also has a gradual drop in grade downstream from Wabash.

4.1.2 Principal Spillway and Abutments

A detailed inspection of the principal spillway was conducted on August 17, 2011. Sandbags and wood were used to obstruct flow over the spillway in selected areas, while visual inspection and hammer soundings of the downstream face were conducted from the bottom. The Lake elevation had been lowered by opening the drain valve overnight, but the valve had to be closed during the inspection. Despite the effort at lowering the Lake elevation, flow over the dam was substantial and difficult to control during the inspection. It was not possible to dry out the entire surface, but sufficient amounts were dried out to provide a representative sample of the dam. After a portion was dewatered, biological growth was removed and the wall then observed and sounded by hammer. A tape was strung over the spillway from the left to the right abutment to provide a location reference during inspection **(Appendix C, Photos S1 and S2)**.

The downstream face of the principal spillway was in satisfactory condition. The concrete surface appeared to be free of spalling or delamination. One small hole was identified and this was determined to be only a minor surface flaw. Near the 48-inch diameter drawdown valve, hollow sounding concrete and minor spalling were identified. These were close to the sharp corners where the 48-inch opening intersected with the sloping plane of the downstream surface **(Photos S3 and S4)**.

The gate valve was visibly leaking. The amount was not quantified, but it appeared to pose no dam safety risk.

The crest of the spillway was sounded in several locations. No delamination or significant spalling was noted.

Downstream of the spillway is a concrete apron. Two or more rows of baffle blocks were noted beneath the water surface. Some of the blocks apparently had been damaged and partly broken off. Further downstream under the bridge was a nominal 2-foot sill. The concrete apron extended beyond the sill to approximately 10 to 15 feet downstream of the bridge **(Photo S9)**. The river bed connected to the downstream of the concrete apron roughly at the top of concrete elevation. No scours or other holes were found downstream of the apron.

Spalling and deterioration were noted on the left abutment, upstream and downstream **(Photos S5 and S6)**. These conditions should be repaired within the next 5 years to prevent damage from becoming severe.

The right abutment showed efflorescence in a crack near the valve gate **(Photo S7)**. This should be investigated and repaired as necessary within the next 5 years.

Downstream of the bridge, most of the right retaining wall was reconstructed in 2010/2011. Approximately a 50-foot section adjacent to the bridge is original **(Photo S8)**. This older portion of wall showed no signs of distress or severe deterioration, although its age warrants re-inspection every 5 years.

4.1.3 Left Auxiliary Spillway

The left auxiliary spillway is a 5-foot wide wood stop log weir that discharges into a 5 foot wide x 4.5 foot high concrete tunnel. The tunnel is approximately 110 feet long and discharges into the old wheel pit.

The left auxiliary spillway was originally intended to supply water for power generation, but now serves only as an auxiliary spillway. Under normal conditions, the water enters from underneath a screen wall and over the log weir **(Appendix C, Photo L2)**. In high flow conditions, the open front of the structure acts as an overflow weir **(Photo L1)**. Just to the right of the main inlet there is a smaller inlet structure of uncertain function. It possibly served as an inlet for an abandoned 6-inch pipe found in the dive inspection of this tunnel. (Please see **Section 4.2** below).

Spalling and deterioration were noted on the top of the concrete walls of the spillway structure. Fencing over the intakes prevents intrusion of large debris into the tunnel, but is also an impediment to flow. Fence of this type is not easy to clean and is therefore susceptible to frequent plugging, reducing the value of this spillway as an auxiliary spillway.

The wood log weir was in satisfactory condition. The spill edge was intact and level, though there was minor leakage between the logs. The logs are set into steel channels and are removable.

The condition of the concrete in the tunnel was satisfactory. For details, see results of the Dive Inspection in **Section 4.2**.

During reconstruction of the right downstream retaining wall in 2010/2011, the tailrace and wheel pit outlet were dewatered and inspected. The walls and ceiling of the tailrace showed minor efflorescent cracking, but the concrete was generally in satisfactory condition **(Photos L7 and L8)**. The concrete at the bottom of the wheel pit **(Photos L3, L4 & L5)** was satisfactory, with no serious defects noted. The draft tube, which is part of the auxiliary spillway, was plugged with wood and debris **(Photo L6)**. This material needs to be cleared from the draft tube if this tunnel is part of the required spillway capacity of the dam. Details on the interior of the wheel pit can be found in **Section 4.2**.

4.1.4 Right Auxiliary Spillway

The right auxiliary spillway, like the left, consists of a 5 foot wide log weir that discharges into a 5 foot wide concrete tunnel. The tunnel is part of the enclosure of what was once an open raceway to the mill building. The first 100 feet of the enclosure consist of the 5-foot wide x 4.5-foot high concrete tunnel. East of Wabash Street, the tunnel appears to connect to a 24-inch diameter gravity sewer pipe at an old concrete headwall that is now mostly buried. The gravity sewer pipe extends approximately 510 feet east to the old mill building. At the mill building, the pipe discharges into the old turbine wheel pit, which is now empty of machinery and only drains into the mill tailrace.

The wood log weir is in fair condition **(Appendix C, Photo R2)**. The spillway edge is uneven, making flow measurement difficult. Logs are not properly seated and the log bay significantly leaks through the logs themselves. Concrete on the inlet structure showed minor concrete deterioration **(Photo R3)**.

The concrete of the tunnel, as far as could be inspected, was in satisfactory condition. The tunnel, however, was significantly plugged with sediment and debris. (Further detail provided in **Section 4.2**).

In the mill building **(Appendix C, Photo R4)**, concrete in the wheel pit showed minor leaks and stalagmites, though no major damage was identified. The draft tube appeared to be clear at the top, though the discharge point could not be inspected due to the tailwater **(Photos R5 and R6)**. It seems probable that the lower end of the draft tube is below the river bed elevation. The wheel pit has an overflow spillway, which crests at elevation 686.96 feet **(Photo R5)**.

4.2 DIVE INSPECTIONS

On July 26, 2011 Great Lakes Diving conducted an underwater inspection of the Powerhouse and Millrace tunnels; and upstream of the 60-foot principal spillway.

The inspection was recorded using audio/visual equipment. A CD copy of the dive video is included in this report **(Appendix D)**.

<u>Findings by area</u>:

4.2.1 Left Auxiliary Spillway Tunnel

Nominal opening size = 5 feet wide x 4.5 feet high. This tunnel receives discharge from the left auxiliary spillway and directs it to the old powerhouse wheel pit.

The water in this tunnel was approximately 2 feet deep, leaving the top and much of the sides above water. Overall, the condition of the concrete was fair to satisfactory.

Table 1 – Summary Of Dive Inspection Findings, Left Aux. Spillway

Approximate Distance from Inlet (feet)	Remarks
10	Top and sides were satisfactory. Slight marine growth underwater on sides.
20	A 6-inch pipe was noted on the bottom right of the tunnel. The pipe ran longitudinal with the tunnel, and was approximately 1 foot from centerline to the right wall; and 6 inches from centerline to the bottom. An 8-bolt union was identified. There was approximately ½ inch of silt on the bottom of the tunnel. Concrete condition was satisfactory.
25	A concrete pipe support for the 6-inch pipe was identified. 6-inch pipe continued.
40	Approx. ¼-inch of marine growth on ceiling; 6 inches of silt on bottom.

	Minor rock-pockets identified in concrete of ceiling.
50	Approx. 6 inches of sediment on bottom. A 4-inch dia. X 5 foot long log was wedged underneath the pipe. Concrete condition was satisfactory.
60	Approx. 6 inches of sediment/pea gravel on bottom.
70	Flange identified for 6-inch pipe. Hardware intact.
80	Joint/crack identified at this location. Minor efflorescence and leakage noted. Concrete condition was fair to satisfactory. Approx. 4 inches of sediment on bottom.
85	Flange of pipe mostly buried in sediment.
90	8 inches to 1 foot of sediment on bottom.
95	Joint on concrete ceiling. Tunnel widened and ceiling dropped.
100	Approximately 6 inches of water. The rest was sediment (approx. 2 feet thick). Small, soft stalactites were on ceiling.
110	Limit of dive. Impassible beyond this point due to the presence of sediment in the tunnel and the lower ceiling height.

4.2.2 Powerhouse Wheel Pit (Left Tunnel)

The diver entered the wheel pit from inside the municipal building due to the restricted opening from the tunnel (please see **Table 1** above). The wheel pit was approximately 10 feet x 12 feet wide, with a depth measured as approximately 6 feet from the entry level floor down to the water. The diver reported approximately 3 feet of sediment on the bottom. The water surface in the wheel pit was common with that of the remainder of the Powerhouse Tunnel, and given the slow velocity of the water, it can be assumed that the water surface elevation from the tunnel entrance to the wheel pit was constant.

The middle of the wheel pit was the top of the old turbine inlet ring, measuring 36 inches in diameter. The inlet opening was covered with wood and other debris, but was still nominally open to the tailrace water level below. There was approximately 1 inch of water spilling over the edge of the 36-inch inlet ring.

The concrete ceiling of the wheel pit apparently was covered when the generator was removed. The concrete patch could be identified from below. Exposed rebar was noted on the bottom of the slab at the patch area.

Ladder rungs down into the pit were in poor condition. There were two open conduit ends in the pit, approximately 1-inch diameter each. There was also noted in the east wall a square opening, with cover.

4.2.3 Right Auxiliary Spillway Tunnel

Nominal opening size = 5 feet wide x 4.5 feet high. This tunnel directs flow from the right auxiliary spillway to the old mill race (see description in **Section 4.1.4** above).

The tunnel was flooded with turbid water. Visibility was very low. At the inlet of the tunnel, there were only a few inches of debris on the floor. Approximately 20 feet into the tunnel, the silt was as thick as 3 feet, making it impossible for the diver to pass further. In the accessible areas, marine growth was noted on the walls, but no delamination or spalling was identified in the concrete.

4.2.4 Principal Spillway and Retaining Walls

The spillway upstream face and retaining walls were inspected by the diver. There was a slight marine growth on the spillway face, but no spalling or delamination was noted on the concrete. The diver noted no deficiencies on the upstream wing walls.

The depth upstream was a maximum of approximately 10 feet, with shallower depths to the sides of this due to the deposition of sediment. No scour holes were found at the base of the dam. In the deepest part, a concrete apron was found that extended upstream of the dam. This apparently was the structural base. The river bottom was level with the upstream most part of the apron, with no undercutting of the slab noted. (The dive inspection did not check downstream of the dam for scour. Downstream was checked by wading on another date. See **Section 4.1.2** above.)

On the right side of the dam, the diver identified the bottom drain gate. There was no bar grate or screen in front of the gate inlet. Old steel members upstream of the gate apparently were from an earlier construction that had been demolished. The gate disc was a 4 ½ foot square plate that travelled up and down in two rails on the sides. Fully closed, the top of the plate was approximately 5 feet below the water surface. The face of the gate was approximately 5 inches off the face of the dam. Between the disc and the dam face, a circular thimble, approximately 4 feet in diameter, connected the valve to the concrete passage through the dam. No deficiencies were noted with the gate mechanism.

5.0 Spillway Capacity and Hydraulic Analysis

The spillway capacity of Milan Dam was evaluated for compliance with Michigan Department of Environmental Quality (MDEQ) dam safety requirements. The required spillway capacity for compliance with MDEQ regulations is passage of the 0.5-percent return-interval (RI) hydrologic event (200-year storm). This analysis was performed using site-specific topographic data collected as part of this study, information developed as part of previous studies, and hydraulic evaluations performed using a numerical hydraulic model developed for this work.

5.1 EXISTING INFORMATION

Relevant existing information obtained and reviewed for this work included a dam safety inspection of Milan Dam performed by MDEQ and the current Federal Emergency Management Agency (FEMA) Flood Insurance Study (FIS) and Flood Insurance Rate Map (FIRM).

5.1.1 MDEQ Dam Safety Inspection Report

A dam safety inspection report prepared by MDEQ dated October 29, 2008, was reviewed as part of this study. That report lists the dam as having an overall length of approximately 900 feet, a structural height of 18 feet, and normal pool hydraulic head of approximately 8 feet with 7.4 feet of freeboard. The dam is listed as having a “Significant” Hazard Potential Classification.

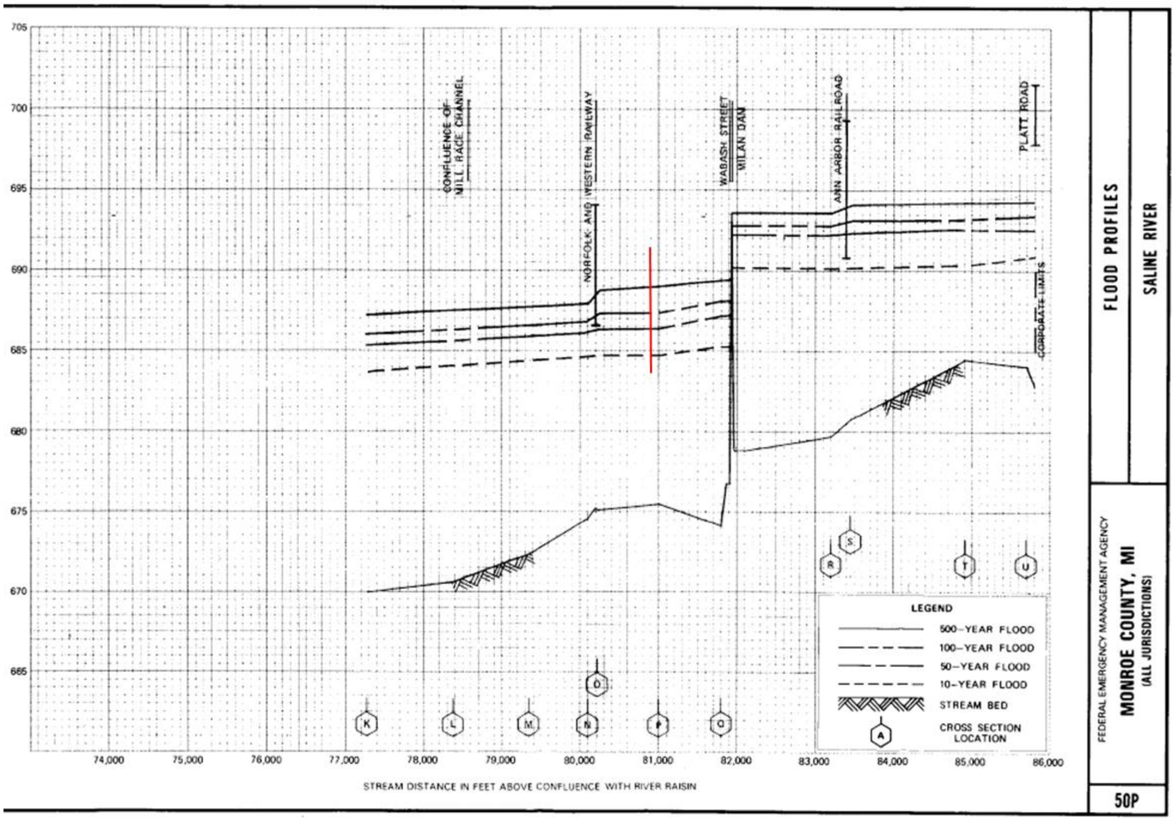
The principal spillway of the dam is a 60-foot wide concrete ogee-crest located approximately 20 feet upstream from the upstream face of the Wabash Street Bridge. The spillway abutment walls extend downstream from the dam and form the abutment wall of the referenced road bridge.

A 48-inch diameter drawdown valve is located within the spillway. To the right of the principle spillway are two auxiliary spillways that divert water to two separate out-of-service turbine buildings.

5.1.2 FEMA Studies

The previously-developed FEMA FIS and FIRM (FIRM Panel Map Number 26115C0035 D, Effective Date April 20, 2000) for Monroe County, Michigan were reviewed for this study. Peak flow statistics listed in the FIS for given return-interval hydrologic events are equivalent to values provided by MDEQ in correspondence with Stantec. Information shown on the Flood Profile Panel No. 50 of the FIS were reviewed and used to develop downstream stage boundary conditions for the project hydraulic model that is discussed in this report. This panel is shown in **Figure 1** along with a vertical line (colored red) indicating the downstream limit of the hydraulic model developed for this study.

Figure 1: FEMA FIS Profile of Project Reach



5.2 HYDROLOGY

Hydrologic peak flow statistics for the project reach of the Saline River that were used for this study were provided by MDEQ in an email to Paul Malocha of Stantec dated April 13, 2011. Peak flow statistics are presented in **Table 2**, with peak flows expressed in cubic-feet-per-second (CFS).

Table 2 – Peak Flow Hydrologic Statistics

Annual Return Probability (%)	50%	20%	10%	4%	2%	1%	0.5%	0.2%
RI (Years)	2	5	10	25	50	100	200	500
Peak Flow (CFS)	900	1,600	2,060	2,800	3,490	4,060	4,600	5,190

5.3 HYDRAULIC ANALYSIS

A hydraulic analysis of the project dam for use in evaluating spillway capacity was performed using a one-dimensional, numerical hydraulic model developed using the U.S. Army Corps of Engineers HEC-RAS (Version 4.1) software system. Two hydraulic model geometries were developed that included 1) a single-reach model with a single cross-section representing the dam spillway and the overlying section of Wabash Street, including the section of road that is overtopped during high-flow events, and 2) a model comprised of multiple reaches and junctions upstream and downstream from the dam. The later model was developed to allow for calculation of discharge and flow speeds for water discharging over Wabash Street during high-flow events, and is the basis for results presented in this report.

5.3.1 Geometric Domain

The hydraulic model geometric domain was developed using information obtained from a dedicated topographic survey performed for this study. The topographic model did not include bathymetric data downstream from the project dam, and assumed channel dimensions were therefore used for cross-sections of normally wetted areas. The channel inverts for the assumed cross-section dimensions in the normally-wetted portion of the river were established based on the flood profile presented in Figure 1. The use of assumed data in the channel appears to be reasonable in that the supercritical flow over the dam for all of the evaluated flows precludes downstream water surface elevations from influencing spillway capacity.

Cross-section data were extracted from a terrain model developed using the site-specific topographic survey where available. In particular, a cross-section was extracted along the centerline of Wabash Street and used as the profile for the crest of the dam at the Wabash Street Bridge and the dam. Approximate (assumed) cross-section data was used to represent the body of the dam impoundment and the right bank of the river downstream from the dam.

5.3.2 Boundary Conditions

Boundary conditions for the hydraulic model included 1) specified inflows at the upstream boundary condition using peak flows presented in **Table 2,** and 2) hydrologic event-specific water surface elevations for a downstream boundary condition based on information presented on the FEMA FIS profile **(Figure 1)**; these water surface elevations (WSELs) are presented in **Table 3**.

Table 3 – FEMA FIS Downstream Water Surface Elevations

Annual Return Probability (%)	FEMA Data				**Dam Safety Event**
	10%	2%	1%	0.2%	**0.5%**
RI (Years)	10	50	100	500	**200**
Downstream WSEL (feet)	684.7	686.5	687.4	689	**688.13**

Information in **Table 3** was plotted that indicated a linear relation between flow and water surface elevations (stage); this linear relation was used to calculate a water surface elevation for use as a downstream boundary condition for modeling of the 0.5 percent return-interval (200-year) peak flow hydrologic event, which is included in the last column in **Table 3**.

5.4 RESULTS OF HYDRAULIC MODEL

The hydraulic model results are consistent with the FEMA FIS profile in that flow over the dam spillway remains free-flowing for all of the evaluated return-interval events (10% to 0.2%). The calculated water surface elevation immediately upstream from the dam for the 0.5% (200-year) return-interval event was 693.3 feet. This elevation is approximately 0.9 feet greater than the low point of Wabash Street (i.e., EL 692.4, located approximately 550 feet to the left (north) of the dam spillway). The total width of the overtopped portion of the road is approximately 420 feet.

Table 3 presents flow and water surface elevation data (WSEL) obtained from the FEMA FIS and FIRM and from the HEC-RAS model analysis. Columns in Table 3 under the heading “FEMA” represent information obtained from the FEMA FIS and FIRM, and columns under the heading “Model” represent values calculated as part of this study, and include calculated values at the upstream of the model reach segment including Milan Dam (Dam) and the model reach segment representing potential overflow of Wabash Street (Overflow). Water surface elevations upstream from the dam are presented for comparison with the FEMA data.

Table 4 – FEMA and Calculated Parameters

Annual Return Probability (%) (RI-*Year*)	FEMA		Model		
				Flow Partition (CFS)	
	Flow (CFS)	WSEL Upstream from Dam (feet)	WSEL Upstream from Dam (feet)	Overflow	Dam
10% (10-*Year*)	2,060	690.2	691.3	0	2060
4% (25-*Year*)	2,800	N/A	692.2	0	2800
2% (50-*Year*)	3,490	692.2	692.9	100	3390
1% (100-*Year*)	4,060	692.8	693.1	426	3634
0.5% (200-*Year*)	**4,600**	**N/A**	**693.3**	**608**	**3992**
0.2% (500-*Year*)	5,190	693.5	693.5	934	4256

Comparison of data presented in Table 4 indicates that the model results overestimate upstream water surface elevations relative to the FEMA data for the 10% (10-year) return-interval event, are relatively close for the 2% and 1% (50 and 100-year) return-interval events, and are the same for the 0.2% (500-year) return-interval event. At lower-magnitude events (e.g., 10% return-interval event), the difference between the FEMA result and the calculated model water-surface elevations upstream of the Dam are more extreme; but for highermagnitude events, the model result converges with that of FEMA. There would need to be a comparison of the original FEMA model with the model used in this report to determine the reason for the difference in results.

The model results indicate that overtopping of Wabash Street occurs at flows marginally below the 2% (50-year) return-interval event, with the depth of overtopping and flow increasing with increasing flow.

The calculated depth of water over Wabash Street during the 0.5% (200-year) return-interval event is approximately 0.9 feet, and the width of the overtopped section of road is approximately 420 feet. The maximum calculated flow speed in the overflow segment during the 0.5% (200-year) return-interval event is approximately 4.4 FPS, and occurs on top of and down-gradient from Wabash Street. The potential for erosion along Wabash Street and down-gradient areas during overtopping events depends on factors including the flow depth and speed, pavement condition, and soil and herbaceous growth conditions. In general, the calculated flow speeds suggest that a small amount of localized erosion would occur during overtopping, but large-scale erosion that could result in head-cutting through the roadway embankment and resulting failure of the embankment does not appear to be likely during the 0.5% (200-year) return-interval event.

5.5 AUXILIARY SPILLWAYS

The dam has a 48-inch drawdown valve in the principal spillway; and two auxiliary spillways to the right of the principal spillway. Flows for these structures were not included in the HEC-RAS model. The tunnels are assumed unreliable to serve as bypass capacity. The effect of the 48-inch drawdown valve was considered separately as part of the alternative to improve embankment overflow (**Section 6.3** below).

The theoretical flows for these structures were calculated assuming they are used to attempt to prevent overtopping of the embankment during the 200-year (0.5%) event. The embankment begins to overtop at elevation 692.4 feet, which corresponds to a flood between the 25-year (4%) and 50-year (2%) events. The following estimates of flow capacity therefore are based on setting the head water at 692.4 feet; and the tailwater at 688.1 feet (the elevation at during the 0.5% event). Flow over the principal spillway was estimated by interpolating between data in Table 4.

Table 5 – Estimated Flow of Auxiliary Structures
(H.W. = 692.4; T.W. = 688.1)

Location	Estimated Flow (CFS)
48-inch Drawdown	210
Left Aux. Spillway	120
Right Aux. Spillway	20
TOTAL	**350**

Note that flow through the Left Auxiliary Spillway is limited by the 36-inch draft tube in the old wheel pit. The Right Auxiliary Spillway is limited by the 24-inch storm pipe connecting the tunnel to the old mill building.

The estimates assume free passage of water through the structures.

Interpolating from **Table 4** above, the flow over the principal spillway at incipient overtopping (WSEL = 692.4 feet) is approximately 2970 CFS. With the flow during the 0.5% flood = 4600, the spillway capacity is therefore 1630 CFS less than required. Including the auxiliary spill capacities from Table 5 above, the system is 1630 – 350 = 1280 CFS under capacity.

6.0 Description of Alternatives

6.1 MINIMAL ACTION

Under this alternative, no improvements would be made to the embankment or spillway. Only required maintenance would be performed.

This alternative results in overtopping of the embankment at floods equal to the 2% (50-year) occurrence or greater. The embankments rely on their existing grass cover for protection from erosion or incision.

As the August 4, 2011 letter from MDNRE (now MDEQ) noted, "(a)dequate spillway capacity is mandated by Part 315" **(Appendix B)**. Without further action, auxiliary spill capacity remains inadequate, but what auxiliary capacity exists would still need to be maintained. At a minimum, the tunnels would have to be cleared of debris and obstructions. Please see **Section 4.2** above for findings of the tunnel dive inspections.

Repairs to erosion of the upstream embankments should be made within the next 3 years as the embankment is part of the regulated structure.

Minor concrete repairs should be made within the next 5 years. At this stage, the concrete does not represent a dam safety issue, but timely repairs will prevent more expensive issues and possible dam safety issues in the future.

6.2 IMPROVE CAPACITY OF EXISTING AUXILIARY SPILLWAYS

This option considers modifying the existing auxiliary spillways in order to handle excess flood flows and prevent overtopping of the embankment. The following scenarios assume HW = 693.4 and TW = 688.1 (ref. **Section 5.5** above).

The maximum theoretical flow of the left auxiliary spillway is approximately 120 CFS. This rate is based on the limit of the 36-inch draft tube in the old wheel pit. If this draft tube were to be removed, and the opening in the floor widened, the channel could be brought up to a theoretical flow capacity of approximately 520 CFS. This flow rate assumes a wide-open channel. In fact, a bar screen would need to be installed at the tunnel entrance, adding to head loss and decreasing the channel capacity. Further, the inlet would have to be modified by installing an adequately sized slide gate. In general, the 520 CFS flow rate results in very high channel velocities, making the channel more susceptible to plugging, either at the bar screen inlet, or elsewhere in the channel if the bar screen were not installed. Also, the high velocity increases the chance of damage to the tunnel due to the high dynamic energy and momentum of the water stream.

The right auxiliary spillway discharges into a 100 foot tunnel, then to approximately 480 feet of 24-inch pipe to the old mill building. For this tunnel, the 24-inch pipe is the flow-limiting factor. Under the head conditions as noted above, the right auxiliary spillway is limited to approximately 20 CFS. The only way to increase this would be to replace the 24-inch pipe with a tunnel the full 480 feet to the old mill. Replacing the 24-inch pipe with a 5-foot tunnel would increase the right auxiliary spillway capacity to approximately 210 CFS.

In total, without modifying the tunnels, the maximum combined capacity of the three auxiliary spillways is limited to approximately 350 CFS. With modification of the left tunnel, the auxiliary capacity theoretically could be increased to 750 CFS. Modifying both the left and right tunnel brings the theoretical auxiliary capacity to 940 CFS, still falling short of the additional 1630 CFS required to pass the 0.5% event without overtopping (ref. **Section 5.5** above).

Even if tunnel capacity could be brought to the required capacity, a viable plan would be needed for keeping the tunnels clean and well maintained. In their present condition, both tunnels, especially the right, require removal blockages caused by silt and debris (please see **Section 4.2 Dive Inspections** above.)

Overall, it is concluded that modifying the right and left auxiliary spillways is inadvisable as a means for substantially increasing flood capacity due to: 1) the extensive modifications required; 2) the high resultant velocity in the left tunnel; 3) the propensity of tunnels to become blocked; and 4) the fact that the proposed improvements would not provide enough capacity to satisfy the 200 year event without overtopping the embankment.

6.3 IMPROVE EMBANKMENT OVERFLOW

According to the hydraulic model of this current study (**Section 5.0** above), embankment overflow begins just below the 50-year event. During the 100-year (1%) and 200-year (0.5%) events, a significant portion of the flow passes over the embankments. Approximately 8% of the total river flow goes over the embankment during a 1% event; for a 0.5% event, the embankment passes approximately 13% of the total river flow.

Previous MDEQ correspondence has noted that the downstream slopes are generally flat and overtopping velocities therefore were expected to be relatively low (See **Appendix B**). The 2008 Dam Safety Report confirmed that a 1% flood in 1968 did overtop the road, but caused little erosion.

Added ground protection is recommended along the embankment to ensure stability adjacent to the crest during overtopping events.

Also noted is the fact that the 0.5% event is higher than the spillway abutment walls, resulting in overtopping directly adjacent to the spillway. The embankment immediately left of the spillway is more susceptible to incision damage due to the relative steep slope on down side river bank.

Spill over water on the right bank generally is expected to move down Neckel Court. This spill over is a concern mostly due to the presence of buildings in this area that might be affected by the flood water.

6.3.1 Description

In order to make the dam safe for a 0.5% event, modifications are required on the spillway and embankments. The proposed plan generally is as follows: 1) raise the spillway abutment walls; 2) add flood walls and/or temporary measures to prevent overtopping near areas where the embankment downslope is high and where buildings are present; and 3) install embankment and shore armoring as needed. Minor repairs to existing concrete should also be included with this alternative. Provided the plan could adequately demonstrate that the overflow spillway safely passes the required flow, the left and right auxiliary spillways could then be closed.

6.3.1.1 Raise Abutment Walls

In order to provide safe passage of the 200-year flood waters, the abutment walls of the principal spillway should be raised sufficiently above the expected flood elevation.

The existing abutment walls are at elevation 693.08 top of concrete. They should be raised to an elevation somewhat above the 200 year flood elevation of 693.3. A top of abutment elevation of approximately 694 is suggested in order to provide minimal free-board.

6.3.1.2 Install Flood Walls

The steepest slope of the embankment's downstream face is just left of the spillway near the bridge. To prevent water from running toward this area, a floodwall should be installed to the left of the abutment. A length of approximately 200 feet left along the embankment is suggested. The wall could be shortened by adding more shore armoring along the down side river bank. The top of the flood wall should match the new abutment elevation (694+/-).

To the right of the spillway, the downstream embankments are very flat, but the presence of municipal buildings and private homes favor preventing overtopping water from moving in this direction. The floodwall therefore should extend also to the right, approximately 200 feet from the abutment wall, also matching the top of concrete elevation of the abutments.

Since the flood walls constrict passage of water over the embankment, their presence potentially would increase the upstream flood elevation. Based on the HEC RAS model, the presence of the walls would cause the 200-year upstream WSEL to rise 0.1 foot – from 693.3 to 693.4. Opening the 48-inch drawdown valve in theory will offset this rise in the flood elevation (bringing it back to 693.3); however, it first has to be verified whether or not the original FEMA study counted the valve in its calculation of the floodplain. In general, implementing this alternative would require a detailed analysis of potential effects on the floodplain, including a thorough review of the original FEMA study.

This alternative requires the City to have in place a workable emergency plan that includes operating the 48-inch valve when flood stage is expected. Though the valve should be opened well before the maximum flood stage is reached, raising the elevation of the operator platform and access walk nevertheless may be required to ensure that the valve can be operated in the dry. The opening elevation of the gate should be included in the operations plan. Also, a program for maintaining and exercising the valve would also have to be in place.

A flood gage should be installed so that flood stages of Ford Lake readily can be ascertained from the vantage point of Wabash Street.

Subject to MDEQ approval, the flood walls possibly could be substituted with temporary measures, such as stop logs that would be inserted between the road railing posts. These measures are less expensive, but would require the City have on file a detailed plan of action for installing the sand bags or stop logs during a flood event. The plan would require the purchase of materials and equipment and should involve periodic trial runs to ensure workability.

In addition, during a 200-year event, temporary sand bagging should be installed around the west, south and east sides of the fire house as the finished floor is essentially at the 200 year flood elevation.

Please see **Appendix F** for drawings showing details of this alternative.

6.3.1.3 Embankment Armoring

In order to improve the ability of the ground to withstand overtopping, grass pavers are recommended for the area of overflow flow. Pavers allow growth of grass to maintain the park appearance, but have strength to prevent erosion due to surface flow. Proposed location of grass pavers and product example can be seen in **Appendix F**.

In addition, armoring should be added to the downstream waterfront where the embankments are relatively steep. The primary strategy is to prevent significant overflow water from reaching this point, but additional armoring will help ensure that the embankments remain intact. Proposed location of shore armoring is shown in **Appendix F**.

6.3.1.4 Concrete Repairs

Repair of spalled concrete on upstream wing walls should be included with the project plan.

6.3.2 Possible construction sequence

1. Mobilize
2. Make video record
3. Install soil erosion counter-measures

4. Draw down impoundment and/or install cofferdams
5. Construct flood walls and abutment modifications
6. Install grass pavers
7. Install shore armoring on downstream embankment
8. Repair erosion on upstream embankment
9. Restoration
10. Demobilize

6.3.3 Environmental

Short term potential environmental impacts are due mostly to construction efforts. Typical soil erosion control and best practices for handling fuels and oils on site should be sufficient to address any concerns in this regard.

The most significant long-term environmental impact is from installation of down-stream shore armoring along the river bank. Exchanging the present vegetated shoreline with rock may make it less desirable as wildlife habitat. This is not a major impact, however, as the affected area is relatively small compared with the remaining river front.

6.3.4 Regulatory

In their preliminary review, MDEQ Dam Safety has stated that they would allow closure of the tunnels provided that the detailed design showed that the spillway arrangement could safely pass the 200 year flow. Their primary concern will be that the project demonstrate that the 0.5% flood can pass the spillway and over the embankment without significant erosion of the embankments. A Part 315 permit is required to conduct this work.

The permit application will also be subject to Part 31 Flood Plain review. MDEQ regulates the 100-year flood elevation and will review on the basis of potential impacts to that elevation, both temporary and permanent.

6.3.5 Capital costs

Capital costs for this option are expected to be approximately $570,000. Please see **Appendix G** for details.

6.4 ADD NEW AUXILIARY SPILLWAY

In order to hold the pond elevation to below the embankment crest, a new auxiliary spillway is added to the left of the existing principal spillway. Both the new auxiliary spillway and existing principal spillway would together pass the entire 200-year flood flow of 4600 CFS.

6.4.1 Description

The principal spillway is limited to a flow of approximately 2800 CFS (see **Table 4** in **Section 5.4**) if the level of Ford Lake is to remain below the embankment crest. Under this option, the new auxiliary spillway therefore would have to pass 1800 CFS to make up the full 4600 CFS flood flow. A 50 foot wide spillway is recommended to accommodate this flow. The spillway would discharge into an approximate 50 x 10-foot culvert to conduct water underneath Wabash Rd. The culvert would discharge into a new bypass channel that connects to the downstream Saline River. The 50 x 10 culvert size potentially could be reduced if detailed design found that, based on scour potential and the cost of shore additional armoring, higher channel velocities were tolerable.

The new auxiliary spillway should be constructed with a v-notch or other multi-stage weir in order to prevent stagnation by allowing continual flow through the conduit and bypass channel.

The project includes significant concrete, excavation and restoration work. Construction would require cofferdams and/or a draw-down of Ford Lake. A section of Wabash Street would need to be removed for installation of the conduit.

This project should include repair erosion on the upstream and repair of minor concrete deterioration on the existing spillway.

6.4.2 Possible Construction Sequence

1. Mobilize
2. Make video record
3. Install soil erosion counter-measures
4. Draw down impoundment and/or install cofferdam.
5. Close east lane of Wabash Rd and walkway
6. Excavate downstream side of Wabash Rd and embankment
7. Excavate and grade new bypass channel
8. Construct 50 x 10 concrete culvert (east half)

9. Backfill culvert and embankment (east half)

10. Reconstruct east lane of Wabash Rd and walkway

11. Close west lane of Wabash Rd and walkway

12. Excavate upstream side of Wabash Rd. and embankment

13. Construct new 50 foot spillway, abutments and headwalls

14. Construct 50 foot x 10 foot culvert (west half)

15. Backfill culvert and embankment (west half)

16. Open west lane of Wabash Rd. and walkway

16. Restore site

17 Demobilize

6.4.3 Environmental

Potential environmental impacts stem mostly from construction. Digging of the bypass channel requires employment of best practices of soil erosion prevention and mitigation.

The bypass channel changes the aquatic environment on the downstream side to the extent that it adds water and would change the flow regime in that area. It would be desirable to have at least a nominal amount of water continuously moving through the bypass channel in order to prevent stagnation. The normal flows east of Wabash therefore would be split between the two channels, reducing the flow on the existing channel to that point.

6.4.4 Permits

This project would require a Part 315 Dam Safety Permit and a Part 31 Floodplain Permit, both issued by MDEQ.

6.4.5 Capital Costs

The cost for this option is approximately $2.7 million. Please see **Appendix G** for details.

6.5 DAM REMOVAL

This option removes the principal spillway, permanently draining Ford Lake and allowing for free passage of flood waters. The majority of the sediment in the lake would remain in place following removal.

6.5.1 Description

Removal of the dam includes demolition of the principal concrete spillway and abandonment of the auxiliary spillways. Since the existing spillway apron provides scour protection of the bridge abutments, this alternative assumes that the most of the apron would be left in place, removing only the concrete baffle blocks as needed to prevent obstruction of the river. The apron could possibly be demolished, however, depending on a detailed study of the bridge abutments and scour potential.

6.5.2 Possible Construction Sequence

1. Mobilize
2. Make video record
3. Install soil erosion counter-measures
4. Draw down impoundment
5. Install additional soil erosion counter-measures along river
6. Demolish 60-foot concrete spillway
7. Remove baffle blocks from spillway apron
8. Bulkhead and fill auxiliary spillway tunnels with light concrete
9. Regrade old lake bottom as needed
10. Install plantings and restoration as per design
11. Demobilize

6.5.3 Environmental

In the short term, this alternative has a high potential impact in that it produces silt and sediment in the river. The design would have to minimize the risk of silts moving downstream. In the long term, the habitat would considerably change due to the loss of the 22 acre Ford Lake and upstream impounded areas. Though the existing silt level is an impediment to the habitat of

Ford Lake, MDNR Fish and Wildlife has suggested that if depth variation were restored by dredging, Ford Lake and the upstream wetlands could be a valuable habitat to the area. Saline River generally lacks “in line” wetlands, and, if restored, this system would be a boon to fish habitat.

Dam removal, on the other hand, improves flood plain performance by adding storage and significantly reducing flood stages. The added storage caused by removal of the dam would benefit both upstream and downstream reaches of the Saline River. In addition, removal of the dam allows free passage of fish to upper reaches from the lower. The existing wooded wetlands upstream of the Ann Arbor Railroad would be altered, but would remain as valuable wetland habitat.

6.5.4 Permits

Permits for this project would include the Part 31 Floodplain. In general, MDEQ favors dam removals and would be amenable to leaving sediments of this type in place, provided that due care of the remaining property was demonstrated.

6.5.5 Capital Costs

This option is estimated to cost approximately $1.3 Million. Please see the **Appendix G** for details.