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Inspection, Analysis, and Recommendations Report

Ford Manchester Dam

River Raisin
Manchester, Michigan
EGLE Dam ID No. 391

Submitted to:
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Submitted by:
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February 14, 2024
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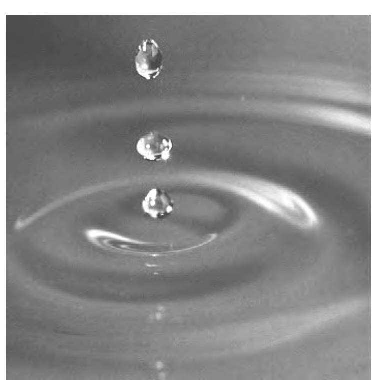

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1. Introduction

The Ford Manchester Dam was constructed on the River Raisin in Manchester, Michigan, in 1940 by the Henry Ford Motor Company to generate hydroelectric power. Since then, the use of hydropower generation has been abandoned. The dam and powerhouse were purchased in 2000 by the Village of Manchester and the powerhouse was reconfigured into the village offices. As of 2004, the dam is regulated and inspected by the Department of Environment, Great Lakes, and Energy (EGLE) Dam ID No. 391 and is rated as a High Hazard Dam. Prior to 2004, the dam was inspected by numerous other companies with the oldest provided inspection report dating back to 1978 prepared for the U.S. Army Corp of Engineers. Historic documents utilized are provided in **Appendix A**. The site location is depicted in **Figure 1-1**.

Figure 1-1: Site Location Aerial View

1.1 Project Description

The dam structures consist of, from left to right[1], a 540-foot-long left earth embankment, an abandoned intake and powerhouse, an 80.5-foot-wide concrete spillway, and a 190-foot-long right earth embankment. The dam structures are depicted in **Figure 1-2**. The spillway consists of a single ogee crest concrete spillway with two 4-foot-diameter sluice gates.

Figure 1-2: Dam Structure Locations

[1] Left and right are referenced looking downstream.

The drawdown sluice gates are situated on the left and right sides of the spillway with a trashrack directly upstream of each. The sluice gates are currently in the closed position and there have been no documented events where the sluice gates would have been operated (raised and/or lowered) to lower and refill the reservoir upstream. Therefore, the condition of the components above water can be observed but the operability is questionable since there are no records of the sluice gate having been operated.

The intake structure is situated directly to the left of the spillway and consists of an 8-foot square concrete penstock with an angled trashrack. The penstock feeds two (2) twin turbines located at the powerhouse. The head gates at the powerhouse are currently in the closed position and there have been no documented events where the head gates would have been operated (raised and/or lowered) in the recent past. Therefore, the condition of the components can be observed above water but the operability is questionable since there are no records of the head gate having been operated.

The dam has a structural height of 26.5 feet and a hydraulic height of 24.6 feet. During normal conditions, the dam has approximately 20 feet of head with 3.5 to 4 feet of freeboard. Under normal flow conditions the impoundment is approximately 45 acres. Normal headwater is approximately elevation (El.) 877.5 feet[2] and a tailwater El. 857.7 feet. The dam has no auxiliary spillway.

The earthen embankments have crest widths of approximately 35 feet and server as the road bead for M-52. A bridge is situated over the river channel directly downstream of the spillway. The upstream and downstream slopes are approximately 3 horizontal to 1 vertical.

Per original construction drawings, the concrete structures making up the dam (including the powerhouse, spillway, walls, and intake) are founded on native hard sand gravel clay and boulder foundation. The concrete structures are supported by a slab-on-grade. The spillway slab on-on-grade has a steel sheet pile (SSP) seepage cutoff wall integral with the slab upstream of the spillway and three (3) seepage drains beneath the downstream spillway slab. The downstream spillway slab has a wier approximately 15 feet downstream of the M-52 bridge.

The walls consist of a combination of earth retaining walls, bridge abutment walls, powerhouse superstructure support, intake, and draft bay walls. The upstream walls consist of a left wing wall that abuts the penstock intake and a right wing wall that abuts the spillway. Between the spillway and the M-52 bridge there is a left retaining wall that also functions as the right side of the penstock and a right retaining wall that support the upstream side of the right embankment. Downstream of the spillway the M-52 bridge is supported on left and right abutments walls that also act as retaining walls for the left and right embankments. In addition, the left abutment wall supports the right side of the penstock.

[2] Elevations are in reference to the USGS datum.

Downstream of the M-52 bridge there are left and right retaining walls. The left retaining wall supports the downstream side of the left embankment (grassy area in front of the powerhouse) and the right side of the penstock. The right retaining wall supports the downstream side of the right embankment. At the left downstream retaining wall abutment at the powerhouse there are two (2) vault areas that are divided by the head gates. The operating equipment for the headgates is located within the vaults. At the left retaining wall abutment to the powerhouse, the wall transitions into a structural wall that supports the superstructure of the powerhouse, the powerhouse intake, including the turbine and the draft bay. To the left of the draft bay section of the structural wall is a basement wall that supports the superstructure of the powerhouse. Downstream of the powerhouse the basement wall transitions into a retaining wall that supports an outdoor seating area for the village staff.

A steel trashrack is present on the upstream end of the intake, which is supported by a sill plate at the bottom and a steel channel at the top. Stop log slots are present immediately upstream of the intake trashrack and approximately 40 feet downstream from the head gates. The upstream stop log slots are formed in the concrete intake structure and the downstream stop log slots are formed in the concrete tailrace piers. Stop logs do not currently exist for the upstream or downstream slots.

1.2 Background and Purpose

EGLE preformed a dam inspection on May 17, 2022, to evaluate the structural condition and hydraulic capacity of the dam. Based on the visual inspection, the dam was rated in fair condition and the high hazard status remained appropriate. EGLE provided the following recommended actions in order of priority:

1. Complete a detailed structural evaluation of the principal spillway and powerhouse structures, including a plan and schedule for any necessary repairs, within the next year. This had previously been recommended in each inspection since 2013 to be completed by 2023.

2. Continue efforts to remove all trees and brush from the earthen embankments. After clearing, mow and/or treat the entire embankment a minimum of two times per year to prevent further establishment of woody vegetation and facilitate visual inspection. When cleared, the embankment should have proper, non-woody vegetative cover established and maintained. Trees and brush to be removed were observed at the upstream slopes of both embankments and on the downstream slope of the right embankment where woody vegetation is encroaching on to the embankment past the groin and downstream toe. The embankment should be cleared to 10 feet beyond the groin of the embankment and to 10 feet beyond the downstream toe.

3. Fill the animal burrow at the left end of the spillway deck. Monitor the site for additional burrows and fill them as observed.

4. Restore at least one of the drawdown gates to an operational condition or establish other methods of impoundment drawdown should it become necessary.

5. Monitor the storm sewer outfall on the downstream slope of the right embankment for further erosion. If erosion progresses, further armor the flow path.

6. Develop a written Operation and Maintenance Plan (O&M Plan) for the dam and submit a copy to the Dam Safety Program.

7. Provide the updated Emergency Action Plan to the Dam Safety Unit by December 31, 2022.

GEI was contracted by the Village of Manchester to address action items 1, 4, and 6 listed above. The purpose of this report is to summarize the results of the field inspection and data review, present and evaluate viable repair options, and to provide conclusions, recommendations, and a preliminary Engineer's Option of Probable Construction Cost for recommended repairs.

2. Field Observations and Findings

To assess the current conditions at the site, GEI reviewed available reference information, including the condition assessments performed by EGLE (2022) and original design drawings. In addition, Ms. Morgan Carden, P.E., from GEI conducted a site visit on September 19-20, 2023, along with a subcontracted dive inspection team from J.F. Brennan Company (JFB). Photos from the GEI site visit are provided in **Appendix B**. Findings from the dive inspections are included in the inspection report prepared by JFB in **Appendix C**. The following sections summarize the current condition of the various structures at the site as observed by GEI and JFB.

2.1 Summary of Field Inspection Findings, Conclusions, and Recommendations

In general, the field inspection found the Ford Manchester Dam to be in fair condition. The following items were identified and considered noteworthy during the inspection:

1. Spalling of concrete at the top of the majority of the walls at the site, with some areas extending down the wall faces and deep enough to expose rebar and fall protection imbeds.
2. Spalling and delamination on face of downstream walls.
3. Spalling of concrete at the freeze thaw line at the upstream and downstream pier bullnoses.
4. Spalling of the concrete along the edges of the operator deck and intake deck with areas deep enough to expose rebar and fall protection embedments.
5. Leaking through construction joints in penstock and left wall face upstream of powerhouse.
6. Hairline cracking inside penstock.
7. Heavy marine growth and rusting of all trashracks.
8. Heavy rust and delamination inside draft tubes.
9. Unknown operability of all gates, drains, and turbines.

Based on the GEI field inspection findings plus prior EGLE recommendations and conclusions, we recommend the following corrective measures be implemented in the timeframes as noted:

1. Repair deteriorated concrete within the next 4 years.
2. Replace one (1) drawdown sluice gate including trashrack within the next 4 years.
3. Fill the intake chamber within the next 4 years.
4. Vegetation removal and animal burrow infill ongoing maintenance item.
5. Runoff and erosion on downstream right embankment ongoing maintenance item.
6. O&M manual provided in **Appendix G**.
7. Update EAP and provide to Dam Safety as soon as possible.
8. Continue inspections per cycle required by EGLE.

2.2 Upstream Wing Walls

The upstream wing walls consist of left and right wing walls as shown as red lines in **Figure 2-1** and **Figure 2-2**. The left wing wall is fully submerged and the right wing wall is partially submerged.

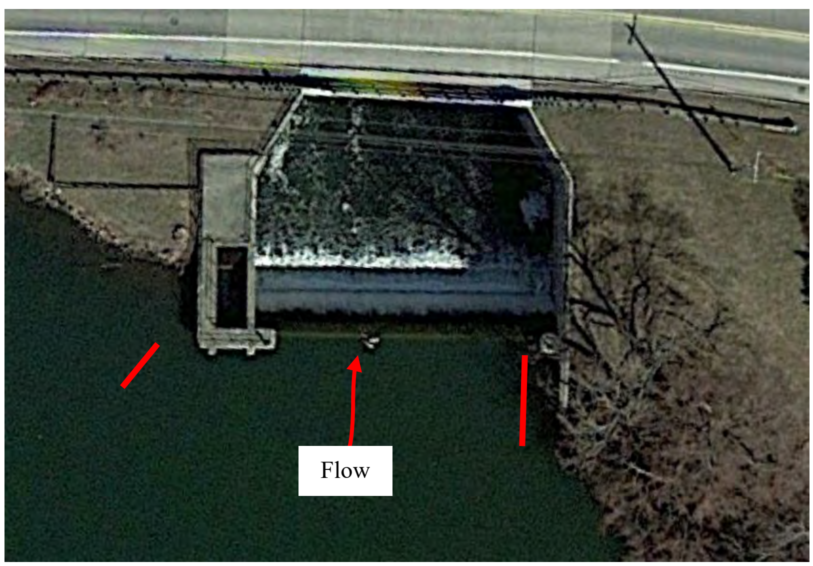
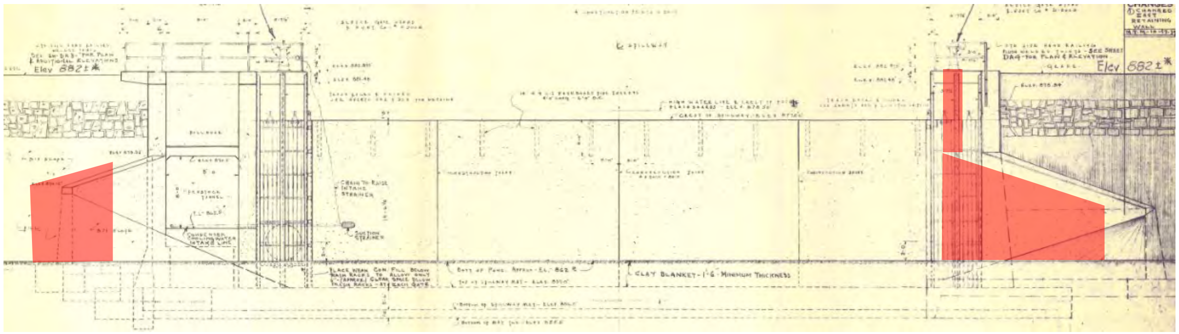


Figure 2-1: Upstream Wing Walls - Plan



Left wing wall **Right wing wall**

Figure 2-2: Upstream Wing Walls - Elevation Looking Downstream

<u>Upstream Left Wing Wall</u>

JFB observed the condition of the face and foundation of the upstream left wing wall outlined in red in **Figure 2-1** and highlighted in red in **Figure 2-2** below headwater elevation. At the time of observation, there was light scaling on the face of the wall with no significant deterioration noted and no apparent undermining of the wall.

<u>Upstream Right Wing Wall</u>

GEI observed the condition of the top of the exposed upstream right wing wall outlined in red in **Figure 2-1** above. The top of wall concrete had spalling in localized areas with some areas extending down the face of the wall up to 18 inches. See Photos 14 through 16 in **Appendix B**.

JFB observed the condition of the face and foundation of the upstream right wing wall outlined in red in **Figure 2-1** and highlighted in red in **Figure 2-2** below headwater elevation. At the time of observation, there was light scaling on the face of the wall with no significant deterioration noted and no apparent undermining of the wall.

2.3 Spillway and Sluice Gates

The spillway consists of a concrete gravity structure with left and right sluice gates as shown in **Figure 2-3**. The sluice gates each have an operator deck and a vertical trashrack as shown in **Figure 2-5**. Per original drawings, the upstream spillway slab has a steel sheet pile cutoff wall below, cast into the reinforced concrete slab as shown in orange in **Figure 2-4**.

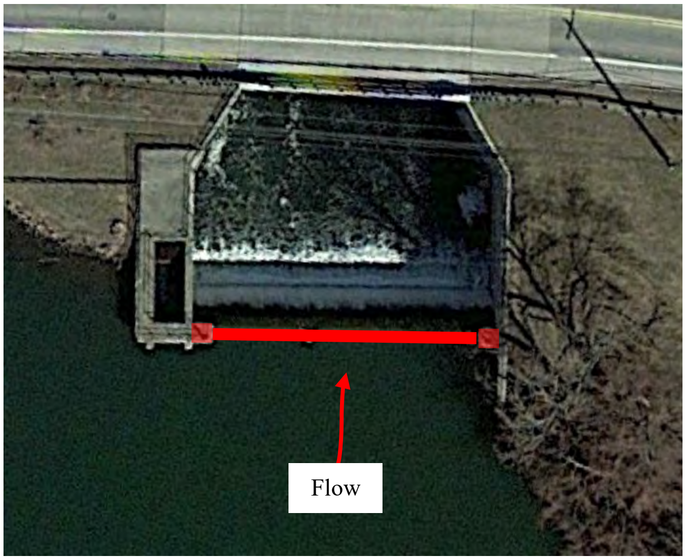


Figure 2-3: Spillway and Gates - Plan

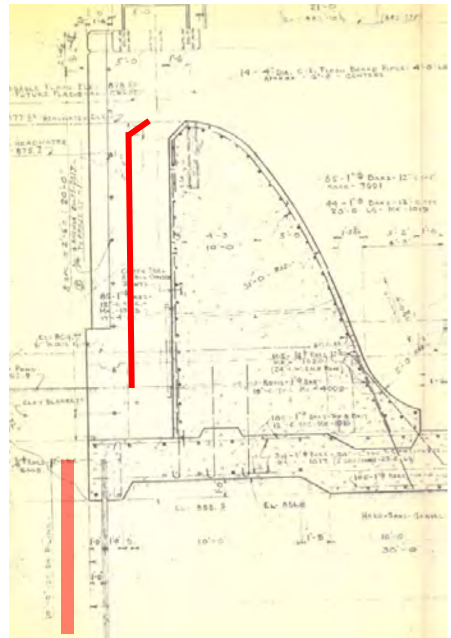


Figure 2-4: Spillway - Section Looking Left

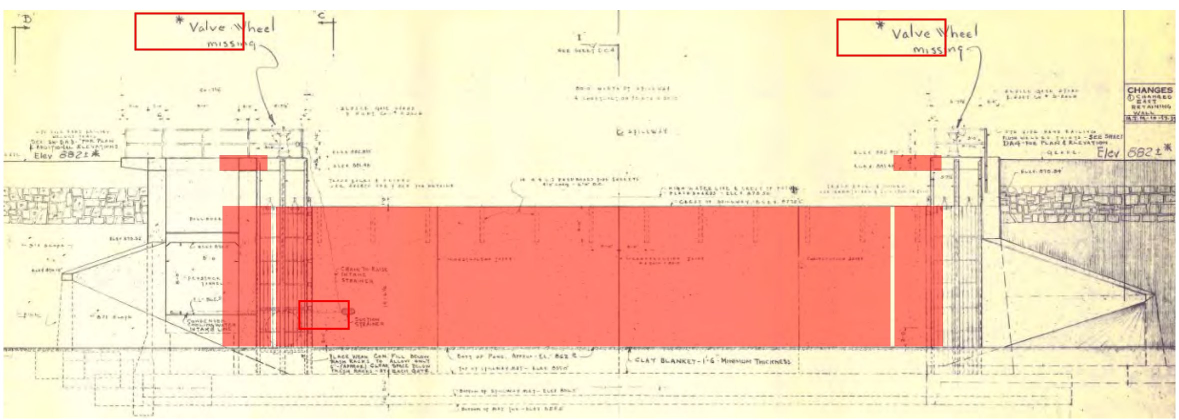


Figure 2-5: Spillway and Gates - Elevation Looking Downstream

<u>Upstream Spillway Face</u>

JFB visually observed the condition of the upstream face of the concrete gravity dam structure. At the time of observation, there was light scaling on the upstream face of the spillway with no significant deterioration noted and no apparent undermining of the gravity structure. In addition, moderate marine growth was noted across the face. The sheet piles as depicted in orange in **Figure 2.4** above were not able to be confirmed as they are embedded in the upstream spillway slab and soil below. The suction strainer depicted in **Figure 2-5** above was observed during inspection; however, the chain was not found.

Left Sluice Gate

GEI visually observed the condition of the left sluice gate operator deck. The operator deck concrete was in poor condition with several inches of spalling all on all three sides, exposed rebar and fall protection embeds, and a missing valve wheel. See Photos 17 through 19 in **Appendix B**.

JFB visually observed the condition of the sluice gate and trashrack. During the observations, the gate appeared to be completely closed with little to no flow present. The operability of the gate is unknown. The trashrack had an abundance of marine growth and rust present. See Figure 33 in **Appendix C**.

Right Sluice Gate

GEI visually observed the condition of the right sluice gate operator deck. The operator deck concrete was in poor condition with several inches of spalling all on all three sides, exposed rebar and fall protection embeds, and a missing valve wheel. See Photos 20 through 23 in **Appendix B**.

JFB visually observed the condition of the sluice gate and trashrack. During the observations, the gate appeared to be completely closed with little to no flow present. The operability of the gate is unknown. The trashrack had an abundance of marine growth and rust present. See Figure 32 in **Appendix C**.

2.4 Penstock Intake and Penstock

The penstock intake consists of a concrete box structure with an approximately 8-foot-by-25-foot opening on top into the intake as shown in **Figure 2-6**. The opening to the intake is 8-foot square on the upstream face of the intake structure with a bullnose situated on either side of the opening as shown in **Figure 2-7**. The intake vault area contains left and right stop log slots and a trashrack at the opening to the 8-foot square concrete penstock as shown in **Figure 2-8**. The profile of the penstock intake and penstock are depicted in **Figure 2-9**.

Figure 2-6: Penstock Intake and Penstock - Plan

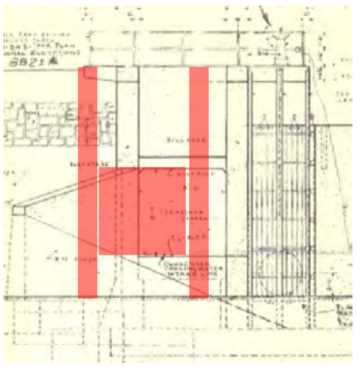


Figure 2-7: Penstock Intake - End Looking Downstream

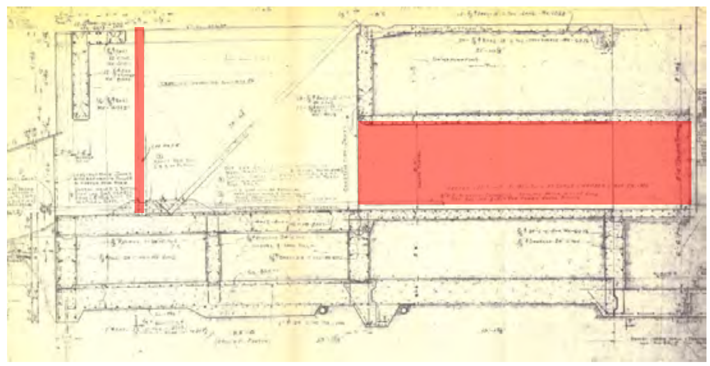


Figure 2-8: Penstock Intake and Penstock - Elevation Looking Left

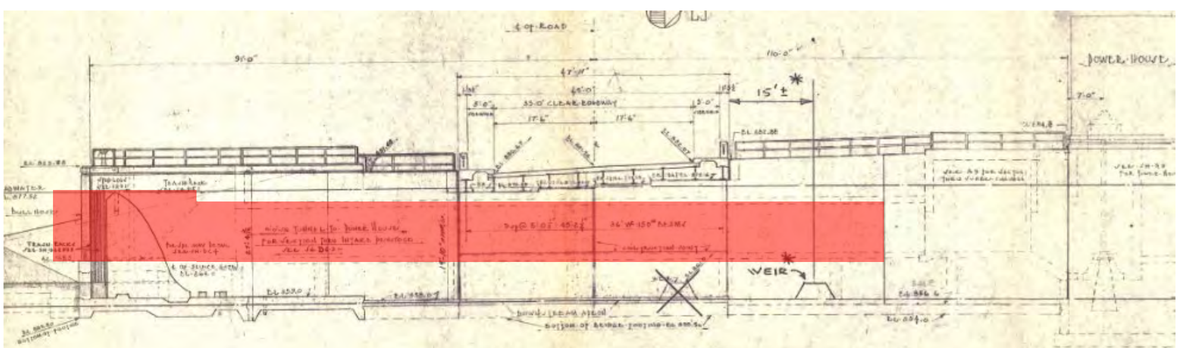


Figure 2-9: Penstock Intake and Penstock - Profile Looking Left

Penstock Intake

Concrete Deck

GEI observed the condition of the deck above the penstock intake as highlighted in red in **Figure 2-6** above. The top face of the concrete deck appears to have hair line cracking. The edges and exposed sides on the concrete deck show signs of distress in the form of spalling for the majority of the deck perimeter. See Photos 24 through 36 in **Appendix B**.

Exterior of Structure

JFB observed the condition of the face and foundation of the exterior walls of the penstock intake as outlined in red in **Figure 2-6** and the condition of the bullnoses highlighted in red in **Figure 2-7** above. The face of the exterior walls (left, upstream and right) of the penstock intake do not appear to have any signs of distress observed. Both bullnoses on the upstream face of the penstock intake have significant spalling at the freeze thaw line. In addition, the upstream left bullnose has an area of scaling below the headwater elevation. See Photos 24 through 36 in **Appendix B** and Figures 34, 36, and 37 in **Appendix C**.

Interior of Structure

GEI observed the condition of the face of the interior walls highlighted in red in **Figure 2-9** and stoplogs slots and trashrack of the penstock intake as outlined in red in **Figure 2-8** above the headwater elevation. The interior intake walls above the headwater elevation had minor hair line cracking and signs of efflorescence. The trashrack above the headwater elevation appeared to have minor rusting and localized marine growth along the water line. The stoplog slots above the headwater elevation had signs of rusting. No stored stoplogs were present at the site. See Photos 37 through 40 in **Appendix B**.

JFB observed the condition of the face of the interior walls highlighted in red in **Figure 2-9** and stoplogs slots and trashrack of the penstock intake as outlined in red in **Figure 2-8** below the headwater elevation. The face of the interior penstock intakes walls did not appear to have any signs of distress observed below the headwater elevation. The left and right stoplog groves appeared to be in satisfactory condition below the headwater elevation with large amounts of debris present at the bottom of the left grove. The trashrack had approximately 80-90% marine growth coverage and the bars appeared to be moderately to heavily rusted throughout the trashrack below the headwater elevation. The trashrack sill was not able to be observed due to the presence of approximately 3 feet of sediment and debris. See Figure 35 in **Appendix C**.

Penstock

JFB observed the condition of the interior of the penstock utilizing an ROV as highlighted in red on **Figures 2-6**, **2-8**, and **2-9** above. The bottom of the penstock tunnel was not able to be observed due the presence of sediment. The upstream end of the penstock consists of the intake structure with trashrack as described above. The left interior wall appeared to be in satisfactory condition.

The two (2) gates located upstream of the powerhouse and downstream of the trashrack were closed with little to no flow present. The gates show signs light to moderate rust throughout. The last time the gates were operated is unknown. See Photos 41 and 42 in **Appendix B** and Figures 20 and 21 in **Appendix C**.

The right interior wall has three (3) notable areas of distress, see Figures 22, 23, 26, and 27 in **Appendix C**, which consist of the following:

- 15 feet upstream of gates – construction joint leaking (additional details provided in **Section 2.6**).
- 27 feet upstream of gates – hairline cracking was present from bottom to top of wall.
- 40 feet upstream of gates – hairline cracking was present from bottom to top of wall.

The ceiling of the penstock did not appear to show signs of distress with the exception of spalling noted at the construction joint 15 feet upstream of the gates. See Figures 24 and 25 in **Appendix C**.

2.5 Downstream Spillway Slab

The downstream spillway slab consists of a reinforced concrete slab with an approximate thickness of 2 feet 6 inches. The extent of the slab is depicted in **Figure 2-10**. The downstream concrete slab has three (3) seepage drains beneath the slab as shown in **Figure 2-11** and a concrete weir as shown in **Figure 2-12**.

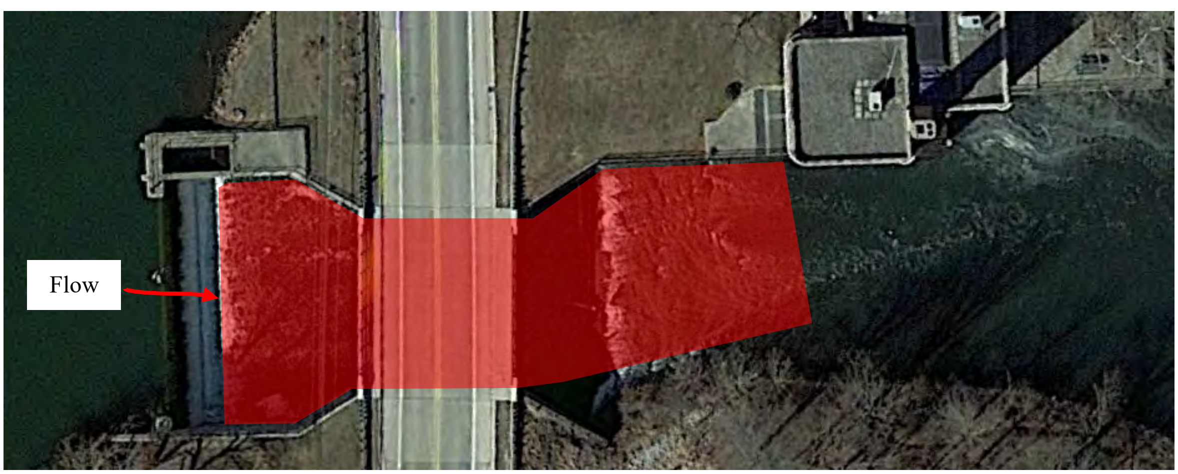


Figure 2-10: Downstream Spillway Slab - Plan

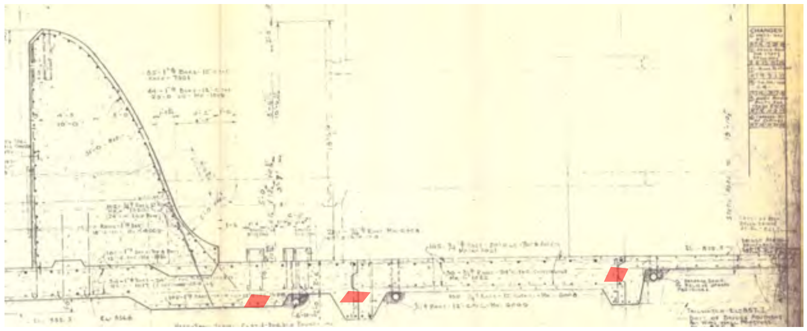


Figure 2-11: Downstream Spillway Slab, 3 seepage drains - Section Looking Left

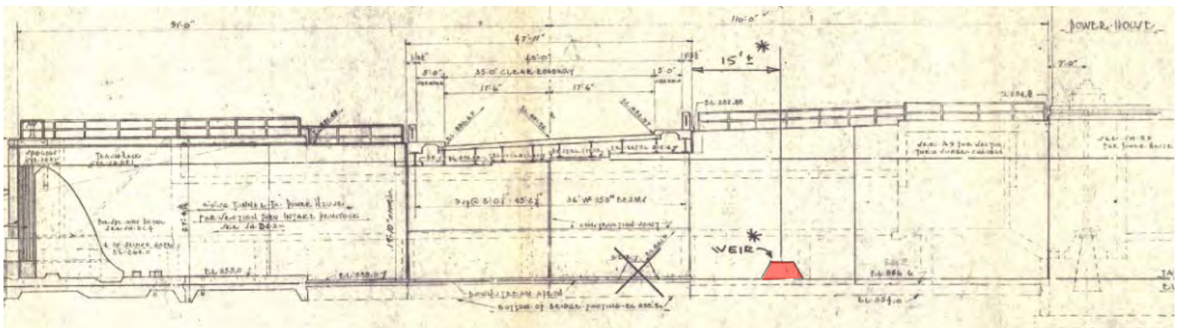


Figure 2-12: Downstream Spillway Slab highlighting weir location - Elevation Looking Left

JFB observed the condition of the downstream left spillway slab as shown in **Figure 2-10** above. An area of undermining was noted at the downstream face of the concrete gravity spillway structure. However, the concrete gravity structure is founded on the spillway slab. Therefore, it appears that there may be separation of the concrete gravity structure and the spillway slab or an area of deteriorated concrete. This location was only observed by feel and would need to be dewatered for further inspection. The weir structure was located during the observations as depicted in **Figure 2-12** above. The drains shown in **Figure 2-11** above were not found during the inspection; however, the slab was not able to be visually inspected due to the amount of water present during the dive observations.

2.6 Downstream Left Walls

The downstream left wall consists of a retaining wall upstream of the M-52 bridge, an abutment wall at the bridge, and a retaining wall downstream of the bridge as shown in **Figure 2-13** and **Figure 2-14**. The left downstream wall forms the right-hand side of the concrete penstock, retains the left embankment, and supports the bridge at the abutment section of the wall.

Figure 2-13: Downtown Left Wall - Plan

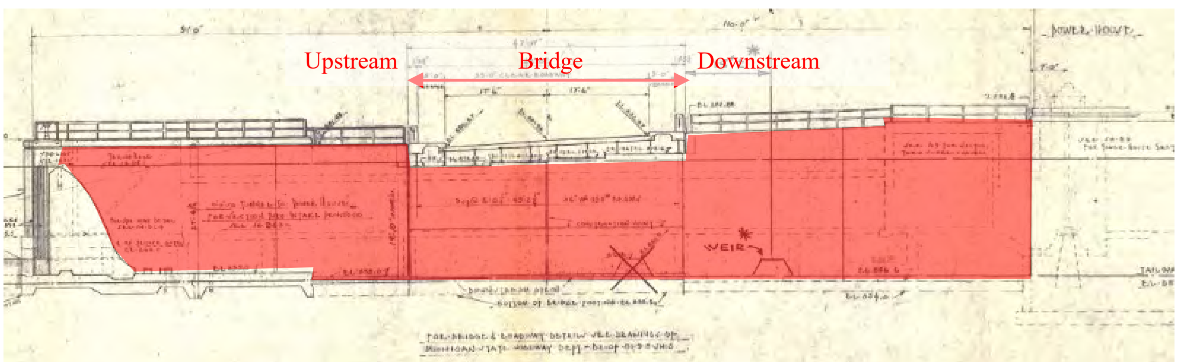


Figure 2-14: Downstream Left Wall - Elevation Looking Left

<u>Wall Upstream of Bridge</u>

GEI observed the condition of the top of the downstream left wall located upstream of the M-52 bridge outlined in red in **Figure 2-13** above. The top of wall concrete had spalling along approximately 30 feet of wall length upstream of the construction joint near the intake location, with some areas extending down the face of the wall up to 18 inches and several inches deep exposing rebar reinforcement. In addition, there appeared to be separation between the concrete wall the bridge abutment parapet wall. See Photos 43 through 46 in **Appendix B**.

JFB observed the condition of the face and foundation of the downstream left wall located upstream of the M-52 bridge as outlined in red in **Figure 2-13** and highlighted in red in **Figure 2-14** above. The wall did not appear to have any undermining. However, spalling was noted on the face of the wall above the tailrace water elevation at two (2) locations, with one (1) location having exposed rebar and vegetation growth. See Photo 47 in **Appendix B** and Figures 9 through 11 in **Appendix C**.

Bridge Abutment Wall

JFB observed the condition of the face and foundation of the left abutment wall as outlined in red in **Figure 2-13** and highlighted in red in **Figure 2-14** above. The abutment wall did not appear to have any undermining or visible concrete deterioration.

Wall Downstream of Bridge

GEI observed the condition of the top of the downstream left wall located downstream of the M-52 bridge outlined in red in **Figure 2-13** above. The top of wall concrete had localized areas of spalling along approximately 55 feet of wall, with some areas extending down the face of the wall up to 18 inches and several inches deep exposing rebar reinforcement. See Photos 48 and 51 through 63 in **Appendix B**.

JFB observed the condition of the face and foundation of the downstream left wall located downstream of the M-52 bridge as outlined in red in **Figure 2-13** and highlighted in red in **Figure 2-14** above. The wall did not appear to have any undermining. However, spalling with exposed reinforcing steel was noted on the face of the wall at the construction joint above the tailrace water elevation. In addition, to spalling water was present seeping through the construction joint from the inside of the penstock. See Photos 49 and 50 in **Appendix B** and Figures 12 through 14 in **Appendix C**.

2.7 Downstream Right Walls

The downstream right wall consists of a retaining wall upstream of the M-52 bridge, an abutment wall at the bridge and a retaining wall downstream of the bridge as shown in **Figure 2-15**. In general, the downstream right wall appears to be in good condition.

Figure 2-15: Downstream Right Wall - Plan

Wall Upstream of Bridge

GEI observed the condition of the top of the downstream right wall located upstream of the M-52 bridge outlined in red in **Figure 2-15** above. The top of wall concrete had spalling along approximately 45 feet of wall length upstream of the construction joint, with some areas extending up to 18 inches down the face of the wall, several inches deep, exposing steel reinforcement. Spalling was also observed on the face of the wall located upstream of the construction joint. See Photos 64 through 73 in **Appendix B**.

Also of note, GEI observed that the condition of the retained soil directly behind the wall to be noticeably softer than the rest of the upstream right embankment soils. There appears to be no drainage features along the back of the wall to facilitate surface water runoff away from the wall.

JFB observed the condition of the foundation and face of the downstream right wall located upstream of the M-52 bridge outlined in red in **Figure 2-15** above. The wall did not appear to have any undermining. However, spalling and delamination were noted on the face of the wall above the tailrace water elevation. In addition, efflorescence was present along the entire face of the wall above the tailwater elevation. See Photos 74 and 75 in **Appendix B** and Figures 3, 5, and 6 in **Appendix C**.

Bridge Abutment Wall

JFB observed the condition of the foundation and face of the right abutment wall outlined in red in **Figure 2-15** above. The abutment wall did not appear to have any undermining or visible concrete deterioration. See Photo 76 in **Appendix B** and Figure 4 in **Appendix C**.

Wall Downstream of Bridge

GEI observed the condition of the top of concrete wall and JFB observed the condition of the face and foundation of wall the downstream right wing wall downstream of the M-52 bridge outlined in red in **Figure 2-15** above. The wing wall did not appear to have any undermining or visible concrete deterioration. See Photos 76 and 77 in **Appendix B** and Figure 4 in **Appendix C**.

2.8 Powerhouse Walls and Draft Bay

The powerhouse has two (2) turbines located on the upstream side of the powerhouse that are accessible from the concrete slab upstream of the powerhouse and the draft bay. The powerhouse and draft bay are comprised of several walls that serve as support for the powerhouse superstructure and form the two (2) draft bays. The configuration of the walls is outlined in red and the draft bay is highlighted in red in **Figure 2-16** below. The exterior walls are highlighted in red in **Figure 2-17** and the interior of the turbine vault and draft bay are highlighted in red in **Figure 2-18** below.

Figure 2-16: Powerhouse Wall and Draft Bay - Plan

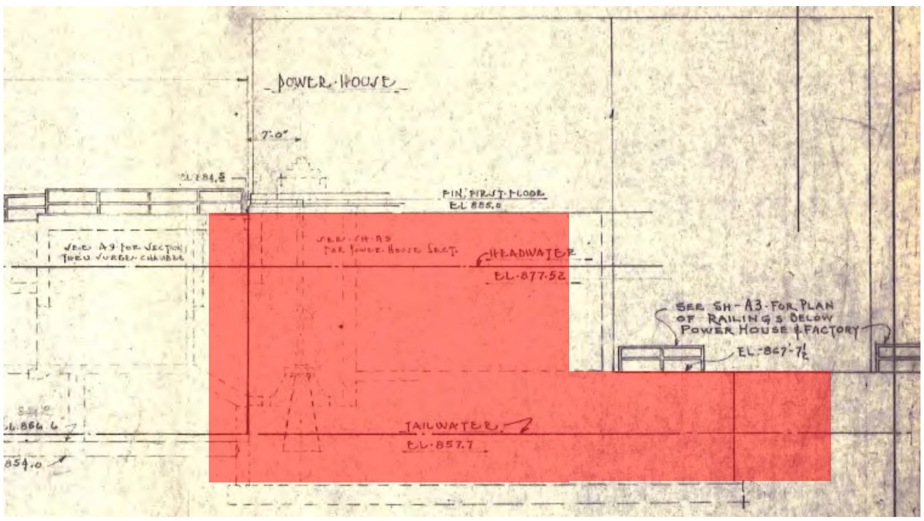


Figure 2-17: Powerhouse Structure Wall - Elevation Looking Left

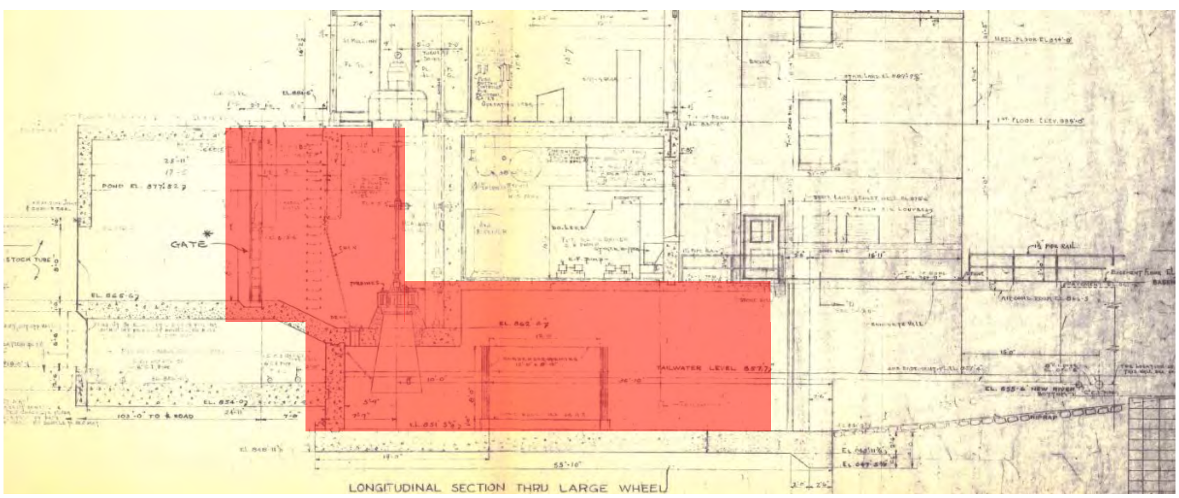


Figure 2-18: Powerhouse Basement Wall and Draft Bay - Section Looking Left

Right Exterior Wall

JFB observed the condition of the face and foundation of the right exterior wall. At the time of observation, there was light scaling on the face of the wall with no significant deterioration noted and no apparent undermining of the wall. See Photo 78 in **Appendix B**.

Turbine Vault

JFB observed the condition of the interior of the turbine vaults utilizing an ROV. The interior of the vaults appeared to be in satisfactory condition. There was notable sediment observed at the bottom of the vaults. The ladder attached to the left wall of the vault was heavily rusted. The chain to a drain appeared to be present; however, the drain was not observed due to the presence of sediment on the bottom of the vault. The turbines appeared to be in satisfactory condition; however, the last time they were operated is unknown. See Photos 79 and 80 in **Appendix B** and Figures 28 and 29 in **Appendix C**.

Draft Tubes

JFB observed the interior of steel draft tubes in the left and right bay. Both draft tubes appeared to be heavily rusted with light delamination. The turbine gates were closed at the time of the inspection with an air gap between the tailwater elevation and the gate. See Figures 30 and 31 in **Appendix C**.

Draft Bay Walls

JFB observed the condition of the left and right draft bay walls. At the time of observation, there was light scaling on the face of the walls with no significant deterioration noted and no apparent undermining of the walls.

Basement Wall and Condensing Unit

GEI observed the interior of the basement wall that forms the left draft bay wall. The wall did not appear to show any major signs of distress. Several pipes were observed to penetrate the wall into the draft bay along with a pit with standing water. The system piping into the draft bay appears to be decommissioned. See Photos 83 and 84 in **Appendix B**.

JFB confirmed the presence of piping penetrating the left draft bay wall during their observations.

Draft Bay Split Wall

JFB observed the condition of the draft bay split wall. At the time of observation, there was light scaling on both sides of the face of the wall with no significant deterioration noted and no apparent undermining of the wall. See Photo 82 in **Appendix B**.

Draft Bay Outlet

GEI observed the condition of the bullnoses above the tailwater elevation. The left bullnose has scaling present and has spalling at the freeze thaw line. See Photo 81 in **Appendix B**.

JFB observed the condition of the bullnoses and stop log slots for the left and right draft bay outlets. The bullnoses appeared to be in satisfactory condition with light scaling present. The left and right stoplog grooves in both bays appeared to be in satisfactory condition with broken timbers present at the bottom of both bays. See Photo 82 in **Appendix B**.

2.9 Downstream Seating Area Retaining Wall

The downstream seating area retaining wall is situated downstream of the powerhouse and supports an outdoor seating area for the village as shown in **Figure 2-19** and **Figure 2-20** below. In general, the downstream seating wall appeared in good condition.

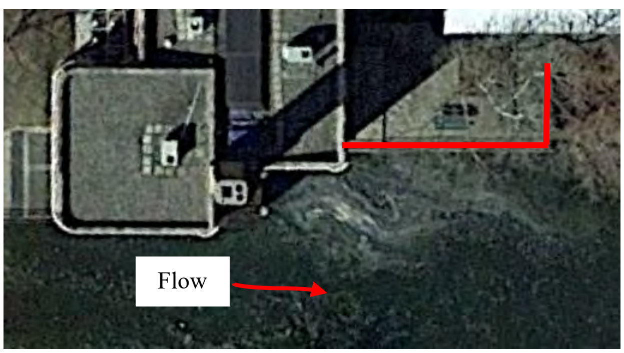


Figure 2-19: Seat Area Wall - Plan

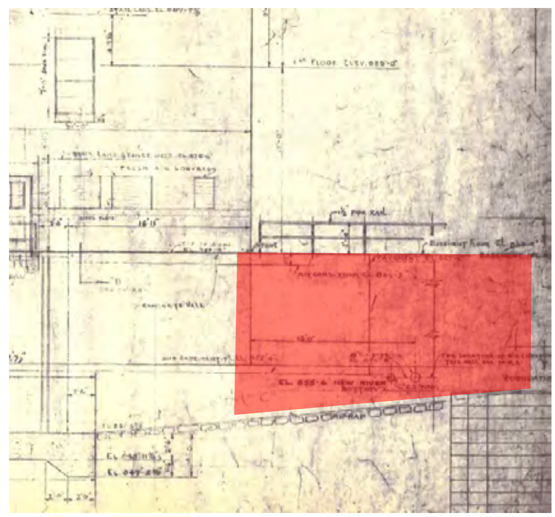


Figure 2-20: Seating Area Wall - Elevation Looking Left

GEI observed the condition of the top of concrete wall outlined in red in **Figure 2-19** above and the slab on grade supported by the wall. The top of wall concrete has pitting and localized areas of spalling, with some areas extending down the face of the wall up to 18 inches and several inches deep exposing rebar reinforcement. The areas appeared to correspond with the locations of the fence posts embedded into the top of wall. These deficiencies are not considered a dam safety issue. However, the deficiencies are a building code safety issue regarding the requirement to withstand 50 pounds horizontally per linear foot of guardrail and potential failure of the guardrail post anchorage under this load. The concrete slab directly adjacent to the top of the wall appeared in good condition with no apparent signs of distress. See Photos 85 to 91 in **Appendix B**.

JFB observed the condition of the face and foundation of walls outlined in red in **Figure 2-19** and highlighted in red in **Figure 2-20** above. At the time of observation, there was light scaling on the face of the wall with no significant deterioration noted and no apparent undermining of the wall. However, there was a significant amount of debris along the toe of the wall limiting the ability to inspect the face of wall below the tailwater. In addition, there were several penetrations along the face of the wall as shown in **Figure 2-20** above.

3. Stability Analysis

GEI performed a stability analysis of the structures as requested by the client and as recommended by EGLE. Spillway stability was analyzed as a conventional global stability analysis based on a rigid, two-dimensional gravity section with loads taken across a 1-foot unit width. Sliding stability was analyzed using the shear friction factor (SFF) of safety method, assuming zero cohesion at the concrete / foundation interface, in general accordance with EM1110-2-2502 Retaining and Flood Wall Engineering and Design (Ref. USACE, 1989). No prior global stability analyses were provided for the spillway structure.

3.1 Loading Conditions

Stability was analyzed under the following load cases:

Table 1: Analyzed Load Cases

Load Case	Headwater El. (ft, NGVD)	Tailwater El. (ft, NGVD)
Case I – Normal Operating Conditions, Headwater at Overflow Spillway Crest	877.5	861.0 (1)
Case IIA – Unusual Operating Conditions, Headwater at Overflow Spillway Crest + Ice	877.5 (2)	861.0 (1)
Case II – Unusual Operating Conditions, Flood Discharge Conditions (200-Year Flood)	880.2 (3)	863.0 (4)

Notes:
(1) Assumes equal to top of weir above slab 3. Project drawings indicate tailwater El. 857.7 near the powerhouse (Ref. Ford 1939)
(2) No lowering of reservoir during winter months.
(3) High hazard project. Headwater based on calculations from EGLE inspection report (Ref. EGLE, 2022).
(4) Flood tailwater value selected based on 100-year and 500-year flood levels established in the Flood Insurance Study for River Raisin (Ref. FEMA, 2012).

The project site is located in Zone 1 of the Seismic Zone Map. An earthquake analysis is not required for structures in Zone 1 unless site studies indicate the presence of capable faults or recent earthquake epicenters lie near enough to the dam to cause damage. There are no known capable faults in the vicinity of the dam; therefore, no seismic analysis is required.

3.2 Assumptions

The following assumptions were made during the analysis:

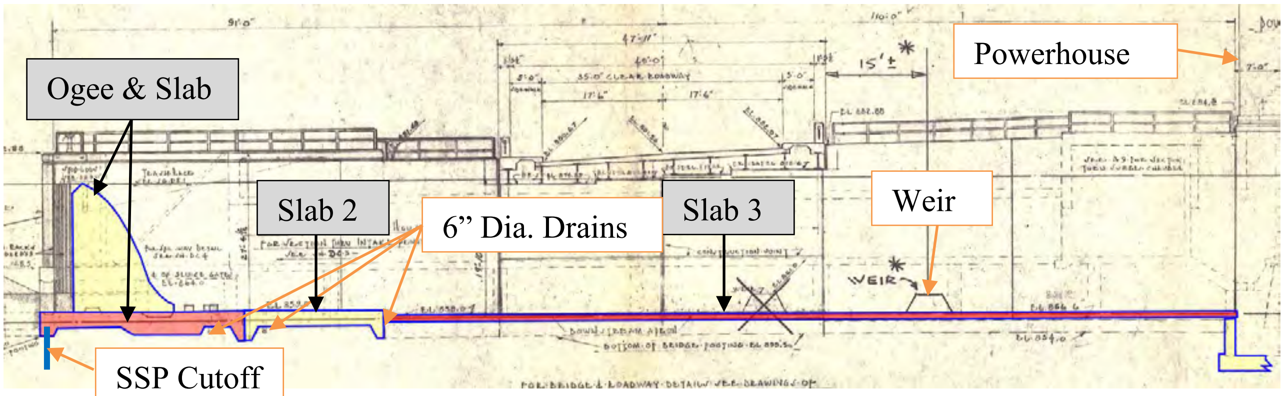


Figure 3-1: Section View of Spillway Structure

- The reinforced concrete ogee, slab 1, slab 2, and slab 3 are included in the sliding stability analysis of the structure. Only the concrete ogee and slab 1 are included to establish base resultant locations and base pressures. Any resistance to sliding provided by the upstream SSP cutoff or the adjacent abutment walls were ignored in this analysis.
- The assumed sliding plane is along El. 855.9 at the horizontal concrete to soil interface below the ogee concrete. The plane at El. 855.9 is a weighted average of the keyed foundation interface elevations of slab 1. Project drawings indicate the structure is founded on a Hard-Sand-Gravel-Clay & Boulder foundation (Ref. Ford, 1939). No soil boring logs were available for review; however, the region has been repeatedly glaciated and the structure lies upon a glacial outwash plain. Soils in the area are a product of weathering and decomposition of the glacial deposits. The soils are of the gray-brown podzolic group and are generally well drained sands and loamy sands (Ref. USACE, 1978). The spillway slabs are keyed into the foundation soil which would mobilize the full internal friction angle of the soil. A sliding plane interface friction angle of 37 degrees was assumed with zero cohesion.
- GEI used dimensions provided on project drawings (Ref. Ford, 1939) to develop model geometry for the computation of structure weights. Project drawings are provided in **Appendix A**.
- Headwater weight and clay soil weight were included upstream of the of the overflow spillway. Weight of tailwater above the slabs downstream of the spillway ogee was included in the analysis. Under flood conditions, the headwater and tailwater weights were maintained equal to the normal operation conditions.

- The modeled uplift beneath the structure was based on the weighted creep method including the length of the upstream SSP cutoff and assuming the gradient equals tailwater beneath slab 3. The overflow spillway section contains three 6-inch-diameter drains downstream of the spillway ogee. The drains outlet through the downstream slabs into tailwater. The analysis ignores presence of the two upstream drains but assumes the third drain is equal to tailwater. The structure contains no instrumentation measuring uplift beneath the structure. The hydrostatic pressure below the downstream-most section of apron was assumed equal to tailwater above the slab due to its slender structural section.
- Drawings indicate a clay blanket upstream of the ogee spillway. The upstream soil load was computed using an at-rest Rankine earth pressure coefficient. The at-rest soil wedge downstream of slab 2 is ignored for all analyses.
- The modeled headwater hydrostatic load acts between the spillway sill and the bottom of slab 1. Horizontal tailwater resisting load is ignored for all analyses.

  The ice load condition assumes the normal reservoir elevation with an ice load applied 6 inches below headwater (assuming a 12-inch-day thick ice layer). The ice lock-in pressure used in the analysis was 5,000 psf, consistent with FERC Guidelines (Ref. FERC, 2016).

3.3 Material Properties Summary

The following material properties were utilized in the analysis:

Table 2: Material Properties

Material	Property	Value
Water	Unit Weight	62.4 pcf
Concrete	Total unit weight	150 pcf
Upstream Soil, Clay Fill	Saturated Unit Weight	140 pcf
	Internal Friction Angle	30
	At-Rest Earth Pressure Coefficient	0.5
Hard Foundation Soil	Interface friction angle	37 deg
	Cohesion	0 psf
Ice	Ice Load (12” Thickness)	5,000 psf

3.4 Evaluation Criteria

The dam was evaluated for sliding, overturning, bearing pressure, and flotation. Stability criteria was established using Table 4 of EM 1110-2-2100 assuming ordinary site information and a critical structure (Ref. USACE, 2005). Refer to the following table summarizing the stability criteria.

Table 3: Stability Criteria Summary

Loading Condition	Sliding Factor of Safety (FS)	Overturning Criteria Minimum Base Area in Compression	Bearing Safety Factor	Flotation Safety Factor
Usual	2.0	100%	3.0	1.3
Unusual	1.5	75%	2.0	1.2

The structure is founded on very dense sands and gravels with an estimated ultimate bearing capacity of 50 ksf based on the Meyerhoff method (Ref. USACE, 1992). Refer to Appendix D for an estimate of the soil bearing capacity.

3.5 Stability Results

The following table summarizes the stability analysis results for the sluiceway ogee section. Internal stresses were not evaluated as part of these analyses. The analyzed section was found to satisfy stability criteria for all analyzed load cases. Refer to **Appendix D** for the stability analysis computations.

Table 4: Stability Results Summary

Parameter	Normal Load Case I	Normal + Ice Load Case IIA	Flood Pool Load Case II
Headwater El. (ft)	877.52	877.52	880.2
Tailwater El. (ft)	861	861	863
Interface Friction Angle (°)	37	37	37
Cohesion (psi)	0	0	0
% Base in Compression	100%	100%	100%
Eccentricity (1)	L/-13.4	L/20	L/-35.7
Base Pressure at Heel (ksf) (2)	1.36	0.66	1.03
Base Pressure at Toe (ksf) (2)	0.52	1.22	0.73
Sliding Safety Factor	2.0	1.5	1.5
Sliding Safety Factor Req'd	*2.0*	*1.5*	*1.5*
Flotation Safety Factor	2.1	2.1	1.9
Flotation Safety Factor Req'd	*1.3*	*1.2*	*1.2*

Note:
(1) Presented as fraction of the Base Length (L) (Upstream to Downstream Dimension). Edge of kern = L/6.
(2) All presented values considered acceptable bearing capacities for a till foundation material.

4. Conceptual Repairs and Maintenance

Based on the 2022 EGLE inspection report recommendations and the 2023 GEI and JFB site observations, the following repairs and modifications should be considered and are outlined in the Conceptual Drawings provided in **Appendix E**:

1. Repair deteriorated concrete,
2. Remove the trashrack and condensing unit lines in the penstock intake structure,
3. Install a permanent, engineered bulkhead in the stoplog slots of the intake structure and on the upstream opening of the penstock,
4. Infill the intake structure with a controlled low strength material (CLSM), and
5. Remove and replace the left sluice gate and trashrack.

Item 1 – Repair concrete: There are areas of concrete deterioration throughout the structure ranging from delamination to spalling with rebar exposed. Allowing the concrete to continue to deteriorate can results in the loss of structural integrity of the concrete structures. As rebar becomes exposed, it will accelerate corrosion which will result in the loss of steel cross-section area and strength. Repairing surficial concrete deterioration as it occurs can help reduced the chances of more extensive costly repairs in the future.

Item 2 through 4 – Intake structure modifications: Currently, all the gates and turbines are in the closed condition and the hydroelectric operations are abandoned. However, there were no actions taken to abandon flow into the penstock and there is a potential for uncontrolled release of water if the corroding gates and mountings fail.

GEI proposes installing a permanent bulkhead in the penstock intake stoplog slot and on the upstream side of the penstock to permanently seal the penstock and eliminate the flow of water. Furthermore, the penstock intake has a large opening on the top deck with minimal security features to deter the public from entering or falling into the deep intake area. To eliminate this public safety risk and future maintenance concerns, GEI proposes that you infill between the bulkheads with CLSM.

Item 5 – Install new sluice gate: As recommended in the EGLE inspection report, one gate should be operational to allow control (lowering) of the headwater elevation in the event that the impoundment needs to be lowered. The current operable condition of the gates is unknown. Furthermore, the trashracks upstream of the gates are heavily rusted and have significant marine growth. GEI proposes that you remove and replace the left trashrack and sluice gate. The new trashrack and sluice gate should be sized to fit existing concrete dimensions and engineered to operate under heavy debris loading that can be expected after years between use. In addition, the Department of Public Works should be trained to operate the sluice gate and the regular operation and maintenance of the gate should be included in the O&M manual.

4.1 Cost Estimates

GEI has developed construction cost estimates for the items listed above. The estimated costs were developed in accordance with AACE 69R-12 - Class 4 which allows for an accuracy range of plus 20% to 50% on the high end, and minus 15% to 30% on the low end, after the application of contingency. This represents about an 80% confidence level that the actual cost will fall within the bounds of the low and high ranges (AACE 69R-12). Our estimated costs include an assumed 30% contingency to account for unknown risks at this early stage in design.

Line items for the cost estimate were developed from the scope of work discussed above. The line items include a full bay-width cofferdam as an alternate to the upstream bulkhead/stop logs. Quantities used in the cost estimate were estimated from inspection notes and photos, available project drawings, preliminary GEI design concepts, and engineering judgement. Unit prices for each line item were developed using a combination of RS Means construction cost estimating software, contractor bid prices from similar construction projects, and engineering judgement.

The total estimated cost plus contingency is $1,605,000. A detailed cost breakdown is included within **Appendix F**.

5. Conclusions and Recommendations

5.1 Conclusions

GEI reviewed the existing provided documentation, preformed a site visit with JFB including dive inspection to visually assess the existing structure conditions, and performed global stability analysis on the primary spillway structure. Based on our investigation and analysis, the dam is in fair condition. If the dam is to remain in place based on current regulatory requirements, the following would need to be addressed and maintained:

- Replace at least one existing sluice gate, operating it according to the latest Operation and Maintenance (O&M) program.
- Remove and replace the left sluice gate - trashrack.
- Repair deteriorated concrete on the spillway and surrounding structures.
- Install a bulkhead on the upstream side of the penstock intake to obstruct flow into the penstock and powerhouse and backfill intake structure.
- Continue vegetation removal from embankments and address any animal burrows.
- Monitor runoff and erosion on the downstream right embankment and reinforce as needed.
- Regular inspections and operation of the new sluice gate by the Village staff.
- Clearing of debris from the spillway.

However, if the dam is to remain in place and the more stringent regulatory requirements were to be put in place, refer to **Appendix H**, the following additional items would have to be addressed and maintained:

- Increased frequency of engineering inspections.
- Licensing, financial assurance, and insurance requirements.
- EAP increased requirements.
- Required independent comprehensive reviews.
- Potential need for increased spillway capacity based on updated flood requirements.

At this time, the dam in its current condition does not appear to pose an immediate loss of life threat; however, if the above-mentioned repairs and maintenance are not addressed, the condition of the dam will continue to deteriorate over time and repairs will become more costly. In addition, repairs not listed above may also be required as the structure continues to age.

5.2 Recommendations

The structural inspection and analyses have identified repairs that should be addressed in the near-term if the Village plans to continue maintaining the dam. However, based on discussions with the Village, there is limited or no beneficial use for the community from the structure or its impoundment. Given this limited benefit, coupled with limited funding opportunities for dam repair and rehabilitation, it is recommended that the Village explore opportunities for removing the dam and restoring the River Raisin through this reach. **Appendix H** outlines the evaluation of dam disposition options and identifies project partners and funding opportunities for dam removal.

6. References

(EGLE, 2022) Department of Environment, Great Lakes, and Energy (EGLE), *Dam Safety Inspection Report Ford Manchester Dam*, August 4, 2022.

(FEMA, 2012) Federal Emergency Management Agency (FEMA), *Flood Insurance Study No. 26161CV001A*, April 3, 2012.

(FERC, 2016), Federal Energy Regulatory Commission (FERC), *Engineering Guidelines for the Evaluation of Hydroelectric Projects*, Chapter 3 –Gravity Dams, March 4, 2016.

(FORD, 1939) Ford Motor Co., *Design Drawings,* 1939.

(USACE, 1978) U.S. Army Corps of Engineers (USACE), *River Raisin Basin Ford Manchester Dam, Inspection Report, National Dam Safety Program,* September 1978.

(USACE, 1992) USACE, *EM1110-1-1905 – Bearing Capacity of Soils*, October 30,1992.

(USACE, 2005) USACE, *EM1110-2-2100 – Stability Analysis of Concrete Structures*, December 1, 2005.

Appendix A

Historic Documents

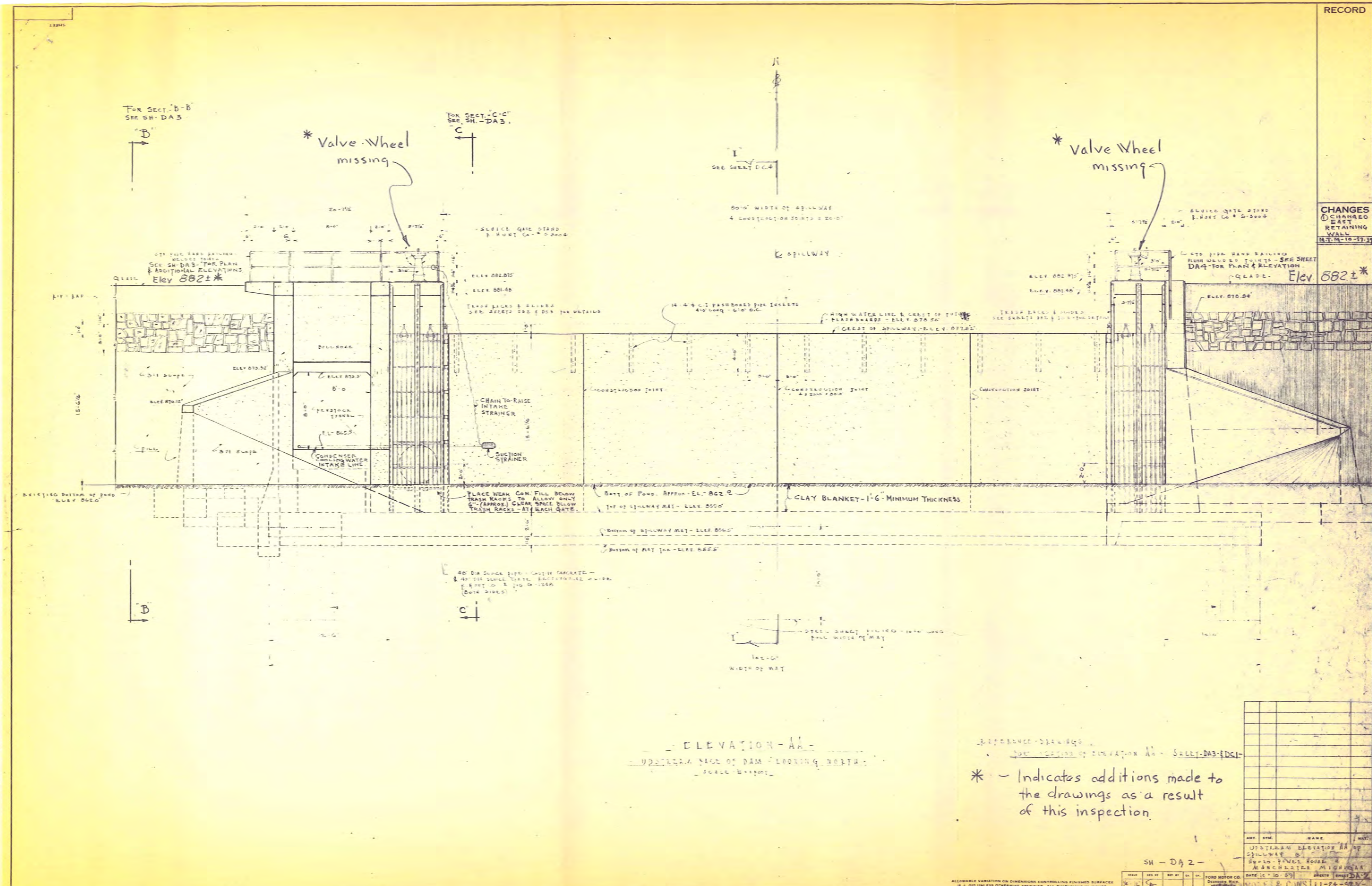
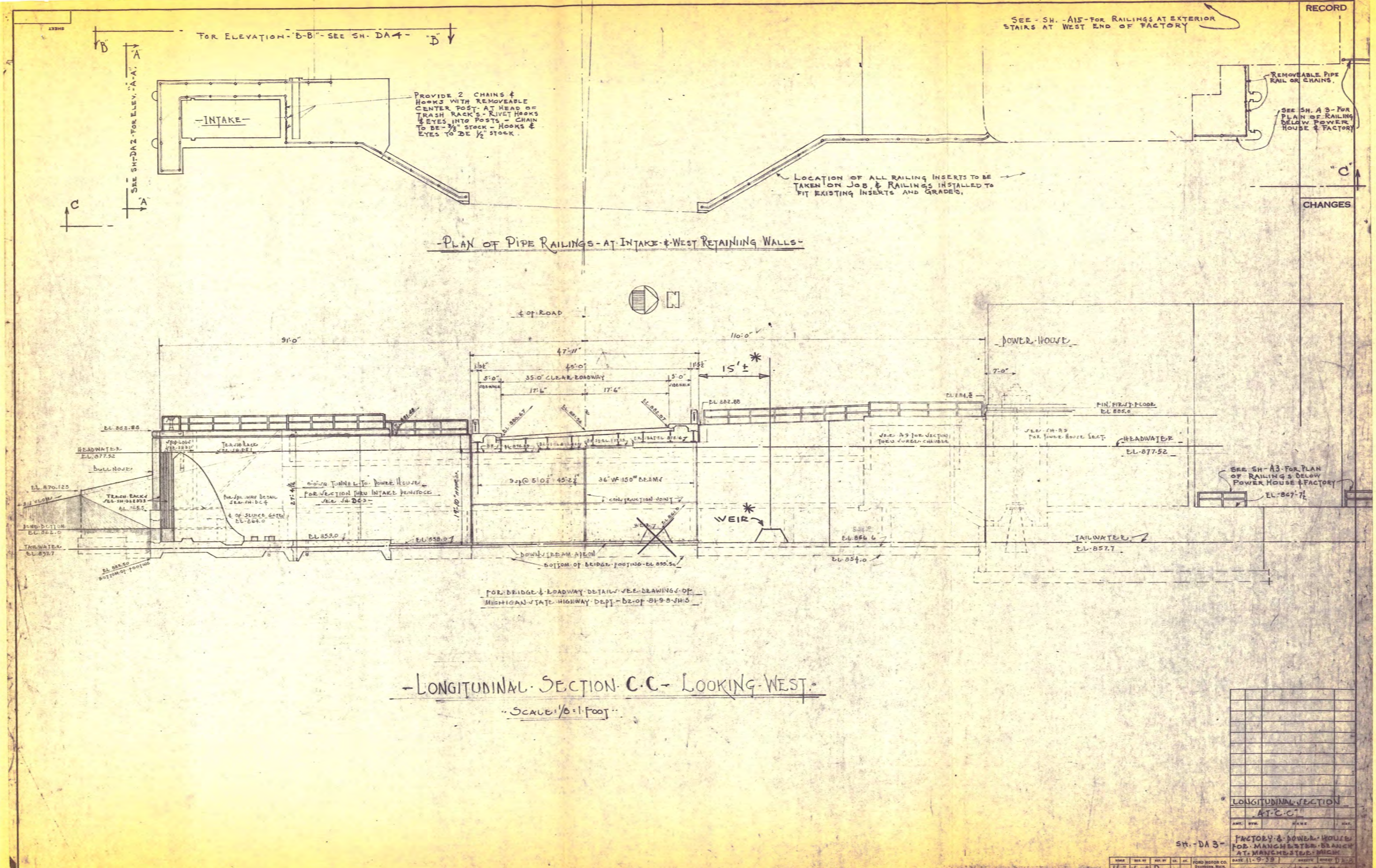
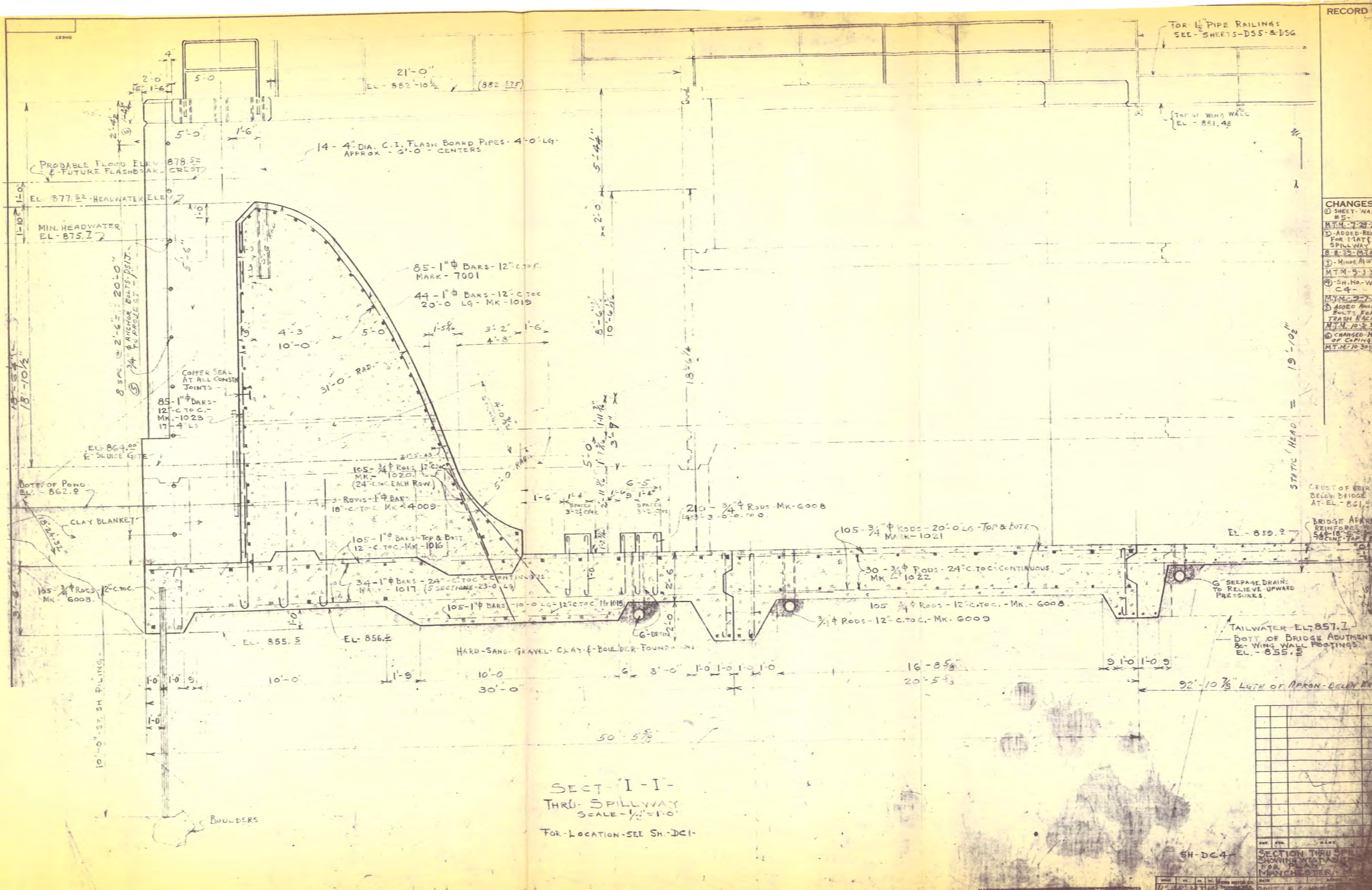

RECORD
CHANGES
① CHANGED EAST RETAINING WALL
FOR SECT. "B-B" SEE SH-DA3
FOR SECT. "C-C" SEE SH.-DA3.
* Valve Wheel missing
* Valve Wheel missing
SEE SHEET DC-4
80'-0" WIDTH OF SPILLWAY
℄ SPILLWAY
SEE SH-DA3- FOR PLAN & ADDITIONAL ELEVATIONS
GRADE
Elev 882± *
Elev. 882± *
ELEV 882.875'
ELEV 881.46'
BULLNOSE
PENSTOCK TUNNEL
CONDENSER COOLING WATER INTAKE LINE
CHAIN TO RAISE INTAKE STRAINER
SUCTION STRAINER
CONSTRUCTION JOINT
HIGH WATER LINE & CREST OF TOP OF FLASHBOARDS - ELEV 878.50'
CREST OF SPILLWAY - ELEV. 877.52'
PLACE WEAK CON. FILL BELOW TRASH RACKS TO ALLOW ONLY 6" (APPROX.) CLEAR SPACE BELOW TRASH RACKS - AT EACH GATE
BOTT. OF POND. APPROX.-EL. 862.0
CLAY BLANKET - 1'-6" MINIMUM THICKNESS
EXISTING BOTTOM OF POND ELEV 862.0'
102'-0" WIDTH OF MAT
ELEVATION - A-A
UPSTREAM FACE OF DAM - LOOKING NORTH
REFERENCE DRAWINGS
* - Indicates additions made to the drawings as a result of this inspection
SH - DA 2 -
ALLOWABLE VARIATION ON DIMENSIONS CONTROLLING FINISHED SURFACES IS ± .010 UNLESS OTHERWISE SPECIFIED. ALL DIMENSIONS IN INCHES
FORD MOTOR CO. DEARBORN, MICH.
UPSTREAM ELEVATION AA OF SPILLWAY @ HYDRO-POWER HOUSE @ MANCHESTER MICHIGAN
DATE 10-10-39
SHEET DA-2
POWER & CONS

FIGURE 6



FOR ELEVATION "B-B" SEE SH. DA4
SEE SH-DA2 FOR ELEV. "A-A"
INTAKE
PROVIDE 2 CHAINS & HOOKS WITH REMOVEABLE CENTER POST AT HEAD OF TRASH RACKS - RIVET HOOKS & EYES INTO POSTS - CHAIN TO BE 3/8" STOCK - HOOKS & EYES TO BE 1/2" STOCK.
SEE SH. A15 FOR RAILINGS AT EXTERIOR STAIRS AT WEST END OF FACTORY
REMOVEABLE PIPE RAIL OR CHAINS.
SEE SH. A3 FOR PLAN OF RAILING BELOW POWER HOUSE & FACTORY
LOCATION OF ALL RAILING INSERTS TO BE TAKEN ON JOB, & RAILINGS INSTALLED TO FIT EXISTING INSERTS AND GRADES.
RECORD
CHANGES
PLAN OF PIPE RAILINGS AT INTAKE & WEST RETAINING WALLS
℄ OF ROAD
91'-0"
110'-0"
47'-11"
45'-0"
35'-0" CLEAR ROADWAY
5'-0" SIDEWALK
17'-6"
15'±
POWER HOUSE
7'-0"
EL. 882.88
HEADWATER EL. 877.52
BULLNOSE
EL. 870.125
TRASH RACKS
POND BOTTOM EL. 862.0
TAILWATER EL. 857.7
BOTTOM OF FOOTING
GOING TUNNEL TO POWER HOUSE
FOR SECTION THRU INTAKE PENSTOCK SEE SH. DC3
FOR SPILLWAY DETAIL SEE SH. DC4
℄ OF SLUICE GATES EL. 864.0
EL. 859.0
36" WF 150# BEAMS
CONSTRUCTION JOINT
WEIR
DOWNSTREAM APRON
BOTTOM OF BRIDGE FOOTING EL. 855.50
EL. 856.6
EL. 854.0
SEE A3 FOR SECTION THRU SURGE CHAMBER
SEE SH. A3 FOR POWER HOUSE SECT.
FIN. FIRST FLOOR EL. 885.0
HEADWATER EL. 877.52
SEE SH. A3 FOR PLAN OF RAILINGS BELOW POWER HOUSE & FACTORY
EL. 867'-7½
TAILWATER EL. 857.7
FOR BRIDGE & ROADWAY DETAILS SEE DRAWINGS OF MICHIGAN STATE HIGHWAY DEPT. - B2 OF 81-9-8 - SH. 3
LONGITUDINAL SECTION C-C - LOOKING WEST
SCALE: 1/8" = 1 FOOT
LONGITUDINAL SECTION AT "C-C"
SH. DA3
FACTORY & POWER HOUSE FOR MANCHESTER BRANCH AT MANCHESTER, MICH.
DATE 11-9-39
FORD MOTOR CO. DEARBORN, MICH.
POWER & CONSTR. 24-928
1/8" = 1'-0"

FIGURE 7



SECT "I-I"
THRU SPILLWAY
SCALE - 1/2" = 1'-0"
FOR LOCATION - SEE SH. - DC1 -
FOR 1 1/2" PIPE RAILINGS SEE - SHEETS - DS5 & DS6
14 - 4" DIA. C.I. FLASH BOARD PIPES - 4'-0" LG - APPROX - 6'-0" CENTERS
PROBABLE FLOOD ELEV. 878.52 & FUTURE FLASHBOARD CREST
EL. 877.52 - HEADWATER ELEV.
MIN. HEADWATER EL - 875.7
EL - 882'-10 1/2
TOP OF WING WALL EL. - 881.48
85 - 1" ϕ BARS - 12" C.TO C. MARK - 7001
44 - 1" ϕ BARS - 12" C.TO C. 20'-0 LG - MK - 1019
31'-0" RAD.
COPPER SEAL AT ALL CONSTR. JOINTS
EL. 864.00 ℄ SLUICE GATE
BOTT. OF POND EL. - 862.9
CLAY BLANKET
105 - 3/4 ϕ RODS - 12" C.TO C. MK - 6008
105 - 1" ϕ BARS - TOP & BOTT. 12 - C.TO C. - MK - 1016
34 - 1" ϕ BARS - 24" C.TO C. CONTINUOUS MARK - 1017 (5 SECTIONS - 23-0 LG)
105 - 3/4 ϕ RODS - 20'-0 LG - TOP & BOTT. MARK - 1021
30 - 3/4 ϕ RODS - 24" C.TO C. CONTINUOUS MK - 1022
105 - 3/4 ϕ RODS - 12" C.TO C. - MK. - 6008
3/4 ϕ RODS - 12" C.TO C. - MK. 6009
6" SEEPAGE DRAINS TO RELIEVE UPWARD PRESSURES
6" DRAIN
STATIC HEAD = 19'-10 1/2"
CREST OF WEIR BELOW BRIDGE AT EL. - 861.5
EL. - 859.9
TAILWATER EL. 857.7
BOTT. OF BRIDGE ABUTMENT & WING WALL FOOTINGS EL. - 855.5
EL - 855.5
EL - 856.5
HARD - SAND - GRAVEL - CLAY & BOULDER - FOUNDATION
10'-0" ST. SH. PILING
BOULDERS
92'-10 7/8" LGTH OF APRON - BELOW BRIDGE
50'-5 5/8
30'-0"
RECORD
CHANGES
SH - DC4
SECTION THRU SPILLWAY
POWER & CONSTR.

FIGURE 8

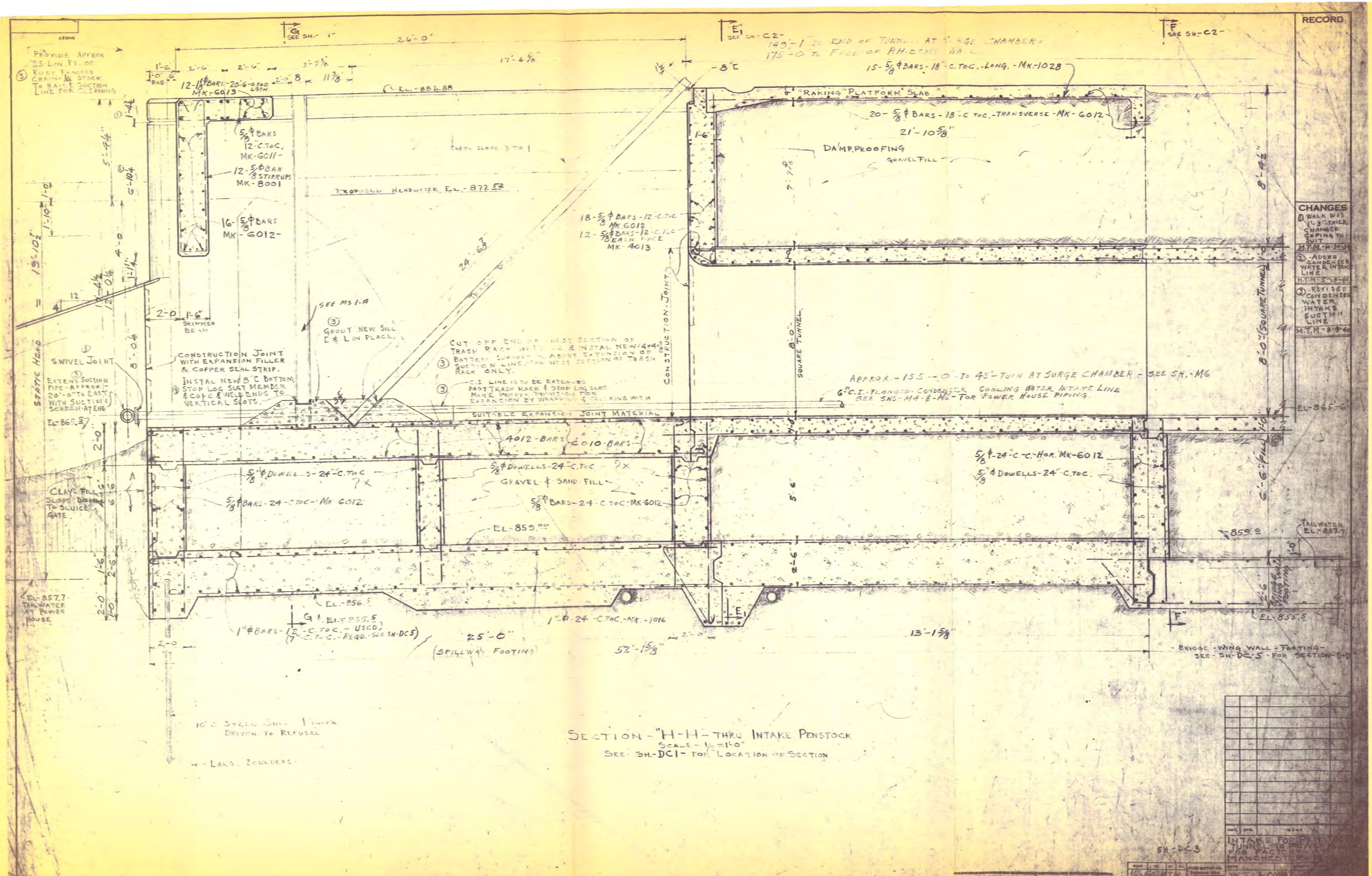

RECORD
CHANGES
SECTION - "H-H" - THRU INTAKE PENSTOCK
SCALE - 1/2" = 1'-0"
SEE SH.-DC1- FOR LOCATION OF SECTION
RAKING PLATFORM SLAB
15 - 5/8"ϕ BARS - 18" C. TOC. - LONG. - MK-1028
20 - 5/8"ϕ BARS - 18" C TOC. - TRANSVERSE - MK-6012
DAMP PROOFING
GRAVEL FILL
EL. - 882.88
PROPOSED HEADWATER EL. - 877.52
SQUARE TUNNEL
8'-0" (SQUARE TUNNEL)
APPROX. - 155'-0" - TO 45° - TURN AT SURGE CHAMBER - SEE SH. - M6
6" C.I. - FLANGED - CONDENSER COOLING WATER INTAKE LINE
SEE SHS - M4 - & - M6 - FOR POWER HOUSE PIPING
SUITABLE EXPANSION JOINT MATERIAL
CONSTRUCTION JOINT
CONSTRUCTION JOINT WITH EXPANSION FILLER & COPPER SEAL STRIP.
INSTAL NEW 8" C BOTTOM STOP LOG SLAT MEMBER & COPE & WELD ENDS TO VERTICAL SLOTS.
GROUT NEW SILL C & L IN PLACE.
SEE MS 1-A
SWIVEL JOINT
EXTEND SUCTION PIPE - APPROX. - 20'-0" TO EAST WITH SUCTION SCREEN AT END
PROVIDE APPROX. 25 LIN. FT. OF RUST PROOFED CHAIN - 1/4" STOCK TO RAISE SUCTION LINE FOR CLEANING
STATIC HEAD
SKIMMER BEAM
CLAY FILL SLOPE DOWN TO SLUICE GATE
EL. - 857.7 TAILWATER AT POWER HOUSE
GRAVEL & SAND FILL
EL. - 859.00
EL. - 856.5
EL. - 855.5
25'-0" (SPILLWAY FOOTING)
52'-1 5/8"
13'-1 5/8"
26'-0"
17'-4 5/8"
21'-10 5/8"
BRIDGE - WING WALL - FOOTING - SEE - SH - DC - 5 - FOR SECTION - D-D
TAILWATER EL. - 857.7
EL - 865.6
10'0" STEEL SHEET PILING DRIVEN TO REFUSAL
SH - DC3
INTAKE FOR PENSTOCK TUNNEL
MANCHESTER
POWER & CONST.

FIGURE 9

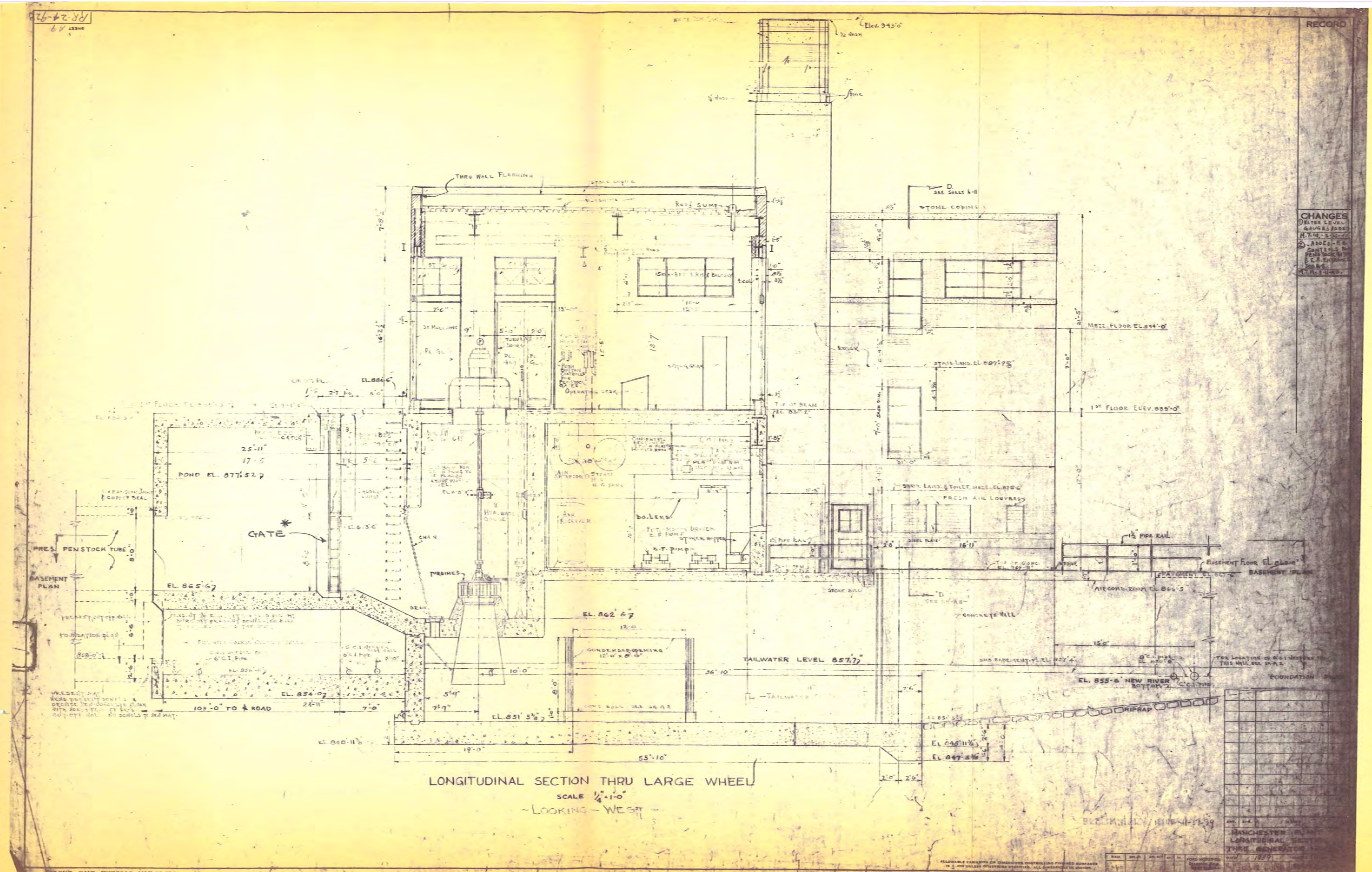

RR 24-92
RECORD
CHANGES
Elev. 943'-0"
THRU WALL FLASHING
STONE COPING
ROOF SUMP
MEZZ. FLOOR EL 894'-0"
STAIR LAND. EL 889'-9"
1ST FLOOR ELEV. 885'-0"
EL. 884'-6"
POND EL. 877'-5 2/7
GATE *
PRES. PENSTOCK TUBE
BASEMENT PLAN
FOUNDATION PLAN
EL. 865'-6 7
EL. 862'-6 7
CONDENSER OPENING 12'-0" X 8'-0"
TAILWATER LEVEL 857'-7
BOILERS
TURBINES
FRESH AIR LOUVRES
1 1/2" PIPE RAIL
BASEMENT FLOOR EL
AIR COND. ROOM EL 866'-5
CONCRETE WALL
EL. 855'-6 NEW RIVER BOTTOM
RIPRAP
FOUNDATION PLAN
EL. 854'-0 7
103'-0" TO ℄ ROAD
EL. 851' 5 3/8"
EL. 848'-11 3/8"
EL. 847' 5 3/8"
19'-0"
55'-10"
LONGITUDINAL SECTION THRU LARGE WHEEL
SCALE 1/4"=1'-0"
~LOOKING ~ WEST ~
MANCHESTER PLANT
LONGITUDINAL SECTION
THRU GENERATOR

FIGURE 10

DAM SAFETY INSPECTION REPORT
FORD MANCHESTER DAM – DAM ID NO. 391
RIVER RAISIN
WASHTENAW COUNTY – SECTION 1, T 04S, R 03E

OWNER(S)/OPERATOR(S):	Village of Manchester 912 City Road PO Box 485 Manchester, MI 48158 (734) 428-7877
HAZARD POTENTIAL CLASSIFICATION:	High
INSPECTION DATE:	May 17, 2022
REPORT DATE:	August 4, 2022

PREPARED AND INSPECTED BY:

Thomas Horak, E.I.T.
Dam Safety Unit
Water Resources Division
Dept. of Environment, Great Lakes, and Energy
P.O. Box 30458
Lansing, Michigan 48909
517-231-8594

Lucas A. Trumble, P.E.
Dam Safety Unit
Water Resources Division
Department of Environment, Great Lakes, and Energy
P.O. Box 30458
Lansing, Michigan 48909
517-420-8923

STATE OF MICHIGAN
LUCAS A. TRUMBLE
ENGINEER NO.
58295
LICENSED PROFESSIONAL ENGINEER

INTRODUCTION

The purpose of this inspection was to evaluate the structural condition and hydraulic capacity of the Ford Manchester Dam, as required by Part 315, Dam Safety (Part 315), of the Natural Resources and Environmental Protection Act, 1994 PA 451, as amended. This inspection was conducted by the Department of Environment, Great Lakes, and Energy (EGLE) in response to a request by the owner of the dam, Village of Manchester Washtenaw County . The report is limited to a discussion of observations based on a visual investigation and review of any available previous inspection reports, plans, and data. This report should not be considered an in-depth engineering investigation. All references to "right" and "left" in this report are based on the observer facing downstream.

CONCLUSIONS AND RECOMMENDATIONS

The Ford Manchester Dam is in Fair condition. 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. The following recommended actions are listed by priority:

1. Complete a detailed structural evaluation of the principal spillway and powerhouse structures, including a plan and schedule for any necessary repairs, within the next year. This has previously been recommended in each inspection since 2013 to be completed by 2023.

2. Continue efforts to remove all trees and brush from the earthen embankments. After clearing, mow and/or treat the entire embankment a minimum of two times per year to prevent further establishment of woody vegetation and facilitate visual inspection. When cleared, the embankment should have proper, non-woody vegetative cover established and maintained. Trees and brush that should be removed were observed at the upstream slopes of both embankments and on the downstream slope of the right embankment where woody vegetation is encroaching onto the embankment past the groin and downstream toe. The embankment should be cleared to 10 feet beyond the groin of the embankment and to 10 feet beyond the downstream toe.

3. Fill the animal burrow at the left end of the spillway deck. Monitor the site for additional burrows and fill them as observed.

4. Restore at least one of the drawdown gates to an operational condition or establish other methods of impoundment drawdown should it become necessary.

5. Monitor the storm sewer outfall on the downstream slope of the right embankment for further erosion. If erosion progresses, further armor the flow path.

6. Develop a written Operation and Maintenance Plan (O&M Plan) for the dam and submit a copy to the Dam Safety Program.

7. Provide the updated Emergency Action Plan to the Dam Safety Unit by December 31, 2022.

The dam's current High hazard potential rating remains appropriate.

PROJECT INFORMATION

The Ford Manchester Dam was originally constructed in 1940 by the Henry Ford Motor Company to generate hydroelectric power. Hydropower generation has since been abandoned. The dam, located on the River Raisin in the Village of Manchester, was purchased by the Village in 2000, who converted the former powerhouse into its office space. The dam consists of an 80.5-foot wide concrete principal spillway, an abandoned concrete and brick powerhouse, a 540-foot long left earthen embankment, and a 190-foot long right earthen embankment.

The principal spillway structure consists of a single ogee crest spillway with two 4-foot diameter, gated drawdown structures at each end of the spillway. The powerhouse intake structure lies immediately adjacent to the left end of the principal spillway and feeds twin turbines located within the powerhouse building. Head gates at the powerhouse currently block all flow through the intake and turbines. No auxiliary spillway exists at the dam.

The earthen embankments serve as the roadbed for M-52, with a bridge passing over the river channel immediately downstream of the principal spillway. The embankments have crest widths of approximately 35 feet and upstream and downstream slopes of approximately 3 horizontal to 1 vertical (3H:1V).

The dam has a structural height of 26.5 feet and a hydraulic height of 24.6 feet. It maintains approximately 20 feet of head with 3.5-4.0 feet of freeboard, creating a 45-acre impoundment under normal flow conditions.

The Ford Manchester Dam was initially inspected by Burgess & Niple, Ltd. in 1978 for the U.S. Army Corps of Engineers as part of the National Dam Safety Program. Subsequently, the dam was inspected by Mr. Gary Croskey, P.E., in 1992, 1995, and 1998; Tetra Tech in 2000; and under Part 315 by EGLE staff in 2004, 2007, 2010, 2013, 2016, and 2019. The 1978 Corps report includes copies of the original design plans for the dam and the 2000 Tetra Tech report includes an underwater inspection by Stolt Comex Seaway, Incorporated. Copies of these reports, along with engineering sketches of the dam, are on file with the Dam Safety Program. The 2019 inspection report prepared by EGLE's Dam Safety Unit was used as a reference for this report.

SITE INVESTIGATION

The following discussion of the dam's physical condition and appurtenances is based on observations and photographs obtained on the inspection date.

In addition to the specific findings listed below, it is important to continue good maintenance practices. These practices include regular inspection of the dam embankments and hydraulic structures for any deficiencies. Some of the more common issues that are found include growth of trees and brush, development of erosion areas, and animal burrows.

If woody vegetation is allowed to mature, it could develop an extensive root system. These root systems can lead to piping failure or if the brush and trees are uprooting in a storm, can cause extensive deterioration of the embankment. Embankments should be clear of woody vegetation and mowed 10 feet past the toe of the embankment. Similarly, animal burrows and surface erosion, can propagate into increased seepage and potentially piping failure, as well as lead to slope stability issues.

The following data was collected on the date of the inspection and includes deficiencies observed during the inspection and necessary actions for remediation of the observed deficiencies.

Upstream Slope	
Pool elevation at time of inspection (ft)	*54" from water to top of steel stoplog guide at right drawdown orifice. 55"-60" below concrete deck at drawdown gate; Top of concrete is 0.875' above embankment grade, per 1939 plan set.*
Upstream slope ground cover	*Minor brush, some bare spots*
What issues are present on the upstream slope?	*Trees, Brush, Ground Cover Issues, Rodent Burrows*
Slope Protection	
What types of slope protection are used?	*None*
Trees/Woody Vegetation	
Number of trees	*Sparse*
Tree DBH	*<6"*
Tree location	*Other*
Specify other.	*Upstream slope*
Action required for trees	*Maintenance*
Describe action required	*Remove trees and brush from entire embankment*
Brush coverage	*Sparse*
Brush location	*Upstream Slope*
Action required for brush	*Maintenance*
Describe action required	*Remove trees and brush from entire embankment*
Ground Cover / Vegetation Issues	
Ground cover type	*Brush and grass*
Ground cover issues	*Bare*
Action required for ground cover	*Monitor*
Describe action required	*Monitor and establish non woody vegetation growth or armor. When the embankment is cleared, establish appropriate vegetative cover. Unarmed banks on the left embankment (west) are of less concern than typical for a dam due to the large width of the embankment*
Animal Burrows	
Approximate number of rodent burrows	*One observed*
Location of rodent burrows	*Other*

Specify other.	*Under slab of spillway abutment deck*
Action required for rodent burrows	*Maintenance*
Describe action required	*Fill existing burrow and monitor for additional burrows to fill as observed*

Crest	
Approximate width of crest (ft)	*50*
Approximate freeboard (ft)	*3.5-4.0*
Crest ground cover	*City Road and village office grounds on left embankment*
What issues are present on the crest?	*Trees*
Trees/Woody Vegetation	
Number of trees	*Sparse*
Tree DBH	*6-12"*
Location of the trees	*Other*
Specify other.	*Within village office landscaping*
Action required for trees	*Monitor*
Describe action required	*Monitor and remove trees if they die. Trees are generally discouraged on embankments. However, given the width of the embankment at the location of the village offices where the trees are planted, there is very little concern for dam failure due to the trees from seepage impacts or blowover. If embankment deficiencies are observed around trees, the Dam Safety Unit should be alerted.*

Downstream Slope	
Downstream slope ground cover	*Brush grass and village office landscaping*
What issues are present on the downstream slope?	*Runoff Erosion (Gullies)*
Trees/Woody Vegetation	
Number of trees	*Sparse*
Tree DBH	*<6"*
Tree location	*Other*
Specify other.	*Downstream slope of right embankment (east) and several landscaping trees on village office grounds*
Action required for trees	*Maintenance*
Describe action required	*Remove trees and brush from right embankment, to 10' beyond the downstream toe of the slope and 10' beyond the groin*
Brush coverage	*Sparse*
Brush location	*Downstream slope of right embankment (east)*
Action required for brush	*Maintenance*

Describe action required	*Remove trees and brush from right embankment, to 10' beyond the downstream toe of the slope and 10' beyond the groin*
Runoff Erosion / Gullies	
Quantity of runoff erosion	*Minor at storm sewer outfall on right embankment*
Approximate depth of gullies (ft)	*0.5*
Approximate width of gullies (ft)	*0.5*
Location of runoff erosion	*Storm sewer outfall on right downstream slope*
Action required for runoff erosion	*Monitor*
Describe action required	*Monitor for further erosion. Gullies have some small armoring present, and it seems like they are fairly stable, at least right at the storm sewer outfall. If erosion continues, add more armoring after restoring the slope*
Embankment / Internal Drains	
What types of embankment drains are present?	*Other*
Specify other.	*Wall drains at bottom of spillway's left downstream wall. There is also a storm sewer outfall on the right embankment's downstream slope*
Issues with embankment drains:	*None observed, see storm sewer outfall recommendation above*
Action required for embankment drains:	*None*

Principal Spillway	
What type of spillway is present?	*Weir/Channel*
What type of weir is present?	*Ogee*
What is the primary material used in the spillway?	*Concrete*
Which components are present?	*Low-flow orifice – not functional currently*
What issues are present with the primary spillway?	*Deteriorating Materials*
Material Deterioration	
What materials are deteriorating in the spillway?	*Concrete*
What issues are noted with the concrete components?	*Efflorescence, Spalling, Exposed Rebar, Other*
Specify other.	*Unknown concrete structural condition*
How large is the impacted area (in)?	*Deterioration is not an isolated deficiency, it is observed throughout,*
Where are the issues located?	*Throughout, especially at the spillway abutment walls and the drawdown gate operator decks*
Action required for concrete components of the spillway	*Maintenance, Engineer*
Describe action required	*Complete a detailed structural evaluation of the principal spillway and powerhouse structures, including a plan and schedule for any necessary repairs*
Erosion Control / Energy Dissipation	
What type of erosion control structure is in place?	*End Sill*
Are there any issues with the outlet erosion control structure?	*None*
Does the outlet erosion control structure include any drains?	*None observed*

Gates / Valves	
Does the spillway include a gate?	*Yes*
What type of gate?	*Slide Gates*
Are there any issues with the gate?	*Operability*
Action required for the gate	*Maintenance, Engineer*
Describe action required	*Consider replacing, rehabilitating, or decommissioning with an alternative means for drawdown developed*
Additional comments	*During the inspection, the Village asked about pumping contractors to have on call as needed. Please review with your consulting engineer and ask if they know any local companies. EAPs of nearby dams were reviewed, but no pumping contractors were included in them.*

Auxiliary Spillway	
What type of spillway is present?	*Previous powerhouse sluiceway. Is currently out of commission*
What is the primary material used in the spillway?	*Concrete*
What issues are present with the auxiliary spillway?	*None observed, although the structure should be evaluated as recommended previously*
Additional comments	*Could not access sluiceway upstream of powerhouse*

Appurtenant Structures	
Inspect appurtenant structures	
Describe the appurtenant structure	*Village office lobby with historic powerhouse equipment. See photos for old powerhouse equipment*

Other	
Warning signs, alarms, buoys:	*Signs posted: "No Trespassing No Fishing"*
Security features:	*Fences around spillway inlet and outlet by village office*
Staff gage:	*None*

The above monitoring and maintenance items should be addressed in accordance with the Conclusions and Recommendations section of this report.

STRUCTURAL STABILITY

While significant concrete deterioration is evident throughout the structure, this condition does not pose an immediate threat to the dam's stability. This deterioration was observed in previous inspections and has not increased significantly since the 2019 inspection. It was recommended in the 2019 inspection report to have a comprehensive structural analysis of the spillway and powerhouse completed within four years. No analysis of the spillway structure or major dam repairs have been completed, so it is recommended that such an analysis be completed within the next year and any necessary repairs be implemented as recommended in that report.

HYDROLOGY AND HYDRAULICS

The contributing drainage area to the River Raisin at the Ford Manchester Dam is approximately 149 square miles. The design discharge for this high hazard potential dam is the 0.5-percent annual chance (200-year) flood discharge, which is estimated to be 1,300 cubic feet per second (cfs).

Using the weir equation with an ogee weir coefficient of 3.8, the 80.5-foot long spillway can pass the design flood inflow with approximately 2.65 feet of head. This leaves approximately 1.85 feet of freeboard at the earthen embankments. Therefore, the dam is considered to have adequate spillway capacity to safely convey the design flood.

Copies of the hydraulic calculations used to make this determination are on file with the Dam Safety Program.

OPERATION AND MAINTENANCE

Operation of the dam is by staff of the Village of Manchester. According to our records, a written O&M Plan has never been prepared for this dam. An O&M Plan should be prepared that addresses day-to-day operation, as well as operation during flood conditions. This plan should be reviewed regularly, with updated copies provided to the Dam Safety Program.

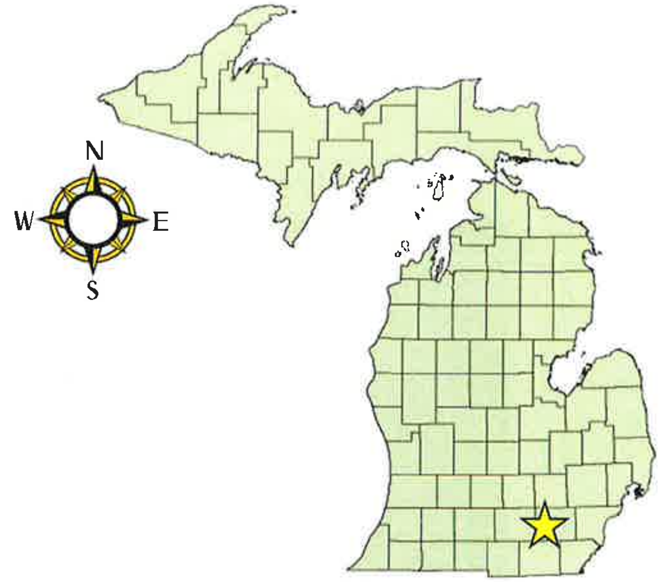
EMERGENCY ACTION PLAN

The Ford Manchester Dam has been assigned a high hazard potential rating. As such, the owner is required under Part 315 to prepare, and keep up to date, an Emergency Action Plan (EAP) for the dam. A written EAP was originally prepared in 1995. An updated copy of the EAP was provided to this office on July 10, 2019. The owner shall review, and update as necessary, the dam's EAP in coordination with Washtenaw County Emergency Management. The results of this review, and any updates, should be provided to the Dam Safety Program by December 31, 2022.

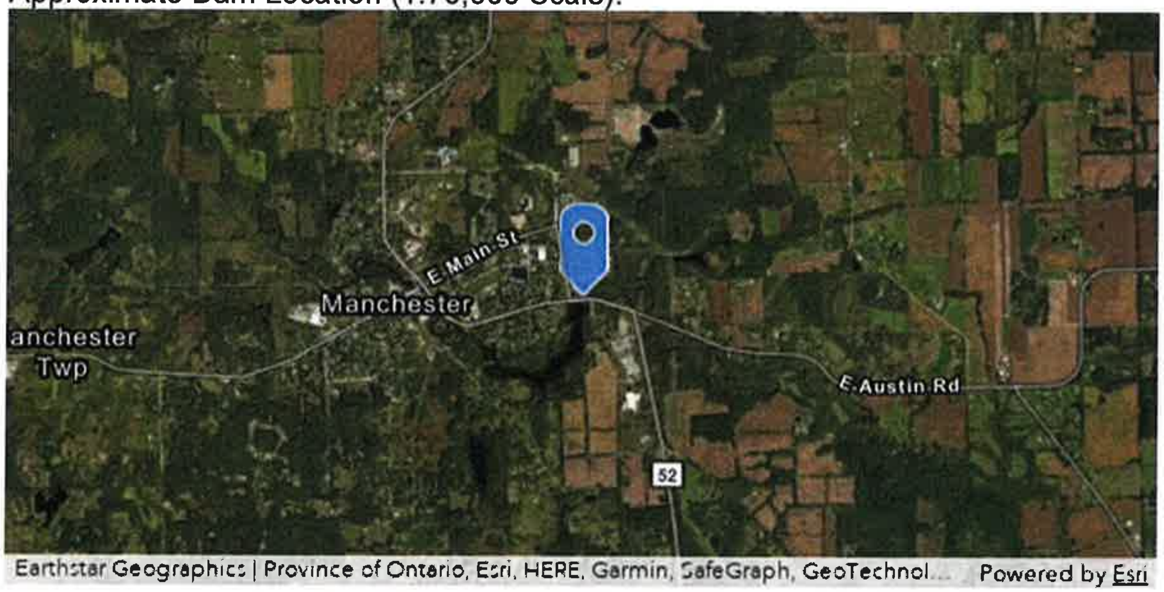
APPENDICES

A location map, inspection photographs, hydraulic calculations, and 2022 EGLE estimated flood flows are attached.

Ford Manchester Dam
Dam ID No. 391
Section 1
T 04S
R 03E
Washtenaw County



Approximate Dam Location (1:70,000 Scale):



Approximate Dam Location (1:40,000 Scale):

Photo #1 - Upstream slope of left embankment at abutment with spillway

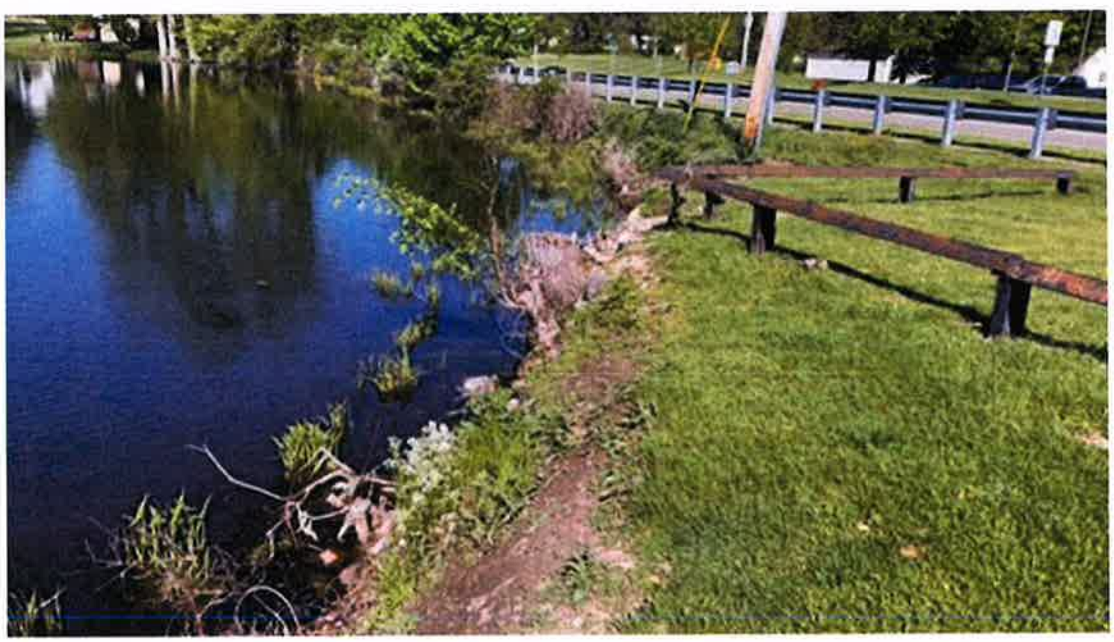


Photo #2 - Upstream slope of left embankment viewed from spillway

Photo #3 - Upstream slope of left embankment. Brush and trees present

Photo #4 - Upstream slope of left embankment. Some trees and brush present on embankment near the waterline

FORD MANCHESTER DAM
WASHTENAW COUNTY
DAM ID NO. 391

Photo #5 - Upstream slope of left embankment

Photo #6 - Upstream slope of left embankment

Photo #7 - Upstream slope of left embankment at embankment end

Photo #8 - Upstream slope of right embankment at abutment with spillway

Photo #9 - Upstream slope of right embankment. Tree and brush present towards end of embankment

Photo #10 - Upstream slope of right embankment. Tree and brush present towards end of embankment

Photo #11 - Crest of left embankment viewed from spillway

Photo #12 - Crest of left embankment viewed from left towards spillway

Photo #13 - Burrow observed under spillway concrete at left end of spillway

Photo #14 - Crest of left embankment; Roadway near village offices

FORD MANCHESTER DAM
WASHTENAW COUNTY
DAM ID NO. 391

Photo #15 - Bridge over spillway outlet

Photo #16 - Crest of right embankment

Photo #17 - Crest of right embankment

Photo #18 - Crest of right embankment

Photo #19 - Crest of embankment viewed from right

Photo #20 - Crest of left embankment in front of village offices

Photo #21 - Crest of left embankment in front of village offices

Photo #22 - Crest of left embankment viewed from left at left end

Photo #23 - Crest/Downstream slope of left embankment in front of village offices

Photo #24 - Crest/Downstream slope of left embankment in front of village offices

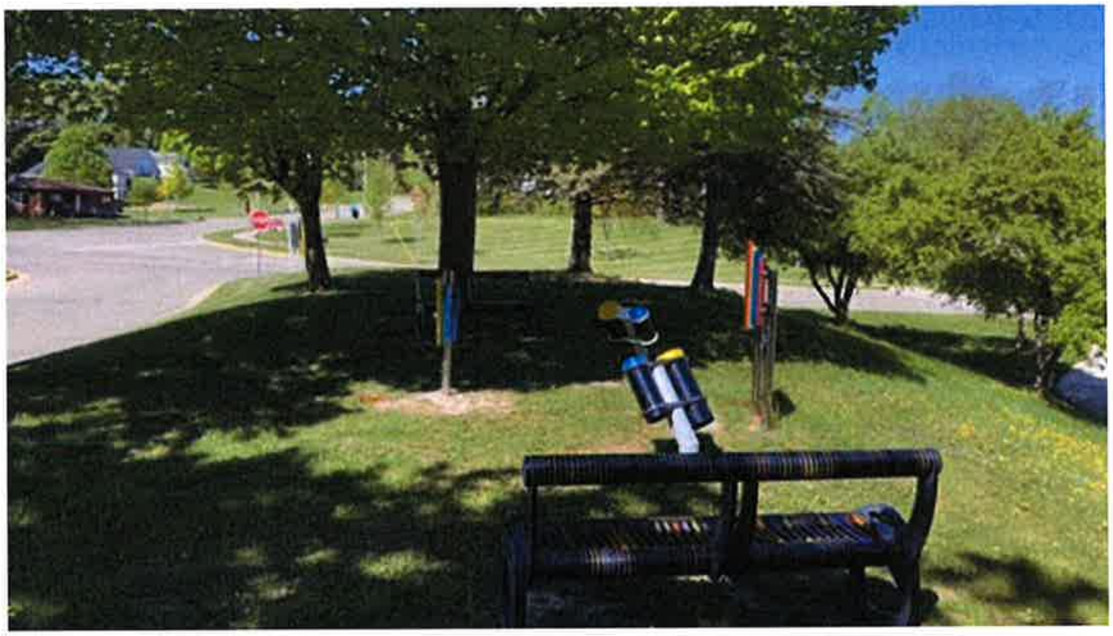


Photo #25 - Crest/Downstream slope of left embankment beside village offices

Photo #26 - Downstream slope of right embankment viewed from crest. Trees and brush encroaching past groin and past toe

Photo #27 - Downstream slope of right embankment viewed from crest. Some brush is present and encroaching upward beyond toe

Photo #28 - Downstream slope of right embankment viewed from crest

Photo #29 - Downstream slope of right embankment viewed from crest

Photo #30 - Downstream slope of right embankment viewed from crest adjacent to spillway

Photo #31 - Storm sewer outfall on right embankment downstream slope

Photo #32 - Storm sewer outfall downstream flow path. Some of the flow path has been armored. Downstream of the armoring in the photo, runoff gullies are present

Photo #33 - Downstream slope of right embankment viewed from near downstream toe

Photo #34 - Downstream slope of the left end of the left embankment

Photo #35 - Downstream slope of left embankment at village offices

Photo #36 - Downstream slope of left embankment on village office grounds

Photo #37 - Spillway crest. Note branch stuck on crest that should be removed

Photo #38 - Left end of spillway crest

Photo #39 - Upstream impoundment viewed from spillway

Photo #40 - Spillway crest viewed from left

Photo #41 - Downstream spillway receiving channel under bridge

Photo #42 - Spillway left upstream abutment wall. Note concrete deterioration of gate operator deck

Photo #43 - Left downstream spillway abutment wall. Efflorescence present throughout. Some minor seepage observed at darker area of concrete

Photo #44 - Left downstream spillway abutment wall at joint with bridge

Photo #45 - Left end of spillway

Photo #46 - Left downstream spillway abutment wall. Efflorescence present. Spalling present at the top of the wall. Cracking and loss of concrete observed near the bottom of the wall

Photo #47 - Cracking and loss of concrete observed near the bottom of the left downstream abutment wall

Photo #48 - Spalling on top of left spillway abutment wall

Photo #49 - Spillway crest

Photo #50 - Some vegetation growing within concrete spillway structure that should be removed

Photo #51 - Concrete deterioration at spillway deck over sluiceway

Photo #52 - Posted security sign

Photo #53 - Spillway crest

Photo #54 - Right upstream spillway abutment wall. Also note deterioration of gate operator deck

Photo #55 - Right downstream spillway abutment wall. Efflorescence present throughout, spalling present at the top of the wall

Photo #56 - Concrete delamination on spillway's right downstream abutment wall

Photo #57 - Right downstream spillway abutment wall at joint with bridge. Efflorescence present throughout, spalling present at the top of the wall

Photo #58 - Spalling on top of the spillway's right downstream abutment wall

Photo #59 - Spalling on top of the spillway's right downstream abutment wall

Photo #60 - Spalling on top of the spillway's right downstream abutment wall and gate operator deck

Photo #61 – Concrete deterioration on spillway's right downstream abutment wall

Photo #62 - Spillway crest viewed from right

Photo #63 - Top of spillway's right upstream abutment wall. Minor spalling observed

Photo #64 - Top of spillway's right upstream abutment wall. Loss of concrete at waterline

Photo #65 - Spillway right abutment wall right face. Spalling present

Photo #66 - Left drawdown gate operator. Note severe deterioration of concrete on operator deck

Photo #67 - Spalling, exposed rebar at left spillway gate operator deck. Guardrail connection to concrete is compromised

Photo #68 - Stoplog grooves at left drawdown gate operator deck

Photo #69 - Right drawdown gate operator

Photo #70 - Right drawdown gate operator. Note severe deterioration of concrete on operator deck

Photo #71 - Spillway crest at right drawdown gate. Drawdown outlet was unable to be observed

Photo #72 - Spillway right downstream abutment wall downstream of bridge

Photo #73 - Spillway right downstream abutment wall downstream of bridge

Photo #74 - Right end of spillway sill downstream of bridge

Photo #75 - Spillway sill downstream of bridge

Photo #76 - Spillway left downstream abutment wall under bridge

Photo #77 - Spillway left downstream abutment wall downstream of bridge. Some efflorescence present. Seepage observed

Photo #78 - Wall drain on left downstream abutment wall

Photo #79 - Seepage in left downstream abutment wall

Photo #80 - Seepage areas on left downstream wall

Photo #81 - Left downstream abutment wall and village offices/historic powerhouse

Photo #82 - Receiving channel left wall at base of previous powerhouse

Photo #83 - Receiving channel left wall at powerhouse outlet

Photo #84 - Receiving channel left wall downstream face

Photo #85 - Receiving channel viewed from bridge

Photo #86 - Spillway left downstream abutment wall viewed from crest in front of village offices

FORD MANCHESTER DAM
WASHTENAW COUNTY
DAM ID NO. 391

Photo #87 - Spillway outlet and sill

Photo #88 - Receiving channel

Photo #89 - Right bank of channel downstream of dam across from the powerhouse. Grouted riprap has become overgrown with trees and brush

Photo #90 - Right bank of channel downstream of dam across from the powerhouse. Grouted riprap has become overgrown with trees and brush

Photo #91 - Spillway outlet viewed from powerhouse outlet deck

Photo #92 - Powerhouse sluiceway inlet. Concrete shows severe spalling at piers

Photo #93 - Loss of concrete at waterline of powerhouse sluiceway inlet, left side

Photo #94 - Loss of concrete at waterline of powerhouse sluiceway inlet, right side

Photo #95 - Previous powerhouse outlet into river. No water currently flows through sluiceway

Photo #96 - Previous powerhouse outlet into river. No water currently flows through sluiceway

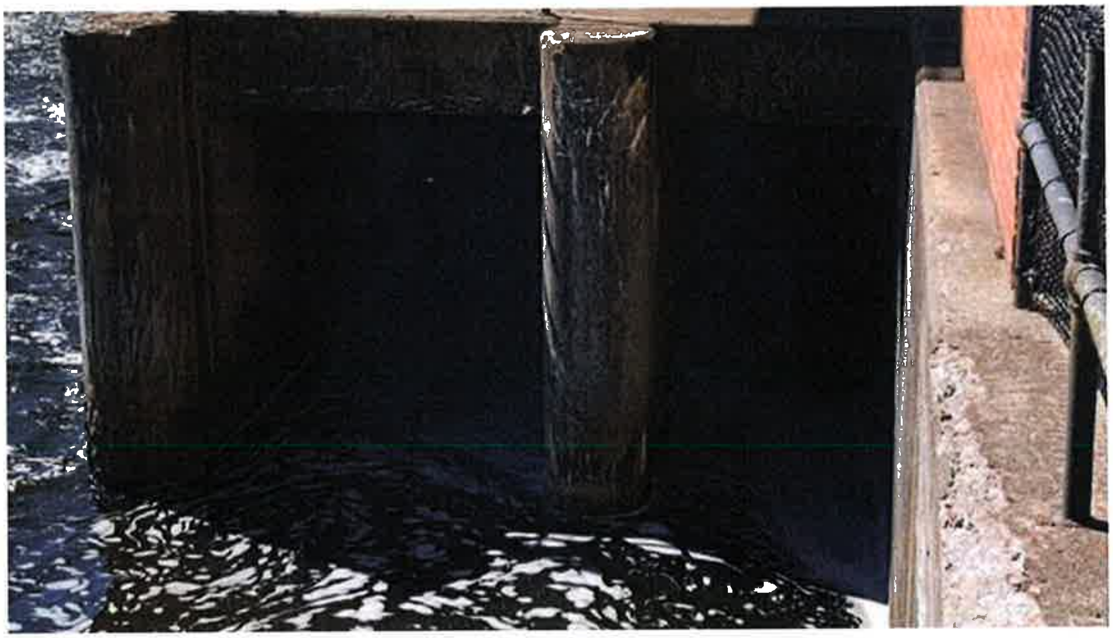


Photo #97 - Powerhouse outlet. Efflorescence present on concrete piers

Photo #98 - Sluiceway inlet adjacent to principal spillway, looking downstream towards powerhouse

Photo #99 - Sluiceway inlet adjacent to principal spillway, looking upstream

Photo #100 - Sluiceway alignment from inlet to powerhouse

Photo #101 - Concrete deck over sluiceway at powerhouse. Sluiceway at this location was not observed during the inspection

Photo #102 - Sluiceway in basement of powerhouse

Photo #103 - Previous location of turbines in powerhouse

Photo #104 - Previous location of turbines in powerhouse

Photo #105 - Village office lobby with historic powerhouse equipment

Photo #106 - Village office lobby with historic powerhouse equipment

Photo #107 - Village office lobby with historic powerhouse equipment

FORD MANCHESTER DAM
WASHTENAW COUNTY
DAM ID NO. 391

HYDRAULIC CALCULATIONS

L	80.5	
C	3.8	
H ft	Q cfs	Freeboard ft
0	0	4.5
0.25	38.2375	4.25
0.5	108.152	4
0.75	198.6879	3.75
1	305.9	3.5
1.25	427.5082	3.25
1.5	561.9742	3
1.75	708.1684	2.75
2	865.2159	2.5
2.25	1032.413	2.25
2.5	1209.176	2
2.65	1319.618	1.85
3	1589.503	1.5
3.25	1792.274	1.25
3.5	2003.003	1
3.75	2221.398	0.75
4	2447.2	0.5
4.25	2680.173	0.25
4.5	2920.104	0

FORD MANCHESTER DAM
WASHTENAW COUNTY
DAM ID NO. 391

HYDROLOGIC DATA

From: EGLE-wrd-qreq <EGLE-wrd-qreq@michigan.gov>
Sent: Wednesday, March 30, 2022 8:49 PM
To: Horak, Thomas (EGLE) <HorakT@michigan.gov>
Subject: RE: flood or low flow discharge request (ContentID - 168812)

We have processed the discharge request submitted by email on March 17, 2022 (Process No. 20220195), as follows:

River Raisin at Ford Manchester Dam, Dam ID 391, Section 1, T4S, R3E, Village of Manchester, Washtenaw County, has a drainage area of 149 square miles. The design discharge for this dam is the 0.5% chance (200-year) flood. The 0.5% chance peak flow is estimated to be 1300 cubic feet per second. (Watershed Basin No. 29 Raisin).

These estimates should be confirmed by our office if an application is not submitted within one year. If you have any questions concerning the discharge estimates, please contact Ms. Susan Greiner, Hydrologic Studies and Floodplain Management Unit, at 517-927-3838, or by email at: GreinerS@michigan.gov.

-----Original Message-----
From: DoNotReply@michigan.gov <DoNotReply@michigan.gov>
Sent: Thursday, March 17, 2022 10:53 AM
To: EGLE-wrd-qreq <EGLE-wrd-qreq@michigan.gov>
Subject: flood or low flow discharge request (ContentID - 168812)

Requestor: Thomas Horak
Company: EGLE
Address: 525 W. Allegan
City: Lansing
Zip: 48933
Phone: 517-231-8594
Date: 2022-03-17
F0.5percent: Yes
ContactAgency: None Selected
ContactPerson:
Watercourse: River Raisin
LocalName:
CountyLocation: Washtenaw
CityorTownship: ?
Section: 01
Town: 04S
Range: 03E
Location: Ford Manchester Dam #391
FFR1: Dam
fpReqEmailAddr: HorakT@michigan.gov

Appendix B

GEI Inspection Photo Log

Ford Manchester Dam Inspection Photos

Date: 09/19/2023 – 09/20/2023
GEI Project No.: 2204052
Client: Village of Manchester

Ford Manchester Dam Inspection Photos

Date: 09/19/2023 – 09/20/2023
GEI Project No.: 2204052
Client: Village of Manchester

Photo No. 1 – Impoundment Looking Upstream

Photo No. 2 – Gravity Spilway Looking Upstream

Photo No. 3 – Upstream of M-52 Brdige Looking Downstream

Photo No. 4 – M-52 Bridge Looking Downstream Toward Powerhouse

Photo No. 5 – Downstream of M-52 Bridge Looking Upstream

Photo No. 6 – Penstock Intake Looking Upstream

Photo No. 7 – Upstream Left Embankment Looking Right

Photo No. 8 – Downstream Left Embankment Looking Right

Ford Manchester Dam Inspection Photos

Date: 09/19/2023 – 09/20/2023
GEI Project No.: 2204052
Client: Village of Manchester

Photo No. 9 – Upstream Right Embankment Looking Right

Photo No. 10 – Downstream Right Embankment Looking Right

Photo No. 11 – Draft Bay Outlet Looking Upstream

Photo No. 12 – Seating Area Downstream of Powerhouse Looking Downstream

Ford Manchester Dam Inspection Photos

Date: 09/19/2023 – 09/20/2023
GEI Project No.: 2204052
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Photo No. 13 – Decommissioned Equipment in Powerhouse Looking Right

Photo No. 14 – Upstream Right Wing Wall and Right Operator Deck Looking Right

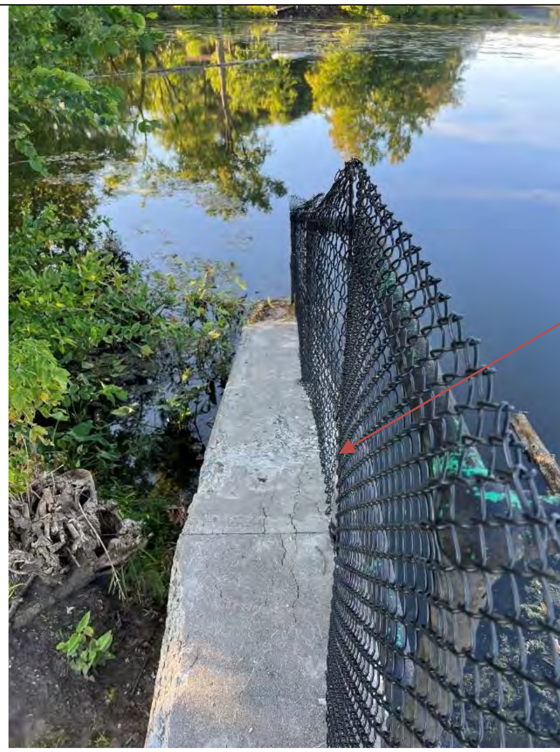

Photo No. 15 – Top of Upstream Right Wing Wall Looking Upstream

Photo No. 16 – Concrete Deterioration on Top of Upstream Right Wing Wall

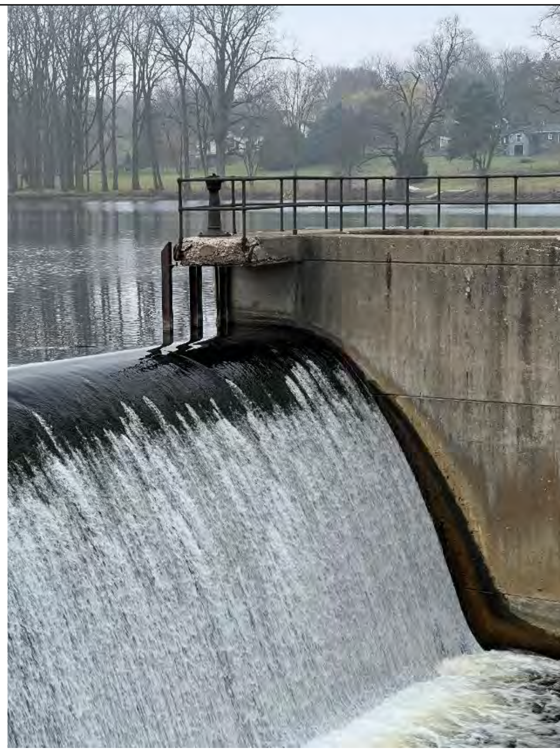

Photo No. 17 – Left Operator Deck Looking Left

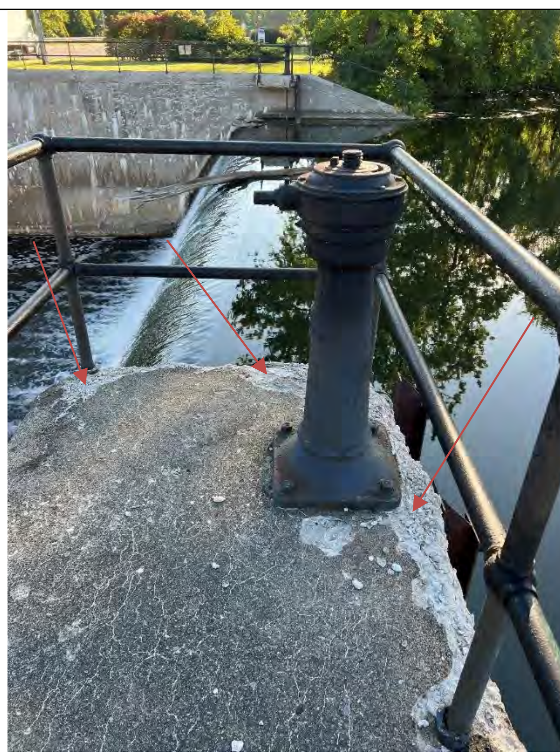

Photo No. 18 – Concrete Deterioration on Right and Upstream Side of Left Operator Deck

Photo No. 19 – Concrete Deterioration on Upstream Side of Left Operator Deck

Photo No. 20 – Right Operator Deck Looking Right

Photo No. 21 – Concrete Deterioration on Downstream Side of Right Operator Deck

Photo No. 22 – Concrete Deterioration on Left and Upstream Side of Right Operator Deck

Photo No. 23 – Concrete Deterioration on Downstream Side of Right Operator Deck

Photo No. 24 – Penstock Intake Looking Right

Photo No. 25 – Penstock Intake Deck Looking Upstream

Photo No. 26 – Right Side of Penstock Deck looking Upstream

Photo No. 27 – Downstream Side of Penstock Operator Deck Looking Upstream Looking Upstream

Photo No. 28 – Concrete Deterioration on Downstream Side of Penstock Intake Deck Looking Upstream

Photo No. 29 – Concrete Deterioration on Downstream Left Corner of Penstock Intake Deck Looking Upstream

Photo No. 30 – Interface of Guardrail and Left Side of Penstock Intake Deck Looking Upstream

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Photo No. 31 – Interface of Left Upstream Face of Penstock Intake and Crest of Left Embankment Looking Upstream

Photo No. 32 – Concrete Deterioration of Left Side of Penstock Intake Deck Looking Right

Photo No. 33 – Concrete Deterioration of Upstream Left Corner of Penstock Intake Deck Looking Upstream

Photo No. 34 – Concrete Deterioration of Upstream Side of Penstock Intake Deck Looking Right

Photo No. 35 – Concrete Deterioration of Right Side of Penstock Intake Deck Looking Downstream

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Photo No. 44 – Downstream Left Wall Upstream of M-52 Bridge Looking Upstream

Photo No. 45 – Top of Downstream Left Wall Upstream of M-52 Bridge Looking Upstream

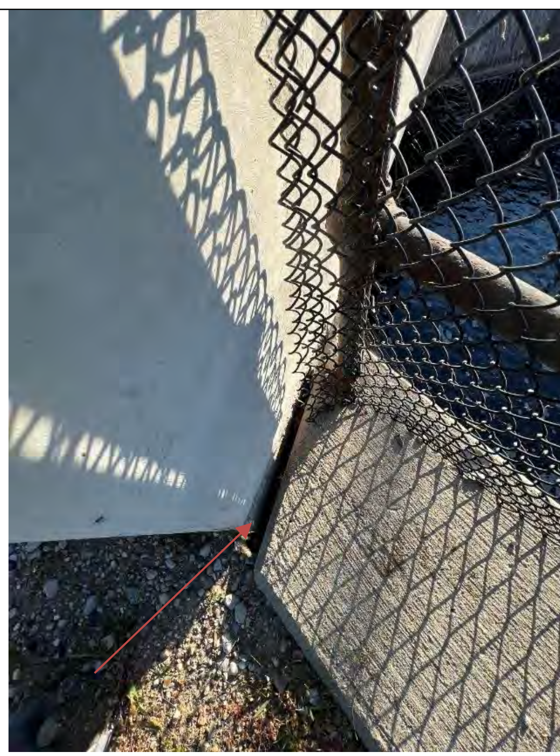

Photo No. 46 – Upstream Interface of Downstream Left Wall and M-52 Bridge

Photo No. 47 – Concrete Deterioration on Downstream Left Wall Upstream of M-52 Bridge Looking Left

Photo No. 48 – Downstream Left Wall Downstream of M-52 Bridge Looking Downstream

Photo No. 50

Photo No. 49 – Concrete Deterioration and Leaking at Construction Joint on Downstream Left Wall Downstream of M-52 Bridge

Photo No. 50 – Concrete Deterioration and Leaking at Construction Joint on Downstream Left Wall Downstream of M-52 Bridge

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Photo No. 53 – Concrete Deterioration at Top of Downstream Left wall Upstream of Powerhouse Looking Right

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Photo No. 55 – Concrete Deterioration at Top of Downstream Left Wall Downstream of M-52 Bridge Looking Right

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Photo No. 57 – Concrete Deterioration at Top of Downstream Left Wall Downstream of M-52 Bridge Looking Downstream

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Photo No. 60 – Top of Downstream Left Wall Downstream of M-52 Bridge Looking Downstream

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Photo No. 63 – Concrete Deterioration on Top of Downstream Left Wall Upstream of Powerhouse Looking Downstream

Photo No. 64 –Downstream Right Wall Upstream of M-52 Bridge Looking Right

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Photo No. 65 – Top of Downstream Right Wall Upstream of M-52 Bridge Looking Right

Photo No. 66 – Concrete Deterioration at Top of Downstream Right Wall Upstream of M-52 Bridge Looking Right

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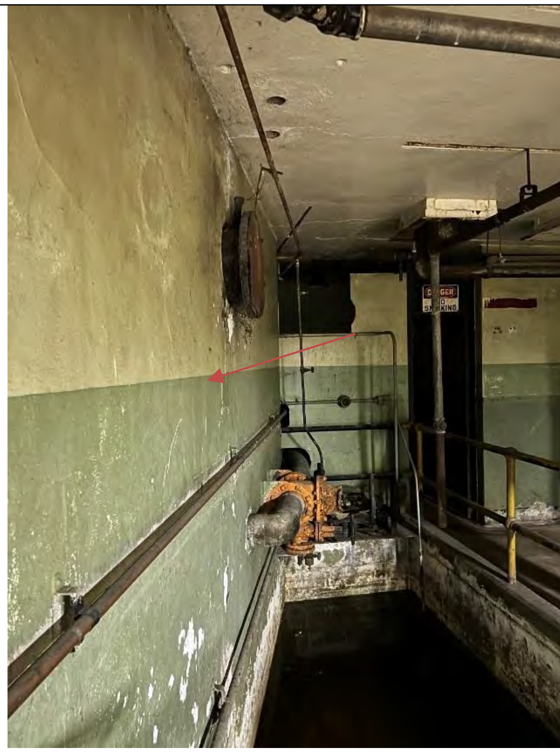

Photo No. 83 – Basement Wall and Right Draft Bay Wall Looking Upstream

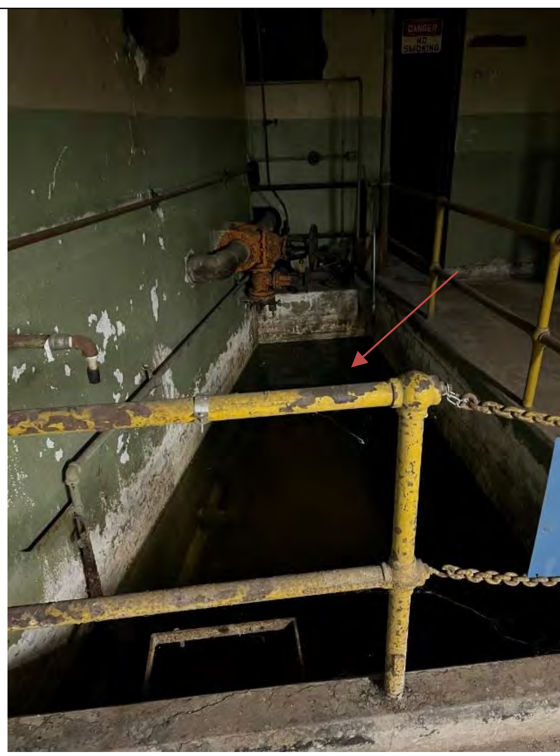

Photo No. 84 – Pit in Basement Adjacent to Right Draft Bay Wall Looking Upstream

Photo No. 85 – Top of Seating Area Wall Looking Upstream

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Photo No. 90 – Concrete Deterioration at Top of Seating Area Wall Looking Left

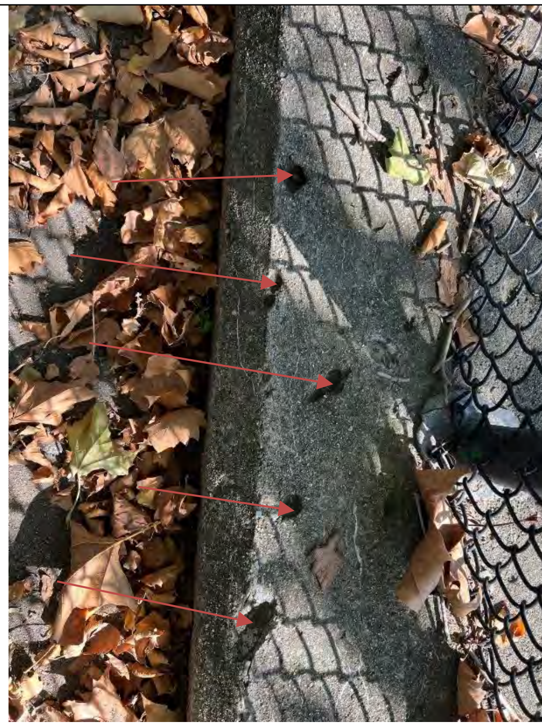


Photo No. 91 – Pitting in Top of Concrete Seating Area Wall

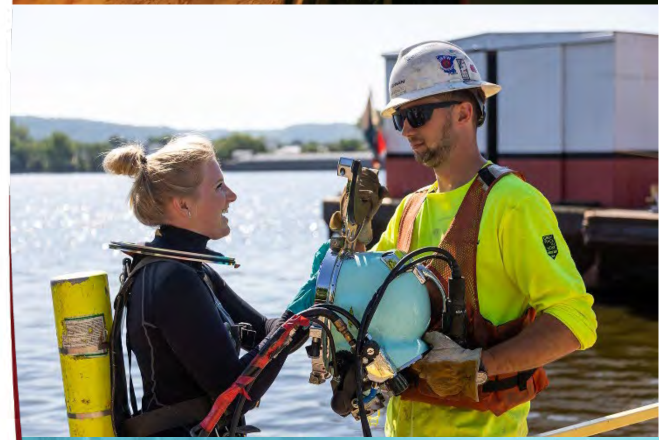
Appendix C

J.F. Brennan Inspection Report

Marine Professionals Since 1919

BRENNAN

UNDERWATER DIVE INSPECTION REPORT



Prepared for:

GEI Consultants

GEI Consultants

Structure: **Manchester Dam**
Location: **Manchester, MI**
Inspection Dates: **September 19-20, 2023**

Prepared by:

J.F. Brennan Company, Inc.
818 Bainbridge Street
La Crosse, WI 54603
Phone: 608.784.7173
jfbrennan.com

Table of Contents

Executive Summary

Project:	Underwater general condition assessment of the Manchester Dam in Manchester, Michigan.
Scope of Work:	Inspection included an overall structure condition assessment of the downstream and upstream sections of the Manchester Dam.
Inspection Team:	Dive Supervisor: Eric Hanson Diver/Tender: Shane Monahan Diver/Tender: Derek Pratt
Inspection Dates:	September 19-20, 2023
Weather:	Cloudy, 70°F
Water Visibility:	Fair, 1-foot.
Coordinates:	42.149570°, -84.023590°
Dive Mode:	Surface Supplied Air via Boat.
Condition Assessment:	Fair.
Brennan Repair Rating:	Low.

Summary of Findings:

- Multiple areas of spalling were present along the downstream portion of the dam.
- An area of undermining was present on the downstream face of the spillway.
- A spalled out vertical construction joint, with exposed rebar, was present on the west wall.
    - A small amount of water was observed to be leaking from the joint.
    - In the intake tunnel, the construction joint that was the source of the leaking water was located.
- Hairline cracks were present on the east wall of the intake tunnel.
- The upstream bullnoses of the intake bay experienced areas of spalling and scaling.

Summary of Recommendations:

- Repair the area of undermining on the downstream face of the spillway.
- Repair the leaking construction joint in the intake tunnel.
- Monitor the areas of spalling present throughout the downstream portion of the dam.
- Monitor the hairline cracks present in the intake tunnel.
- Monitor the areas of spalling present on the upstream bullnoses.

1. Introduction/Background

J.F. Brennan Company, Inc. (Brennan) performed an inspection of the exterior surfaces on the underwater portions of the downstream and upstream sections of the Ford Manchester Dam. Environmental conditions, such as channel bed material, biological growth, and drift/debris, were generally noted.

Structure Data:

Owner:	Village of Manchester
Structure:	Manchester Dam
Location:	Manchester, Michigan
Waterway:	River Raisin

2. Method of Investigation

A Level I visual and tactile inspection of the structure and surrounding riverbed was used to observe signs of distress and deterioration including, but not limited to: movement, cracks, scaling, spalling, honeycombing, scour, and undermining.

The crew and equipment accessed the structure using a mobile platform. The inspection was conducted using surface-supplied air with equipment including a Kirby Morgan dive helmet with full diver-to-surface communications; and a helmet-mounted Outland Video Camera / Light combo with a video recorder providing live streaming at the dive platform.

All dives were conducted in accordance with Brennan's Safe Diving Practices Manual as well as all pertinent ADCI, OSHA, and USCG regulations. Additionally, all dives adhered to the dive schedules and decompression tables outlined in the U.S. Navy Dive Manual, Rev. 7.

All measurements referenced hereinafter were approximate and reflect the conditions on-site at the time of the inspection.

The three (3) levels of underwater inspections are described as:

Level I - A simple visual or tactile (by feel) inspection, without the extensive use of tools or measuring devices. It is usually employed to gain an overview of the structure and will precede or verify the need for a more detailed Level II or Level III inspection.

Level II - A detailed inspection which involves physically cleaning or removing growth from portions of the structure. In this way, hidden damage may be detected and assessed for severity. This level is usually performed on at least a portion of a structure, supplementing a Level I.

Level III - A highly detailed inspection of a structure which is warranted if extensive repair or replacement is being considered. This level requires extensive cleaning, detailed measurements, and testing techniques that may be either destructive or non-destructive in nature.

3. Inspection Findings

*To view/download the footage from the inspection please follow the instructions below. The SharePoint site will remain active for 30 days, during this period please download the files if you want to keep them for your record. After the 30-day period, the site will be removed, and you will no longer be able to access the videos through the SharePoint link.

- Manchester Dam (Click and follow link directly. Your email address must have been given access for you to open up the folder. If you do not have access and need it, please reach out so we can get your email address added.)

Downstream of Spillway

- **Station 1:** An area of spalling was present 5-inches above waterline on the east wall (See 'Appendix A, Figure 5').
  - The spalling measured 3.5-inches wide by 11-inches tall with 1-inch of loss.
- **Station 2:** An area of delamination was present 30-inches above waterline on the east wall near the spillway (See 'Appendix A, Figure 6').
  - The delamination measured 62-inches wide by 57-inches tall with 1.25-inches of loss.
- **Station 3:** An area of undermining was present along the spillway.
  - The undermining measured 4-inches wide by 8-inches tall with 8-inches of loss.
- **Station 4:** An area of spalling was present 36-inches above waterline on the west wall (See 'Appendix A, Figure 9').
  - The spalling measured 14.5-inches wide by 3-inches tall with 3-inches of loss.
- **Station 5:** An area of spalling with exposed rebar was present on the west wall (See 'Appendix A, Figures 10-11').
  - The spalling measured 52-inches wide by 83-inches tall with 7.5-inches of loss.
- **Station 6:** A spalled out vertical construction joint with exposed rebar was present on the west wall (See 'Appendix A, Figures 12-14').
  - A small amount of water was observed to be leaking from the joint.
  - The spalling extended from the top of the wall down to bedrock.
  - The spalling measured 28-inches wide with up to 6-inches of loss.

Downstream of Building

- The concrete was in satisfactory condition with areas of light scaling present throughout.
- No areas of undermining were present.
- Bay 1:
  - Bullnose: The concrete was in satisfactory condition with light scaling present.
  - Left Stoplog Groove: The concrete was in satisfactory condition.
    - A broken timber was present on bottom.
  - Right Stoplog Groove: The concrete was in satisfactory condition.
    - A broken timber was present on bottom.
  - Draft Tube: The draft tube experienced heavy rust and light delamination (See 'Appendix A, Figure 30').
- Bay 2:
  - Bullnose: The concrete was in satisfactory condition with light scaling present.
  - Left Stoplog Groove: The concrete was in satisfactory condition.
    - A broken timber was present on bottom.
  - Right Stoplog Groove: The concrete was in satisfactory condition.
    - A broken timber was present on bottom.
  - Draft Tube: The draft tube experienced heavy rust and light delamination (See 'Appendix A, Figure 31').

Downstream, Intake Tunnel (ROV Inspection)

- Trash Rack: The trash rack experienced moderate to heavy marine growth with 80 to 90% coverage (See 'Appendix A, Figure 19').
- West Wall: The concrete was in satisfactory condition.
- Two gates were present on the downstream end of the tunnel (See 'Appendix A, Figures 20-21').
  - The gates experienced light to moderate rust throughout.

- East Wall:
    - **Station 7:** A construction joint that was the source of the leaking water at Station 6 was present 15-feet upstream of the gates (See 'Appendix A, Figures 22-23').
        - An area of spalling was present on the ceiling (See 'Appendix A, Figures 24-25').
    - **Station 8:** A hairline crack was present on the east wall approximately 27-feet upstream of the gates (See 'Appendix A, Figure 26').
        - The crack extended from the bottom to the top of the wall.
    - **Station 9:** A hairline crack was present on the east wall approximately 40-feet upstream of the gates (See 'Appendix A, Figure 27').
        - The crack extended from the bottom to the top of the wall.

Downstream of Gates (ROV Inspection)

- West Wall: The concrete appeared to be in satisfactory condition.
- A large amount of sediment was present throughout.
- The turbine appeared to be in satisfactory condition (See 'Appendix A, Figures 28-29').

Upstream, Spillway

- East Wingwall: The concrete was in satisfactory condition with light scaling present throughout.
- East Sluice Gate: The trash rack was present with heavy rust present (See 'Appendix A, Figure 32').
    - The guides were in place and experienced heavy rust.
- Spillway: The concrete was in satisfactory condition with light scaling present throughout.
    - Moderate marine growth was present throughout.
    - No areas of undermining were found.
- West Sluice Gate: The trash rack was present with heavy rust present (See 'Appendix A, Figure 33').
    - The guides were in place and experienced heavy rust.

Upstream, Intake Bay

- Bullnose 1, **Station 10**: An area of spalling was present at the freeze/thaw line (See 'Appendix A, Figure 34').
    - The spalling measured 36-inches wide by 27-inches tall with 4-inches of loss.
- Right Stoplog Groove: The concrete was in satisfactory condition.
- Trash Rack: The trash rack experienced moderate to heavy marine growth with 80 to 90% coverage (See 'Appendix A, Figure 35').
    - The bars experienced moderate to heavy rust throughout.
    - No knife edging was present.
- Sill: Up to 3-feet of debris and sediment was present on the sill.
- Left Stoplog Groove: The concrete was in satisfactory condition.
    - A large amount of debris was present in the bottom of the groove.
- Bullnose 2, **Station 11:** An area of spalling was present at the freeze/thaw line (See 'Appendix A, Figure 36').
    - The spalling measured 36-inches wide by 22-inches tall with 6-inches of loss.
    - **Station 12:** An area of scaling was present 7-inches below waterline (See 'Appendix A, Figure 37').
        - The scaling measured 10-inches wide by 16-inches tall with 2-inches of loss.

4. Evaluation and Summary

Based on the dive inspection, Ford Manchester Dam was considered to be in fair condition. Limited minor to moderate defects or deterioration were observed, with localized areas of moderate to advanced deterioration present.

Refer to ‘Routine Underwater Condition Assessment Rating Descriptions’ below for explanations of above noted condition rating(s).

5. Recommendations

It is recommended that the following areas be repaired: the area of undermining present on the downstream face of the spillway, and the leaking construction joint in the intake tunnel.

It is also recommended that the areas of spalling and hairline cracks be periodically monitored to determine if further deterioration has occurred.

Brennan recommends that the entire underwater section of the facility, be inspected within a 60-month maximum interval. An immediate post-event inspection should be conducted on the structure after any significant or unusual event, including, but not limited to: flood, earthquake, storm, vessel impact, or other event that has potential to cause damage to the structure. Drift and debris material should be cleared to prevent scour and undermining or any further damage to the structure.

Routine Underwater Condition Assessment Rating Descriptions

Good: No visible or only minor damage was noted. Structural elements may have shown very minor deterioration, but no overstressing was observed. No repairs were recommended.

Satisfactory: Limited minor to moderate defects or deterioration were observed, but no overstressing was observed. The "Brennan Repair Rating" was low.

Fair: All primary structural elements were sound, but minor to moderate defects or deterioration were observed. Localized areas of moderate to advanced deterioration may have been present but did not significantly reduce the load-bearing capacity of the structure(s). The "Brennan Repair Rating" was low to moderate.

Poor: Advanced deterioration or overstressing was observed on widespread portions of the structure(s) but did not significantly reduce the load-bearing capacity of the structure(s). The "Brennan Repair Rating" was moderate.

Serious: Advanced deterioration, overstressing or breakage, may have significantly affected the load-bearing capacity of primary structural components. Localized failures are possible and load bearing restrictions may be necessary. The "Brennan Repair Rating" was moderate to major.

Critical: Heavily advanced deterioration, overstressing or breakage, has resulted in localized failure(s) of primary structural components. More widespread failures are possible or likely to occur, and load restrictions should be implemented as necessary. The "Brennan Repair Rating" was major.

Brennan Repair Rating Descriptions

Low: Did not significantly reduce the load-bearing capacity or functionality of the structure(s). Repairs may be recommended, but the priority of the repairs is low. At a minimum, continue to monitor with future inspections.

Moderate: Load-bearing capacity of the structure was not in immediate danger, but moderate to advanced deterioration was observed. The observed deterioration could affect the overall functionality of the structure(s). Priority of repair is moderate, repair plans should be put in place and executed when possible.

Major: Load-bearing capacity of the structure was affected and/or failures have already occurred. Load restrictions should be put in place until repairs have been made. Priority of repair is urgent, and repairs should be made as soon as possible.

We appreciate the opportunity to work with the Village of Manchester on this project. If you have any questions or concerns regarding the information in this report or if Brennan can be of any further assistance, please do not hesitate to contact me directly.

Respectfully submitted,

Justin Brendon
Underwater Services – Assistant Dive Project Technician
cell 608.799.1978
jbrendon@jfbrennan.com

Appendix A – Photographs

List of Figures

Figure 1 – Area Map

Figure 2 – Downstream, Spillway

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Figure 4 – Downstream, East Wall

Figure 5 – Downstream, East Wall: Spalling

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Figure 7 – Downstream, West Wall

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Figure 9 – Downstream, West Wall: Spalling

Figure 10 – Downstream, West Wall: Spalling with Exposed Rebar

Figure 11 – Downstream, West Wall: Spalling with Exposed Rebar

Figure 12 – Downstream, West Wall: Spalled Out Construction Joint

Figure 13 – Downstream, West Wall: Spalled Out Construction Joint: Exposed Rebar

Figure 14 – Downstream, West Wall: Spalled Out Construction Joint

Figure 15 – Downstream, Bays 1 and 2

Figure 16 – Upstream, East Sluice Gate

Figure 17 – Upstream, Spillway

Figure 18 – Upstream, West Sluice Gate and Intake Tunnel

Figure 19 – Upstream, Intake Tunnels: Trash Rack

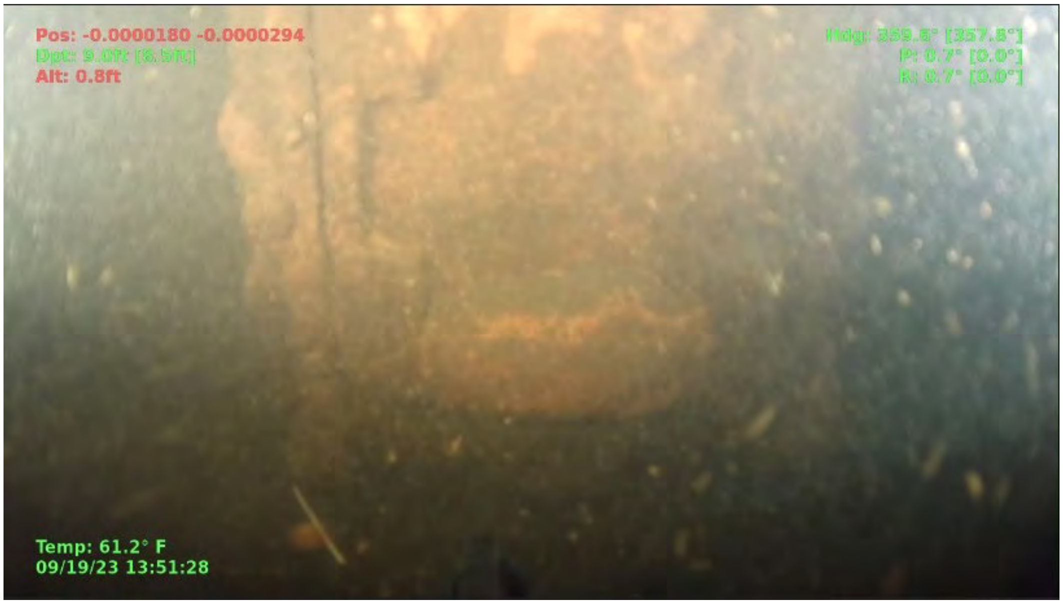

Figure 20 – Upstream, Intake Tunnels: Gate 2

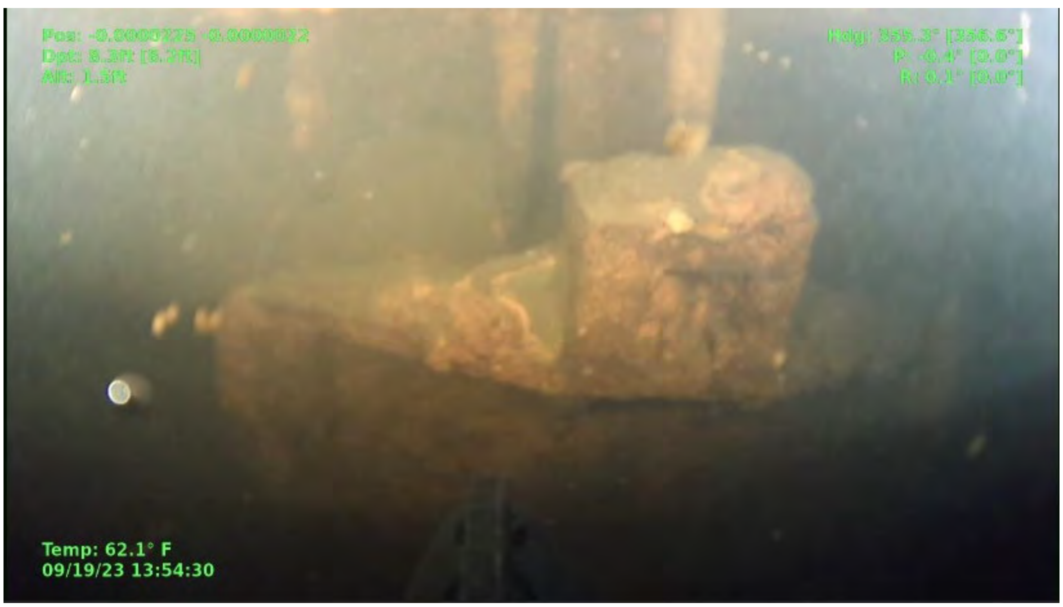

Figure 21 – Upstream, Intake Tunnels: Gate 1

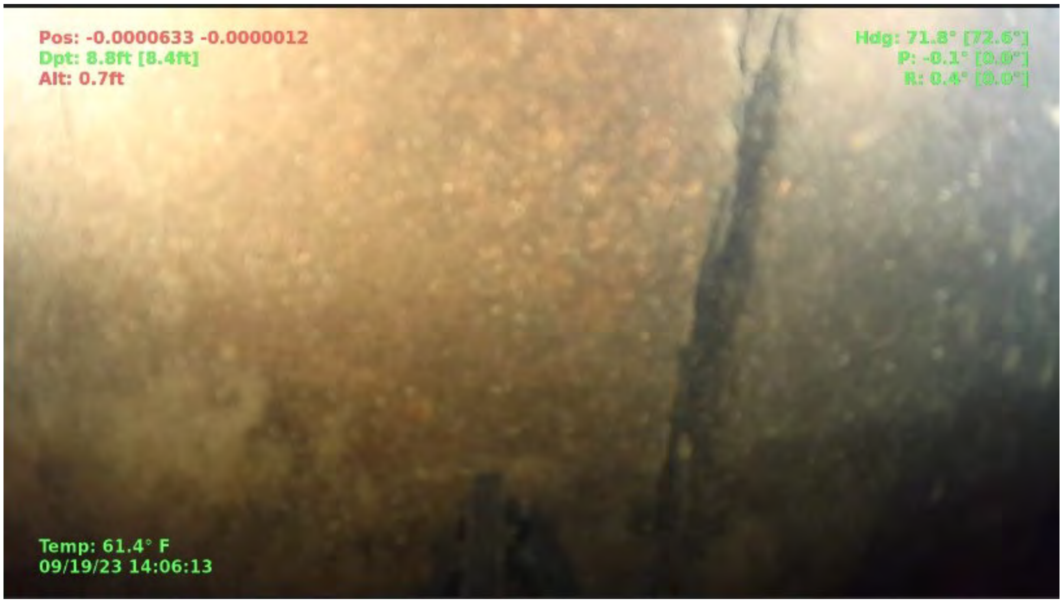

Figure 22 – Upstream, Intake Tunnels East Wall: Construction Joint

Figure 23 – Upstream, Intake Tunnels East Wall: Construction Joint

Figure 24 – Upstream, Intake Tunnels East Wall: Spalling on Ceiling

Figure 25 – Upstream, Intake Tunnels East Wall: Spalling on Ceiling

Figure 26 –Upstream, Intake Tunnels East Wall: Hairline Crack

Figure 27 –Upstream, Intake Tunnels East Wall: Hairline Crack

Figure 28 – Downstream of Gates, Turbine

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Figure 30 – Downstream, Bay 1 Draft Tube

Figure 31 – Downstream, Bay 2 Draft Tube

Figure 32 – Upstream, East Sluice Gate: Trash Rack

Figure 33 – Upstream, West Sluice Gate: Trash Rack

Figure 34 – Upstream, Intake Tunnel, Bullnose 1: Concrete Loss

Figure 35 – Upstream, Intake Tunnel: Trash Rack

Figure 36 – Upstream, Intake Tunnel Bullnose 2: Concrete Loss

Figure 37 – Upstream, Intake Tunnel Bullnose 2: Scaling

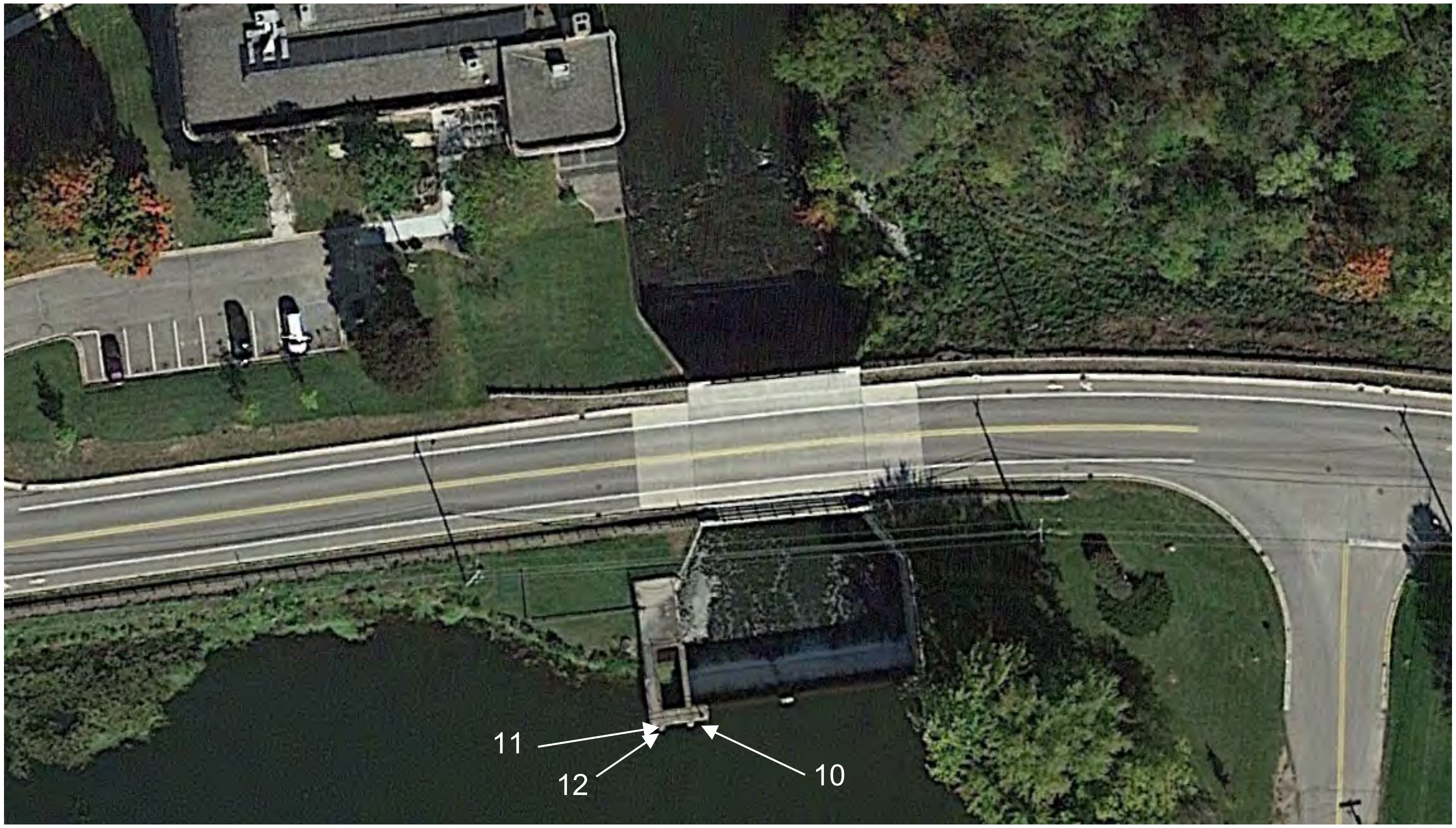
Appendix B – Station References

List of Drawings

- Stationing 1: Manchester Dam: Downstream
- Stationing 2: Manchester Dam: Upstream

Stationing 1 – Cedar Falls Dam: Downstream

Station	Deficiency Type	Width	Height	Loss	Comments
1	Spalling	3.5”	11”	1”	3.5-inches above waterline, on east wall
2	Delamination	62”	57”	1.25”	30-inches above waterline, on east wall
3	Undermining	4”	8”	8”	Along the face of the Spillway
4	Spalling	14.5”	3”	3”	36-inches above waterline, on west wall
5	Spalling – Exposed Rebar	52”	83”	7.5”	West wall
6	Construction Joint Spalling – Exposed Rebar	28”	Entire Wall	6”	West wall, was leaking small amount of water
7	Construction Joint	N/A	N/A	N/A	Intake tunnel, source of leaking water at Station 6
8	Hairline Crack	N/A	N/A	N/A	Intake Tunnel, East Wall 27-feet upstream of the gates
9	Hairline Crack	N/A	N/A	N/A	Intake Tunnel, East Wall 40-feet upstream of the gates



Stationing 1 – Cedar Falls Dam: Upstream

Station	Deficiency Type	Width	Height	Loss	Comments
10	Spalling	36”	27”	4”	At waterline, Bullnose 1
11	Spalling	36”	22”	6”	At waterline, Bullnose 2
12	Scaling	10”	16”	2”	7-inches below waterline, Bullnose 2

Appendix D

Stability Analysis

GEI Consultants	**CALCULATION COVER PAGE**	**Page**	
		Rev. No.	
	Client	City of Manchester	
	Project	Ford Manchester Dam	

GEI Project No.	2204052	**Document No.**	N/A
Calculation Title	Overflow Spillway Stability Analysis		

Summary

The overflow spillway stability was analyzed as a conventional global stability analysis based on a rigid, two-dimensional gravity section with loads taken across a 1-foot unit width. Sliding stability was analyzed using the shear friction factor (SFF) of safety method, assuming zero cohesion at the concrete / foundation interface, in general accordance with EM1110-2-2100 Stability Analysis of Concrete Structures (Ref. USACE, 2005). No conventional record global stability analyses exist for the spillway structure.

The analyzed section was found to satisfy stability criteria for all analyzed load cases.

Signature Block & Record of Revisions

Rev.	Description	Code	Pages/Sections	Name	Date
0	First Issue	P	All	**P. Grodecki**	**11/27/2023**
		R	All	**E. Baffoe**	**11/30/2023**
		A	All	**M. Guirguis**	**12/01/2023**

Codes: P = Prepared; R = Reviewed, A = Approved

GEI Consultants	Client	City of Manchester			Page	
	Project	Ford Manchester Dam			Pg. Rev.	0
	By	P. Grodecki	Chk.	E. Baffoe	App.	M. Guirguis
	Date	11/27/2023	Date	11/30/2023	Date	12/01/2023
Project No.	2204052	Document No.	N/A			
Subject	Overflow Spillway Stability Analyses					

Attachments

Figure

Load Case Computations

Centroids

Headwater and Tailwater Levels

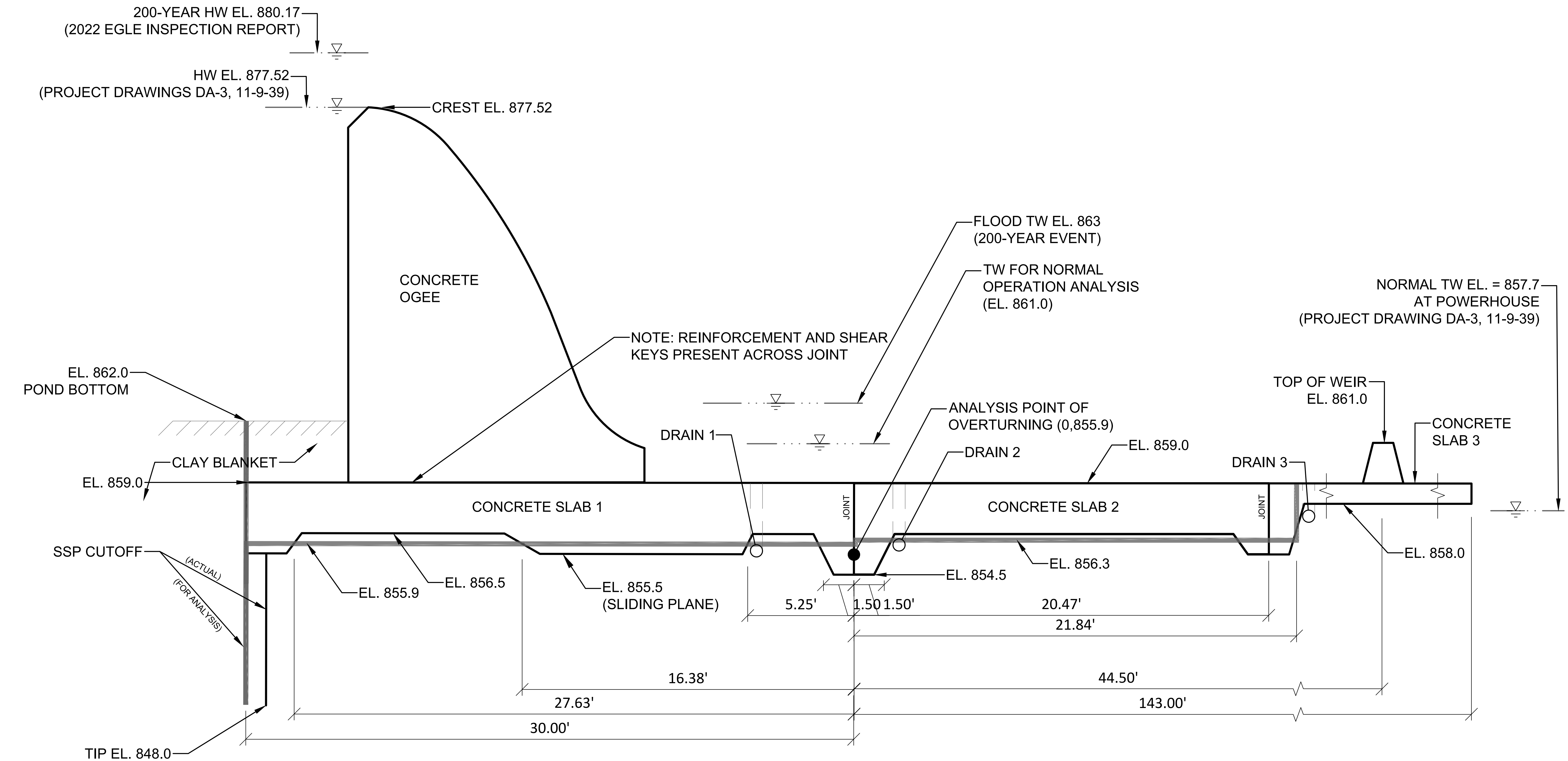
Other Computations

Ultimate Bearing Capacity

Stability Analysis References

- (USACE, 1978) U.S. Army Corps of Engineers (USACE), *River Raisin Basin Ford Manchester Dam, Inspection Report, National Dam Safety Program.* Dated September 1978.
- (FEMA, 2012) Federal Emergency Management Agency (FEMA), *Flood Insurance Study No. 26161CV001A*. Dated April 3, 2012.
- (EGLE, 2022) Department of Environment, Great Lakes, and Energy (EGLE), *Dam Safety Inspection Report Ford Manchester Dam*. Report Date August 4, 2022.
- (FORD, 1939) Ford Motor Co., *Design Drawings*. Dated 1939.
- (USACE, 2005) USACE, *EM1110-2-2100 – Stability Analysis of Concrete Structures, December 1, 2005.*
- (FERC, 2016), Federal Energy Regulatory Commission (FERC), *Engineering Guidelines for the Evaluation of Hydroelectric Projects*, Chapter 3 –Gravity Dams, March 4, 2016.

FIGURE



LEGEND:

STRUCTURE OUTLINE PER PROJECT DRAWINGS

ANALYSIS BASE GEOMETRY

NOTES:

-FOUNDATION DRAINS ARE 6" DIAMETER AND DRAIN THROUGH TOP OF CONCRETE SLABS

- All ELEVATIONS ARE REFERENCED IN NGVD29

BY: P. Grodecki 11/27/2023
CHK: E. Baffoe 11/30/2023
APP: M. Guirguis 12/01/2023

SOURCE:

1. SECTION THROUGH SPILLWAY SHOWING WEST ABUTMENT FOR PLANT. FORD MOTOR COMPANY. DRAWING NUMBER SH-DC4-DATE: 11/9/1939
2. PLAN OF PIPE RAILINGS AT INTAKE & WEST RETAINING WALLS, FORD MOTOR COMPANY, DRAWING NUMBER SH-DC4-DATE:11/9/1939

0 5 10
SCALE, FEET

Inspection and Repair Report Ford Manchester Dam Manchester, MI	GEI Consultants	OVERFLOW SPILLWAY STABILITY ANALYSIS SECTION
Village of Manchester Manchester, MI	Project 2204052	November 2023

LOAD CASE COMPUTATION

GEI Consultants	Client	City of Manchester			Page	
	Project	Ford Manchester Dam			Pg. Rev.	0
	By	P. Grodecki	Chk.	E. Baffoe	App.	M. Guirguis
	Date	11/27/2023	Date	11/30/2023	Date	12/01/2023
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Subject	Overflow Spillway Stability Analyses					

SLIDING STABILITY AND FLOTATION SAFETY FACTORS

SLIDING STABILITY SENSITIVITY TO FOUNDATION FRICTION ANGLE

	LOADS								
	VERT.	**HORIZ.**	**Friction Angle vs Sliding Factor of Safety:**						
LOAD CASE:	(KIPS)	(KIPS)	**32 deg.**	**33 deg.**	**34 deg.**	**35 deg.**	**36 deg.**	**37 deg.**	**38 deg.**
NORMAL	41.9	16	1.6	1.7	1.8	1.8	1.9	**2**	2
NORMAL + ICE	41.9	21	1.2	1.3	1.3	1.4	1.4	**1.5**	1.6
FLOOD	39.9	19.6	1.3	1.3	1.4	1.4	1.5	**1.5**	1.6

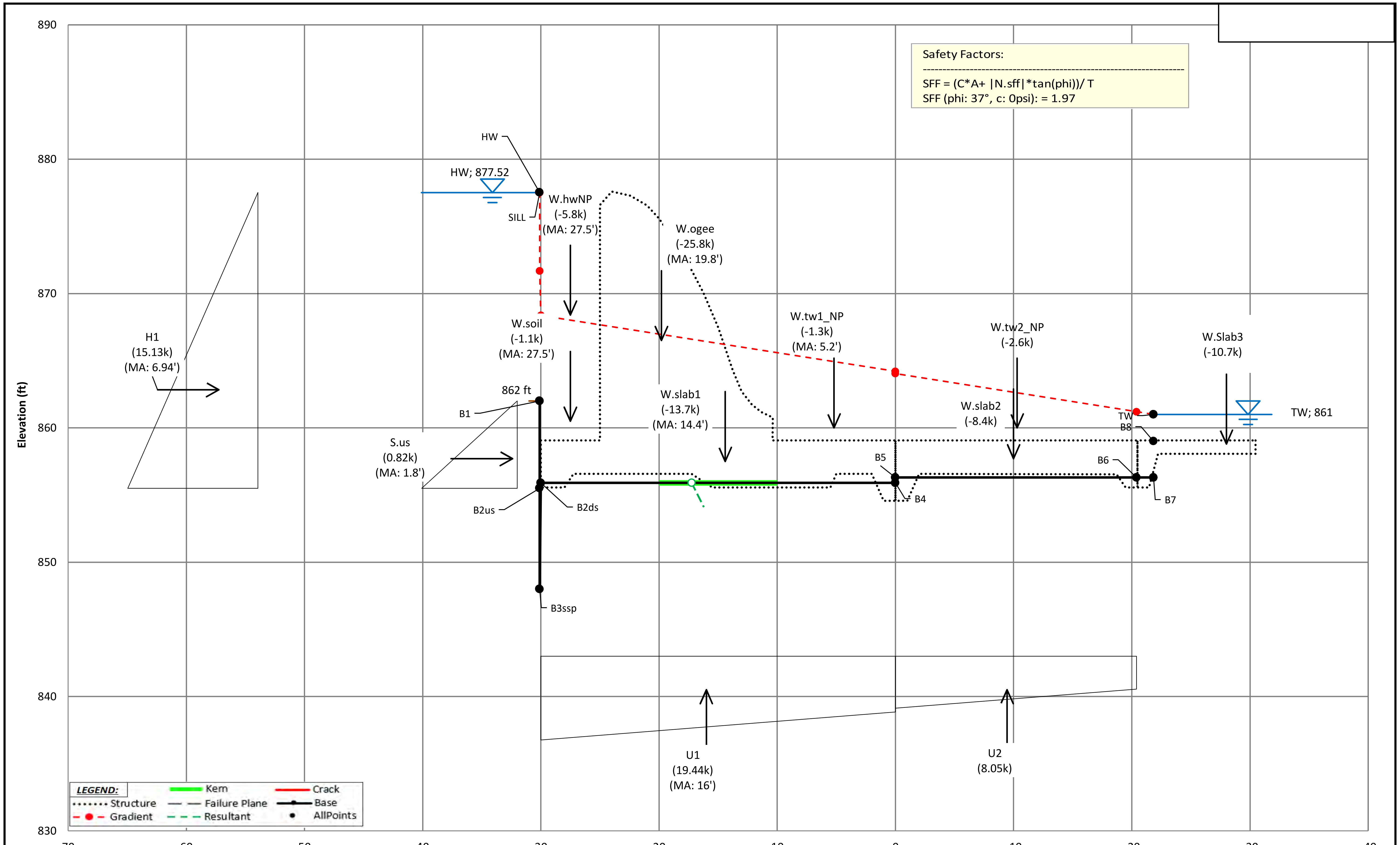
Value used for record analysis

SAFETY FACTOR AGAINST FLOTATION

LOAD CASE:	**UPLIFT (KIP)**	**WEIGHTS* (KIP)**	**FLOTATION SAFETY FACTOR (= WEIGHTS/KIPS)**
NORMAL	27.49	58.7	**2.1**
NORMAL + ICE	27.49	58.7	**2.1**
FLOOD	34.27	63.5	**1.9**

Notes: * Ignores the weight of slab #3

STABILITY - NORMAL POOL CONDITION



Point of Rotation: (0, 855.9)

Notes:

BY: P. Grodecki 11/27/2023
CHK: E. Baffoe 11/30/2023
APP: M. Guirguis 12/01/2023

Force & Moment Orientation	OVERFLOW SPILLWAY SPILLWAY STABILITY	GEI Consultants	CASE I NORMAL OPERATION STABILITY SUMMARY DIAGRAM
	City of Manchester	Project: 2204052	December 2023

GEI Consultants	Client	City of Manchester			Page	
	Project	Ford Manchester Dam			Pg. Rev.	0
	By	P. Grodecki	Chk.	E. Baffoe	App.	M. Guirguis
	Date	11/27/2023	Date	11/30/2023	Date	12/01/2023
Project No.	2204052	Document No.	N/A			
Subject	Overflow Spillway Stability Analyses					

CASE I - RESULTS SUMMARY

LOAD CASE: Overflow Spillway - Case I: Normal Operation

Headwater Elevation (ft): 877.52
Tailwater Elevation (ft): 861.0

Force and Moment Calculation Summary Table

Force Label	Vertical Force (kip)	Horiz. Force (kip)	Moment Arm (ft)	Moment (kip*ft)	Comments
W.ogee	-25.8	--	19.8	-510.84	Weight of Concrete Ogee
W.slab1	-13.7	--	14.4	-197.28	Weight of Concrete Slab Beneath Ogee
W.slab2	-8.4	--	-10	0*	Weight of Concrete Slab DS of Ogee
W.soil	-1.1	--	27.5	-30.25	Buoyant Weight of Upstream Soil
W.hwNP	-5.8	--	27.5	-159.5	Weight of Water Upstream
W.tw1_NP	-1.3	--	5.2	-6.76	Weight of tailwater above Slab1
W.tw2_NP	-2.6	--	-10.3	0*	Weight of tailwater above Slab 2
W.Slab3	-10.7	--	-28	0*	Slab 3 Buoyant Weight (122.6'*1'*87.6pcf)
S.us	--	0.82	1.8	1.48	Horiz. Soil Load
H1	--	15.13	6.94	105.00	Horiz. Hydrostatic Force
U1	19.44	--	16	311.04	Vertical Hydrostatic force
U2	8.05	--	-9.44	0*	Vertical Hydrostatic force

Note: *Force omitted from moment computations, but included in H and V calculations.

Summary Table Totals			**Comments**
Vert. Forces in Moment (kip):	-28.26	(V.am, N)	= (W.ogee+W.slab1+W.soil+W.hwNP+W.tw1_NP+U1)
Other Vert. Forces (kip):	-13.65	(V.a)	= (W.slab2+W.tw2_NP+W.Slab3+U2)
Total Vertical Force (kip):	-41.91	(V, N.sff)	= (W.ogee+W.slab1+W.soil+W.hwNP+W.tw1_NP+U1+W.slab2+W.tw2_NP+W.Slab3+U2)
Horiz. Forces in Moment (kip):	16.0	(H, T)	= (S.us+H1)
Vert. Force Moment (kip*ft):	-593.6	(M.V)	= (W.ogee+W.slab1+W.soil+W.hwNP+W.tw1_NP+U1)
Horiz. Force Moment (kip*ft):	106.5	(M.H)	= (S.us+H1)
Weights (kip):	-69.4	(W.sum)	= (W.ogee+W.slab1+W.slab2+W.soil+W.hwNP+W.tw1_NP+W.tw2_NP+W.Slab3)

Eccentricity, Base Pressures, and Factor of Safety

Input Constants:			**Comments:**
Horiz. Base Length (ft):	30	(B)	(In Compression)
Section Length (ft):	1	(L.D)	(Into page)
Base Area (sf):	50.4	(A)	
Base angle (deg):	0	(a)	
Rotation Elevation (ft):	855.9	(R.el)	

Resultant Location:

Resultant Dist. to toe (ft):	17.24	(R.dist)	= (M.V+M.H)/N = (-593.6kip*ft+106.5kip*ft)/-28.26kip
Eccentricity, from Neutral Axis (ft):	-2.24	(e.e)	= B/2 - R.dist = 30ft /2 - 17.24 ft
D/S Kern Limit, from Neutral Axis (ft):	5		= B/6 = 30ft / 6
Base Press. U/S (ksf):	1.36		= (|N|/(B*L.D)*(1-6*e.e/B) = (|-28.26|/(30'*1'))*(1-6*-2.24ft/30ft)
Base Press. D/S (ksf):	0.52		= (|N|/(B*L.D')*(1+6*e.e/B) = (|-28.26|/(30'*1'))*(1+6*-2.24ft/30ft)
% Base in Compression:	100%		= Resultant in Kern, Entire Base in Compression
SFF:			= (C*A+ |N.sff|*tan(phi))/ T
SFF (phi: 37°, c: 0psi):	**1.97**		= (0 + |-41.91k| * tan(37 deg)) / 16 kip

GEI Consultants	Client	City of Manchester			Page	
	Project	Ford Manchester Dam			Pg. Rev.	0
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Project No.	2204052	Document No.	N/A			
Subject	Overflow Spillway Stability Analyses					

CASE I - SUPPORTING COMPUTATIONS

LOAD CASE: Overflow Spillway - Case I: Normal Operation

Headwater Elevation (ft): 877.52

Tailwater Elevation (ft): 861.0

Calculated Weighted Creep Pressure Head for Defined Points along Base

(Refer to Results Summary Figure for point locations)

Point	Elev. (ft)	Distance from Toe (ft)	Perm. Ratio K.v/K.h	Horz. Creep Length To Point (ft)	Vert. Creep Length to Point (ft)	Weighted Creep Length to Point (ft)	Seepage Potential at Point (ft)	Position Potential at Point (ft)	Pressure Head (ft)	Total Head (ft)	Pressure (ksf)
	(El.)	(H)	(K.v/K.h)	(H.wc)	(V.wc)	(L.wc)	(SP)	(PP)	(SP + PP)		(P)
B1	862.0	30.1	1/3	0.0	0.0	0.0	16.5	-1.0	15.5	877.52	0.968
B3ssp	848.0	30.1	1/3	0.0	14.0	14.0	10.7	13.0	23.7	871.68	1.478
B2ds	855.9	30.0	1/3	0.0	21.9	21.9	7.4	5.1	12.5	868.37	0.778
B4	855.9	0.0	1/3	10.0	21.9	31.9	3.2	5.1	8.3	864.2	0.518
B5	856.3	0.0	1/3	10.0	22.3	32.3	3.0	4.7	7.7	864.04	0.483
B6	856.3	-20.4	1/3	16.8	22.3	39.1	0.2	4.7	4.9	861.2	0.306
B7	856.3	-21.8	1/3	17.3	22.3	39.6	0.0	4.7	4.7	861	0.293
					Total Weighted Creep Distance (ft):	39.61	(L.tot)				

Sample Calculations for Point B2ds:

H.wc[B2ds] = H.wc[B3ssp] + (K.v/K.h) * |H[B3ssp] - H[B2ds]| = 0ft + (0.33)*|30.1ft -30ft| = 0.03 ft

V.wc[B2ds] = V.wc[B3ssp] + |El.[B3ssp] - El.[B2ds]| = 14ft + |848ft -855.9ft| = 21.9 ft

L.wc[B2ds] = H.wc[B2ds] + V.wc[B2ds] = 0.03 ft + 21.9 ft = 21.93 ft

SP = (HW-TW) * ((L.tot - L.wc) / L.tot) = (877.52 ft - 861 ft) * ((39.61 ft - 21.93 ft)/ 39.61 ft) = 7.37 ft

PP = TW - El. = 861 ft - 855.9 ft = 5.1 ft

Calculated Pressure Head for Points along the U/S and D/S Structure Face

(Refer to Results Summary Figure for point locations)

Label	Elevation (ft) (El)	Horiz. Dist To Toe (ft) (X)	Total Head (ft) (h)	Pressure Head (ft) (P)	Pressure (ksf)
	(EL)	(X)	(th)	(P.h)	(P)
HW	877.5	30.1	877.5	0.0	0
SILL	877.5	30.1	877.5	0.0	0
B2us	855.5	30.1	877.5	22.0	1.374
B8	859.0	-21.8	861.0	2.0	0.125
TW	861.0	-21.8	861.0	0.0	0

Sample Calculation for Point HW:

P.h = TH - EL = 877.52' - 877.52' = 0'

*P = 0.0624kcf * P.h = 0.0624 kcf * 0' = 0 ksf*

CASE I - SUPPORTING COMPUTATIONS

LOAD CASE: Overflow Spillway - Case I: Normal Operation
Headwater Elevation (ft): 877.52
Tailwater Elevation (ft): 861.0

Horizontal Hydrostatic Forces

(Refer to calculated pressure head table for point locations) Rotation Elevation, EL.rotate (ft): 855.9

Force Label	U/S Point	D/S Point	U/S Pressure Head (ft)	D/S Pressure Head (ft)	U/S Elevation (ft)	D/S Elevation (ft)	Applied Length (ft)	Load Factor	Horiz. Hydro-static Force (kip)	Moment Arm (ft - from toe)	Moment (kip*ft)
			(P.us)	(P.ds)	(EL.us)	(EL.ds)	(L)	(LF)	(F.h)	(MA)	(M=F.h*MA)
H1	Sill	B2us	0.0	22.02	877.52	855.5	1.0	1	15.13	6.94	105.0

Sample Calculation:

*F.h = [(P.us + P.ds) / 2] * (EL.us - EL.ds) * L * LF * 0.0624 kcf*

F.h [H1] = [(0ft + 22.02ft) / 2] x (877.52 ft - 855.5 ft) x 1ft x 1 x 0.0624 kcf = 15.13 kips

MA [H1] = EL.ds + ((EL.us -EL.ds)/3(2* P.us + P.ds)/(P.us + P.ds)) - EL.rotate*

MA [H1] = 855.5' +((877.52' - 855.5')/3(2* 0' + 22.02')/(0' + 22.02'))-855.9' = 6.94'*

Vertical Hydrostatic (Uplift) Forces

(Refer to calculated pressure head table for point locations) (a): 0 Failure Plane Incline Above Horizontal (deg)

Force Label	U/S Point	D/S Point	U/S Pressure Head (ft)	D/S Pressure Head (ft)	U/S Distance From Toe (ft)	D/S Distance From Toe (ft)	Applied Length (ft)	Base Area (sf)	Load Factor	Uplift Force (kip)	Moment Arm (ft - from toe)	Moment (kip*ft)
			(P.us)	(P.ds)	(X.us)	(X.ds)	(L)	(A)	(LF)	(F.up)	(MA)	(M)
U1	B2ds	B4	12.47	8.3	30.0	0.0	1.0	30.0	1	19.44	16.0	311.0
U2	B5	B6	7.74	4.9	0.0	-20.4	1.0	20.4	1	8.05	-9.44	0*

Sample Calculation: Total Area, A (sf): 50.4

A [U1] = (X.us - X.ds) / cos(a) L = (30' - 0') / cos(0deg)* 1' = 30 sf*

*F.up [U1] = [(P.us + P.ds)/2] * (X.us - X.ds) * L * LF * 0.0624kcf*

*F.up [U1]= [(12.47ft + 8.3ft) / 2] * (30 ft - 0 ft) * 1 ft * 1 * 0.0624 kcf = 19.44 kips*

MA [U1] = X.us -(X.us -X.ds)/3(2* P.ds + P.us)/(P.ds + P.us)*

MA [U1] = 30' -(30' - 0')/3(2* 8.3' + 12.47')/(8.3' + 12.47') = 16'*

Horizontal Soil Loads

Label	γ.s (pcf)	Unit Wt. Type	Φ deg	Earth Pressure Coeff.	Load Factor	Surcharge Stress (psf)	Upper Elevation (ft)	Lower Elevation (ft)	Length (ft)	Moment arm (ft)	Soil Load (kip)	Dir. (U/S, D/S)
-	(g.s)	-	(phi)	(K)	(LF)	(q)	(El.u)	(EL.d)	(L)	(MA)	(F.s)	-
S.us	77.6	Buoy.	30	K.0: 0.5	1	0	862	855.5	1.0	1.8	0.820	D/S

Sample Calculation:

K.0 = 1 - sin Φ

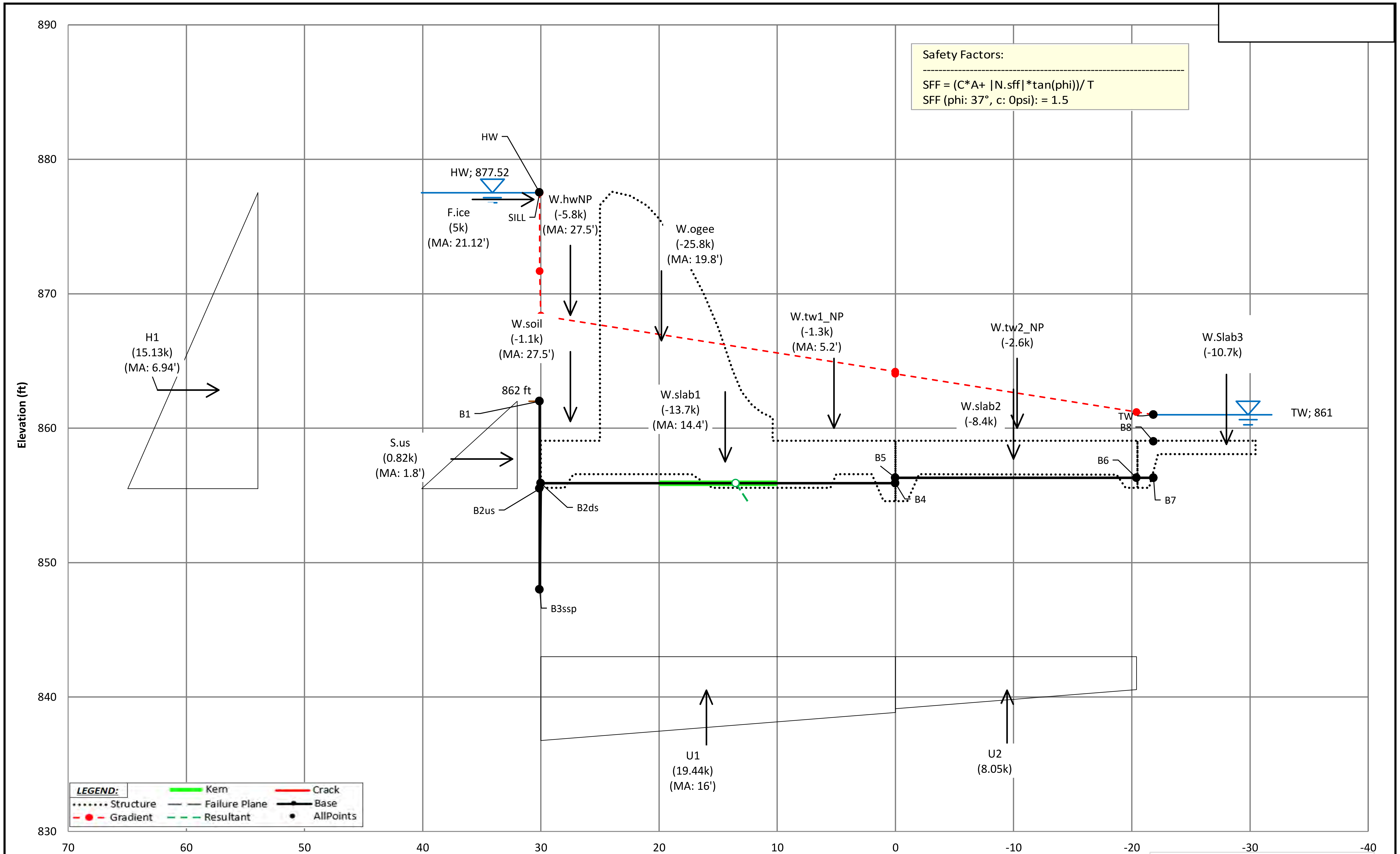
*F.s [S.us] = 0.5 * [2*Surcharge + g.s * (El.upper - El.lower)] * (El.upper - El.lower) * K * (LF) * L*

*F.s [S.us] = 0.5 * [2 * 0psf + 77.6pcf * (862ft - 855.5ft)] * (862ft - 855.5ft) * 0.5 * (1) * 1ft = 0.82 kip*

*MA [S.us] = (EL.u - EL.rotate) + (EL.u - EL.d) * [q + 1/3*(EL.u - EL.d) * g.s] / [2 * q + (EL.u - EL.d) * g.s]*

*MA [S.us] = (855.5' - 855.9')+ (862' - 855.5') * [0psf + 1/3*(862' - 855.5')*77.6pcf] / [2 * 0psf + (862' - 855.5')*77.6pcf] = 1.8'*

STABILITY - NORMAL + ICE POOL CONDITION



Point of Rotation: (0, 855.9)

Notes:

BY: P. Grodecki 11/27/2023
CHK: E. Baffoe 11/30/2023
APP: M. Guirguis 12/01/2023

Force & Moment Orientation	OVERFLOW SPILLWAY SPILLWAY STABILITY	GEI Consultants	**CASE IIA NORMAL + ICE STABILITY SUMMARY DIAGRAM**
	City of Manchester	Project: 2204052	December 2023

GEI Consultants	Client	City of Manchester			Page	
	Project	Ford Manchester Dam			Pg. Rev.	0
	By	P. Grodecki	Chk.	E. Baffoe	App.	M. Guirguis
	Date	11/27/2023	Date	11/30/2023	Date	12/01/2023

Project No.	2204052	Document No.	N/A
Subject	Overflow Spillway Stability Analyses		

CASE IIA - RESULTS SUMMARY

LOAD CASE: Overflow Spillway - Case IIA: Normal + Ice

Headwater Elevation (ft): 877.52
Tailwater Elevation (ft): 861.0

Force and Moment Calculation Summary Table

Force Label	Vertical Force (kip)	Horiz. Force (kip)	Moment Arm (ft)	Moment (kip*ft)	Comments
W.ogee	-25.8	--	19.8	-510.84	Weight of Concrete Ogee
W.slab1	-13.7	--	14.4	-197.28	Weight of Concrete Slab Beneath Ogee
W.slab2	-8.4	--	-10	0*	Weight of Concrete Slab DS of Ogee
W.soil	-1.1	--	27.5	-30.25	Buoyant Weight of Upstream Soil
W.hwNP	-5.8	--	27.5	-159.5	Weight of Water Upstream
W.tw1_NP	-1.3	--	5.2	-6.76	Weight of tailwater above Slab1
W.tw2_NP	-2.6	--	-10.3	0*	Weight of tailwater above Slab 2
W.Slab3	-10.7	--	-28	0*	Slab 3 Buoyant Weight (122.6'*1'*87.6pcf)
F.ice	--	5	21.12	105.60	Ice Force: 5 ksf, 1' thick, 1' into page
S.us	--	0.82	1.8	1.48	Horiz. Soil Load
H1	--	15.13	6.94	105.00	Horiz. Hydrostatic Force
U1	19.44	--	16	311.04	Vertical Hydrostatic force
U2	8.05	--	-9.44	0*	Vertical Hydrostatic force

Note: *Force omitted from moment computations, but included in H and V calculations.

Summary Table Totals			**Comments**
Vert. Forces in Moment (kip):	-28.26	(V.am, N)	= (W.ogee+W.slab1+W.soil+W.hwNP+W.tw1_NP+U1)
Other Vert. Forces (kip):	-13.65	(V.a)	= (W.slab2+W.tw2_NP+W.Slab3+U2)
Total Vertical Force (kip):	-41.91	(V, N.sff)	= (W.ogee+W.slab1+W.soil+W.hwNP+W.tw1_NP+U1+W.slab2+W.tw2_NP+W.Slab3+U2)
Horiz. Forces in Moment (kip):	21.0	(H, T)	= (F.ice+S.us+H1)
Vert. Force Moment (kip*ft):	-593.6	(M.V)	= (W.ogee+W.slab1+W.soil+W.hwNP+W.tw1_NP+U1)
Horiz. Force Moment (kip*ft):	212.1	(M.H)	= (F.ice+S.us+H1)
Weights (kip):	-69.4	(W.sum)	= (W.ogee+W.slab1+W.slab2+W.soil+W.hwNP+W.tw1_NP+W.tw2_NP+W.Slab3)

Eccentricity, Base Pressures, and Factor of Safety

Input Constants:			**Comments:**
Horiz. Base Length (ft):	30	(B)	(In Compression)
Section Length (ft):	1	(L.D)	(Into page)
Base Area (sf):	50.4	(A)	
Base angle (deg):	0	(a)	
Rotation Elevation (ft):	855.9	(R.el)	

Resultant Location:

Resultant Dist. to toe (ft):	13.5	(R.dist)	= (M.V+M.H)/N = (-593.6kip*ft+212.1kip*ft)/-28.26kip
Eccentricity, from Neutral Axis (ft):	1.5	(e.e)	= B/2 - R.dist = 30ft /2 - 13.5 ft
D/S Kern Limit, from Neutral Axis (ft):	5		= B/6 = 30ft / 6
Base Press. U/S (ksf):	0.66		= (|N|/(B*L.D)*(1-6*e.e/B) = (|-28.26|/(30'*1'))*(1-6*1.5ft/30ft)
Base Press. D/S (ksf):	1.22		= (|N|/(B*L.D')*(1+6*e.e/B) = (|-28.26|/(30'*1'))*(1+6*1.5ft/30ft)
% Base in Compression:	100%		= Resultant in Kern, Entire Base in Compression

SFF: = (C*A+ |N.sff|*tan(phi))/ T

SFF (phi: 37°, c: 0psi): **1.5** $= \frac{0 + |-41.91k| * \tan(37\ deg)}{21\ kip}$

GEI Consultants	Client	City of Manchester			Page	
	Project	Ford Manchester Dam			Pg. Rev.	0
	By	P. Grodecki	Chk.	E. Baffoe	App.	M. Guirguis
	Date	11/27/2023	Date	11/30/2023	Date	12/01/2023
Project No.	2204052	Document No.	N/A			
Subject	Overflow Spillway Stability Analyses					

CASE IIA - SUPPORTING COMPUTATIONS

LOAD CASE: Overflow Spillway - Case IIA: Normal + Ice
Headwater Elevation (ft): 877.52
Tailwater Elevation (ft): 861.0

Calculated Weighted Creep Pressure Head for Defined Points along Base
(Refer to Results Summary Figure for point locations)

Point	Elev. (ft)	Distance from Toe (ft)	Perm. Ratio K.v/K.h	Horz. Creep Length To Point (ft)	Vert. Creep Length to Point (ft)	Weighted Creep Length to Point (ft)	Seepage Potential at Point (ft)	Position Potential at Point (ft)	Pressure Head (ft)	Total Head (ft)	Pressure (ksf)
	(El.)	(H)	(K.v/K.h)	(H.wc)	(V.wc)	(L.wc)	(SP)	(PP)	(SP + PP)		(P)
B1	862.0	30.1	1/3	0.0	0.0	0.0	16.5	-1.0	15.5	877.52	0.968
B3ssp	848.0	30.1	1/3	0.0	14.0	14.0	10.7	13.0	23.7	871.68	1.478
B2ds	855.9	30.0	1/3	0.0	21.9	21.9	7.4	5.1	12.5	868.37	0.778
B4	855.9	0.0	1/3	10.0	21.9	31.9	3.2	5.1	8.3	864.2	0.518
B5	856.3	0.0	1/3	10.0	22.3	32.3	3.0	4.7	7.7	864.04	0.483
B6	856.3	-20.4	1/3	16.8	22.3	39.1	0.2	4.7	4.9	861.2	0.306
B7	856.3	-21.8	1/3	17.3	22.3	39.6	0.0	4.7	4.7	861	0.293

Total Weighted Creep Distance (ft): 39.61 (L.tot)

Sample Calculations for Point B2ds:

H.wc[B2ds] = H.wc[B3ssp] + (K.v/K.h) * |H[B3ssp] - H[B2ds]| = 0ft + (0.33)*|30.1ft -30ft| = 0.03 ft
V.wc[B2ds] = V.wc[B3ssp] + |El.[B3ssp] - El.[B2ds]| = 14ft + |848ft -855.9ft| = 21.9 ft
L.wc[B2ds] = H.wc[B2ds] + V.wc[B2ds] = 0.03 ft + 21.9 ft = 21.93 ft
SP = (HW-TW) * ((L.tot - L.wc) / L.tot) = (877.52 ft - 861 ft) * ((39.61 ft - 21.93 ft)/ 39.61 ft) = 7.37 ft
PP = TW - El. = 861 ft - 855.9 ft = 5.1 ft

Calculated Pressure Head for Points along the U/S and D/S Structure Face
(Refer to Results Summary Figure for point locations)

Label	Elevation (ft) (El)	Horiz. Dist To Toe (ft) (X)	Total Head (ft) (h)	Pressure Head (ft) (P)	Pressure (ksf)
	(EL)	(X)	(th)	(P.h)	(P)
HW	877.5	30.1	877.5	0.0	0
SILL	877.5	30.1	877.5	0.0	0
B2us	855.5	30.1	877.5	22.0	1.374
B8	859.0	-21.8	861.0	2.0	0.125
TW	861.0	-21.8	861.0	0.0	0

Sample Calculation for Point HW:
P.h = TH - EL = 877.52' - 877.52' = 0'
*P = 0.0624kcf * P.h = 0.0624 kcf * 0' = 0 ksf*

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CASE IIA - SUPPORTING COMPUTATIONS

LOAD CASE: Overflow Spillway - Case IIA: Normal + Ice
Headwater Elevation (ft): 877.52
Tailwater Elevation (ft): 861.0

Horizontal Hydrostatic Forces

(Refer to calculated pressure head table for point locations) Rotation Elevation, EL.rotate (ft): 855.9

Force Label	U/S Point	D/S Point	U/S Pressure Head (ft)	D/S Pressure Head (ft)	U/S Elevation (ft)	D/S Elevation (ft)	Applied Length (ft)	Load Factor	Horiz. Hydro-static Force (kip)	Moment Arm (ft - from toe)	Moment (kip*ft)
			(P.us)	(P.ds)	(EL.us)	(EL.ds)	(L)	(LF)	(F.h)	(MA)	(M=F.h*MA)
H1	Sill	B2us	0.0	22.02	877.52	855.5	1.0	1	15.13	6.94	105.0

Sample Calculation:

*F.h = [(P.us + P.ds) / 2] * (EL.us - EL.ds) * L * LF * 0.0624 kcf*

F.h [H1] = [(0ft + 22.02ft) / 2] x (877.52 ft - 855.5 ft) x 1ft x 1 x 0.0624 kcf = 15.13 kips

MA [H1] = EL.ds + ((EL.us -EL.ds)/3(2* P.us + P.ds)/(P.us + P.ds)) - EL.rotate*

MA [H1] = 855.5' +((877.52' - 855.5')/3(2* 0' + 22.02')/(0' + 22.02'))-855.9' = 6.94'*

Vertical Hydrostatic (Uplift) Forces

(Refer to calculated pressure head table for point locations) (a): 0 Failure Plane Incline Above Horizontal (deg)

Force Label	U/S Point	D/S Point	U/S Pressure Head (ft)	D/S Pressure Head (ft)	U/S Distance From Toe (ft)	D/S Distance From Toe (ft)	Applied Length (ft)	Base Area (sf)	Load Factor	Uplift Force (kip)	Moment Arm (ft - from toe)	Moment (kip*ft)
			(P.us)	(P.ds)	(X.us)	(X.ds)	(L)	(A)	(LF)	(F.up)	(MA)	(M)
U1	B2ds	B4	12.47	8.3	30.0	0.0	1.0	30.0	1	19.44	16.0	311.0
U2	B5	B6	7.74	4.9	0.0	-20.4	1.0	20.4	1	8.05	-9.44	0*

Sample Calculation: Total Area, A (sf): 50.4

A [U1] = (X.us - X.ds) / cos(a) L = (30' - 0') / cos(0deg)* 1' = 30 sf*

*F.up [U1] = [(P.us + P.ds)/2] * (X.us - X.ds) * L * LF * 0.0624kcf*

*F.up [U1]= [(12.47ft + 8.3ft) / 2] * (30 ft - 0 ft) * 1 ft * 1 * 0.0624 kcf = 19.44 kips*

MA [U1] = X.us -(X.us -X.ds)/3(2* P.ds + P.us)/(P.ds + P.us)*

MA [U1] = 30' -(30' - 0')/3(2* 8.3' + 12.47')/(8.3' + 12.47') = 16'*

Horizontal Soil Loads

Label	γ.s (pcf)	Unit Wt. Type	Φ deg	Earth Pressure Coeff.	Load Factor	Surcharge Stress (psf)	Upper Elevation (ft)	Lower Elevation (ft)	Length (ft)	Moment arm (ft)	Soil Load (kip)	Dir. (U/S, D/S)
-	(g.s)	-	(phi)	(K)	(LF)	(q)	(El.u)	(EL.d)	(L)	(MA)	(F.s)	-
S.us	77.6	Buoy.	30	K.0: 0.5	1	0	862	855.5	1.0	1.8	0.820	D/S

Sample Calculation:

K.0 = 1 - sin Φ

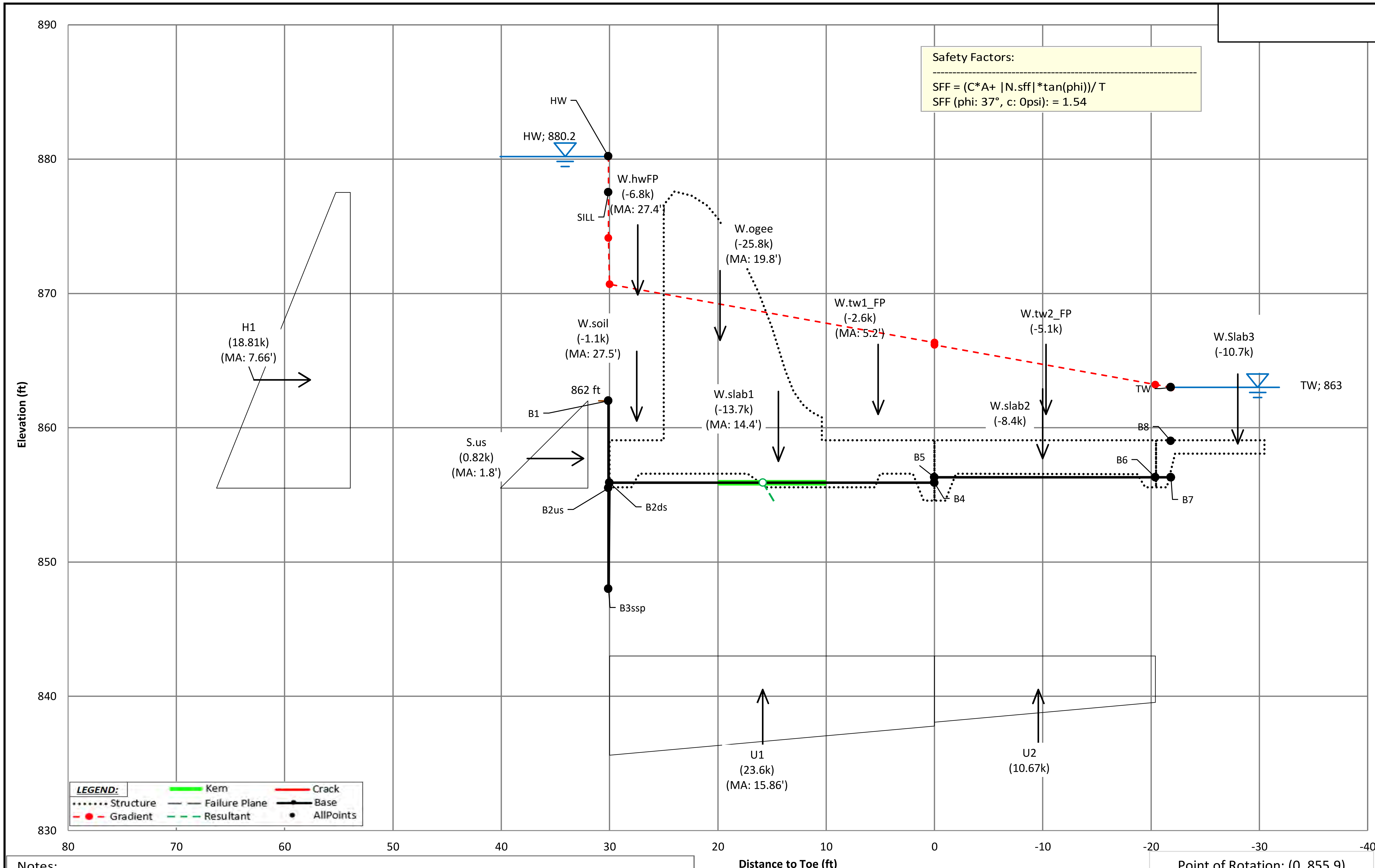
*F.s [S.us] = 0.5 * [2*Surcharge + g.s * (El.upper - El.lower)] * (El.upper - El.lower) * K * (LF) * L*

*F.s [S.us] = 0.5 * [2 * 0psf + 77.6pcf * (862ft - 855.5ft)] * (862ft - 855.5ft) * 0.5 * (1) * 1ft = 0.82 kip*

*MA [S.us] = (EL.u - EL.rotate) + (EL.u - EL.d) * [q + 1/3*(EL.u - EL.d) * g.s] / [2 * q + (EL.u - EL.d) * g.s]*

*MA [S.us] = (855.5' - 855.9')+ (862' - 855.5') * [0psf + 1/3*(862' - 855.5')*77.6pcf] / [2 * 0psf + (862' - 855.5')*77.6pcf] = 1.8'*

STABILITY - FLOOD POOL CONDITIONS



Notes:

BY: P. Grodecki 11/27/2023
CHK: E. Baffoe 11/30/2023
APP: M. Guirguis 12/01/2023

Force & Moment Orientation

OVERFLOW SPILLWAY
SPILLWAY STABILITY

City of Manchester

GEI Consultants

Project: 2204052

CASE II
FLOOD CONDITIONS
STABILITY SUMMARY
DIAGRAM

December 2023

CASE II - RESULTS SUMMARY

LOAD CASE: Overflow Spillway - Case II: Flood Conditions

Headwater Elevation (ft): 880.2
Tailwater Elevation (ft): 863.0

Force and Moment Calculation Summary Table

Force Label	Vertical Force (kip)	Horiz. Force (kip)	Moment Arm (ft)	Moment (kip*ft)	Comments
W.ogee	-25.8	--	19.8	-510.84	Weight of Concrete Ogee
W.slab1	-13.7	--	14.4	-197.28	Weight of Concrete Slab Beneath Ogee
W.slab2	-8.4	--	-10	0*	Weight of Concrete Slab DS of Ogee
W.soil	-1.1	--	27.5	-30.25	Buoyant Weight of Upstream Soil
W.hwFP	-6.8	--	27.4	-186.32	Weight of Water Upstream
W.tw1_FP	-2.6	--	5.2	-13.52	Weight of tailwater above Slab1
W.tw2_FP	-5.1	--	-10.3	0*	Weight of tailwater above Slab 2
W.Slab3	-10.7	--	-28	0*	Slab 3 Buoyant Weight (122.6'*1'*87.6pcf)
S.us	--	0.82	1.8	1.48	Horiz. Soil Load
H1	--	18.81	7.66	144.08	Horiz. Hydrostatic Force
U1	23.6	--	15.86	374.30	Vertical Hydrostatic force
U2	10.67	--	-9.6	0*	Vertical Hydrostatic force

Note: *Force omitted from moment computations, but included in H and V calculations.

Summary Table Totals			**Comments**
Vert. Forces in Moment (kip):	-26.4	(V.am, N)	= (W.ogee+W.slab1+W.soil+W.hwFP+W.tw1_FP+U1)
Other Vert. Forces (kip):	-13.53	(V.a)	= (W.slab2+W.tw2_FP+W.Slab3+U2)
Total Vertical Force (kip):	-39.93	(V, N.sff)	= (W.ogee+W.slab1+W.soil+W.hwFP+W.tw1_FP+U1+W.slab2+W.tw2_FP+W.Slab3+U2)
Horiz. Forces in Moment (kip):	19.6	(H, T)	= (S.us+H1)
Vert. Force Moment (kip*ft):	-563.9	(M.V)	= (W.ogee+W.slab1+W.soil+W.hwFP+W.tw1_FP+U1)
Horiz. Force Moment (kip*ft):	145.6	(M.H)	= (S.us+H1)
Weights (kip):	-74.2	(W.sum)	= (W.ogee+W.slab1+W.slab2+W.soil+W.hwFP+W.tw1_FP+W.tw2_FP+W.Slab3)

Eccentricity, Base Pressures, and Factor of Safety

Input Constants:			**Comments:**
Horiz. Base Length (ft):	30	(B)	(In Compression)
Section Length (ft):	1	(L.D)	(Into page)
Base Area (sf):	50.4	(A)	
Base angle (deg):	0	(a)	
Rotation Elevation (ft):	855.9	(R.el)	

Resultant Location:

Resultant Dist. to toe (ft):	15.84	(R.dist)	= (M.V+M.H)/N = (-563.9kip*ft+145.6kip*ft)/-26.4kip
Eccentricity, from Neutral Axis (ft):	-0.84	(e.e)	= B/2 - R.dist = 30ft /2 - 15.84 ft
D/S Kern Limit, from Neutral Axis (ft):	5		= B/6 = 30ft / 6
Base Press. U/S (ksf):	1.03		= (|N|/(B*L.D)*(1-6*e.e/B) = (|-26.4|/(30'*1'))*(1-6*-0.84ft/30ft)
Base Press. D/S (ksf):	0.73		= (|N|/(B*L.D')*(1+6*e.e/B) = (|-26.4|/(30'*1'))*(1+6*-0.84ft/30ft)
% Base in Compression:	100%		= Resultant in Kern, Entire Base in Compression

SFF: = (C*A+ |N.sff|*tan(phi))/ T

SFF (phi: 37°, c: 0psi): **1.54** $= \dfrac{0 + |-39.93k| * \tan(37\ deg)}{19.6\ kip}$

CASE II - SUPPORTING COMPUTATIONS

LOAD CASE: Overflow Spillway - Case II: Flood Conditions
Headwater Elevation (ft): 880.2
Tailwater Elevation (ft): 863.0

Calculated Weighted Creep Pressure Head for Defined Points along Base
(Refer to Results Summary Figure for point locations)

Point	Elev. (ft)	Distance from Toe (ft)	Perm. Ratio K.v/K.h	Horz. Creep Length To Point (ft)	Vert. Creep Length to Point (ft)	Weighted Creep Length to Point (ft)	Seepage Potential at Point (ft)	Position Potential at Point (ft)	Pressure Head (ft)	Total Head (ft)	Pressure (ksf)
	(El.)	(H)	(K.v/K.h)	(H.wc)	(V.wc)	(L.wc)	(SP)	(PP)	(SP + PP)		(P)
B1	862.0	30.1	1/3	0.0	0.0	0.0	17.2	1.0	18.2	880.2	1.136
B3ssp	848.0	30.1	1/3	0.0	14.0	14.0	11.1	15.0	26.1	874.12	1.63
B2ds	855.9	30.0	1/3	0.0	21.9	21.9	7.7	7.1	14.8	870.68	0.922
B4	855.9	0.0	1/3	10.0	21.9	31.9	3.3	7.1	10.4	866.33	0.651
B5	856.3	0.0	1/3	10.0	22.3	32.3	3.2	6.7	9.9	866.16	0.615
B6	856.3	-20.4	1/3	16.8	22.3	39.1	0.2	6.7	6.9	863.21	0.431
B7	856.3	-21.8	1/3	17.3	22.3	39.6	0.0	6.7	6.7	863	0.418

Total Weighted Creep Distance (ft): 39.61 (L.tot)

Sample Calculations for Point B2ds:

H.wc[B2ds] = H.wc[B3ssp] + (K.v/K.h) * |H[B3ssp] - H[B2ds]| = 0ft + (0.33)*|30.1ft -30ft| = 0.03 ft
V.wc[B2ds] = V.wc[B3ssp] + |El.[B3ssp] - El.[B2ds]| = 14ft + |848ft -855.9ft| = 21.9 ft
L.wc[B2ds] = H.wc[B2ds] + V.wc[B2ds] = 0.03 ft + 21.9 ft = 21.93 ft
SP = (HW-TW) * ((L.tot - L.wc) / L.tot) = (880.2 ft - 863 ft) * ((39.61 ft - 21.93 ft)/ 39.61 ft) = 7.68 ft
PP = TW - El. = 863 ft - 855.9 ft = 7.1 ft

Calculated Pressure Head for Points along the U/S and D/S Structure Face
(Refer to Results Summary Figure for point locations)

Label	Elevation (ft) (El)	Horiz. Dist To Toe (ft) (X)	Total Head (ft) (h)	Pressure Head (ft) (P)	Pressure (ksf)
	(EL)	(X)	(th)	(P.h)	(P)
HW	880.2	30.1	880.2	0.0	0
SILL	877.5	30.1	880.2	2.7	0.167
B2us	855.5	30.1	880.2	24.7	1.541
B8	859.0	-21.8	863.0	4.0	0.25
TW	863.0	-21.8	863.0	0.0	0

Sample Calculation for Point HW:
P.h = TH - EL = 880.2' - 880.2' = 0'
*P = 0.0624kcf * P.h = 0.0624 kcf * 0' = 0 ksf*

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CASE II - SUPPORTING COMPUTATIONS

LOAD CASE: Overflow Spillway - Case II: Flood Conditions
Headwater Elevation (ft): 880.2
Tailwater Elevation (ft): 863.0

Horizontal Hydrostatic Forces

(Refer to calculated pressure head table for point locations) Rotation Elevation, EL.rotate (ft): 855.9

Force Label	U/S Point	D/S Point	U/S Pressure Head (ft)	D/S Pressure Head (ft)	U/S Elevation (ft)	D/S Elevation (ft)	Applied Length (ft)	Load Factor	Horiz. Hydro-static Force (kip)	Moment Arm (ft - from toe)	Moment (kip*ft)
			(P.us)	(P.ds)	(EL.us)	(EL.ds)	(L)	(LF)	(F.h)	(MA)	(M=F.h*MA)
H1	Sill	B2us	2.68	24.7	877.52	855.5	1.0	1	18.81	7.66	144.1

Sample Calculation:

*F.h = [(P.us + P.ds) / 2] * (EL.us - EL.ds) * L * LF * 0.0624 kcf*

F.h [H1] = [(2.68ft + 24.7ft) / 2] x (877.52 ft - 855.5 ft) x 1ft x 1 x 0.0624 kcf = 18.81 kips

MA [H1] = EL.ds + ((EL.us -EL.ds)/3(2* P.us + P.ds)/(P.us + P.ds)) - EL.rotate*

MA [H1] = 855.5' +((877.52' - 855.5')/3(2* 2.68' + 24.7')/(2.68' + 24.7'))-855.9' = 7.66'*

Vertical Hydrostatic (Uplift) Forces

(Refer to calculated pressure head table for point locations) (a): 0 Failure Plane Incline Above Horizontal (deg)

Force Label	U/S Point	D/S Point	U/S Pressure Head (ft)	D/S Pressure Head (ft)	U/S Distance From Toe (ft)	D/S Distance From Toe (ft)	Applied Length (ft)	Base Area (sf)	Load Factor	Uplift Force (kip)	Moment Arm (ft - from toe)	Moment (kip*ft)
			(P.us)	(P.ds)	(X.us)	(X.ds)	(L)	(A)	(LF)	(F.up)	(MA)	(M)
U1	B2ds	B4	14.78	10.43	30.0	0.0	1.0	30.0	1	23.6	15.86	374.3
U2	B5	B6	9.86	6.91	0.0	-20.4	1.0	20.4	1	10.67	-9.6	0*

Sample Calculation: Total Area, A (sf): 50.4

A [U1] = (X.us - X.ds) / cos(a) L = (30' - 0') / cos(0deg)* 1' = 30 sf*

*F.up [U1] = [(P.us + P.ds)/2] * (X.us - X.ds) * L * LF * 0.0624kcf*

*F.up [U1]= [(14.78ft + 10.43ft) / 2] * (30 ft - 0 ft) * 1 ft * 1 * 0.0624 kcf = 23.6 kips*

MA [U1] = X.us -(X.us -X.ds)/3(2* P.ds + P.us)/(P.ds + P.us)*

MA [U1] = 30' -(30' - 0')/3(2* 10.43' + 14.78')/(10.43' + 14.78') = 15.9'*

Horizontal Soil Loads

Label	γ.s (pcf)	Unit Wt. Type	Φ deg	Earth Pressure Coeff.	Load Factor	Surcharge Stress (psf)	Upper Elevation (ft)	Lower Elevation (ft)	Length (ft)	Moment arm (ft)	Soil Load (kip)	Dir. (U/S, D/S)
-	(g.s)	-	(phi)	(K)	(LF)	(q)	(El.u)	(EL.d)	(L)	(MA)	(F.s)	-
S.us	77.6	Buoy.	30	K.0: 0.5	1	0	862	855.5	1.0	1.8	0.820	D/S

Sample Calculation:

K.0 = 1 - sin Φ

*F.s [S.us] = 0.5 * [2*Surcharge + g.s * (El.upper - El.lower)] * (El.upper - El.lower) * K * (LF) * L*

*F.s [S.us] = 0.5 * [2 * 0psf + 77.6pcf * (862ft - 855.5ft)] * (862ft - 855.5ft) * 0.5 * (1) * 1ft = 0.82 kip*

*MA [S.us] = (EL.u - EL.rotate) + (EL.u - EL.d) * [q + 1/3*(EL.u - EL.d) * g.s] / [2 * q + (EL.u - EL.d) * g.s]*

*MA [S.us] = (855.5' - 855.9')+ (862' - 855.5') * [0psf + 1/3*(862' - 855.5')*77.6pcf] / [2 * 0psf + (862' - 855.5')*77.6pcf] = 1.8'*

CENTROID

CENTROID - OVERFLOW SPILLWAY STRUCTURE

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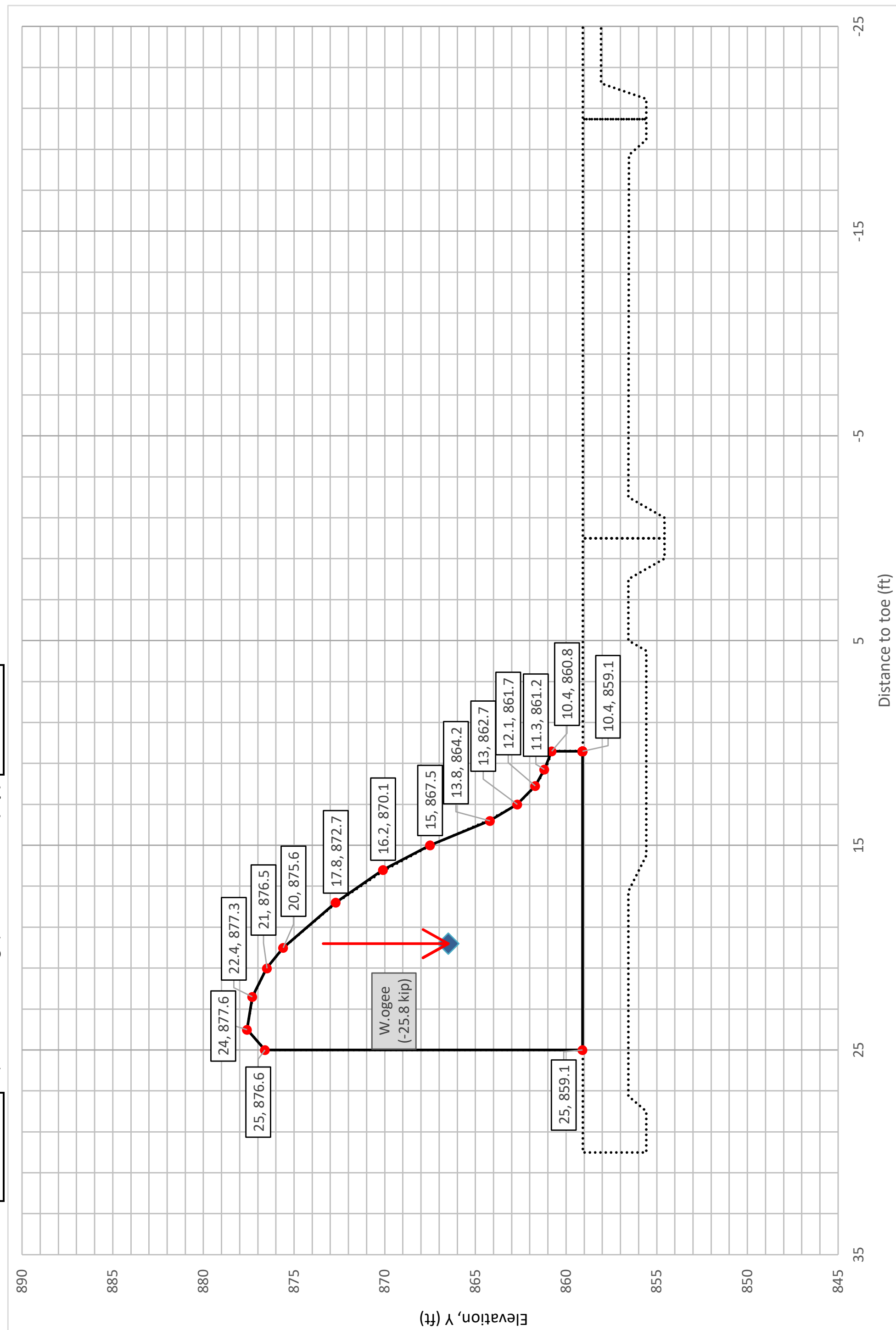
Section Geometry, Centroid Location, and Weight

Force Label:	W.ogee	Description:	Weight of Concrete Ogee

Area Type:	SECTION	
Near Face:	0	ft (Meas. Into Page)
Far Face:	1	ft (Meas. Into Page)
Thickness:	1	ft (Meas. Into Page)

Total Area (sf):	171.8
Volume (CF):	171.8
Unit Weight:	-0.15 kcf
Total Force (kip):	-25.8

Centroid:			
	X:	19.8	(Horiz. Dist. To Toe, ft)
	Y:	866.5	(Elevation, ft)
	Z:	0.5	(Perp. To Analysis Section, ft)



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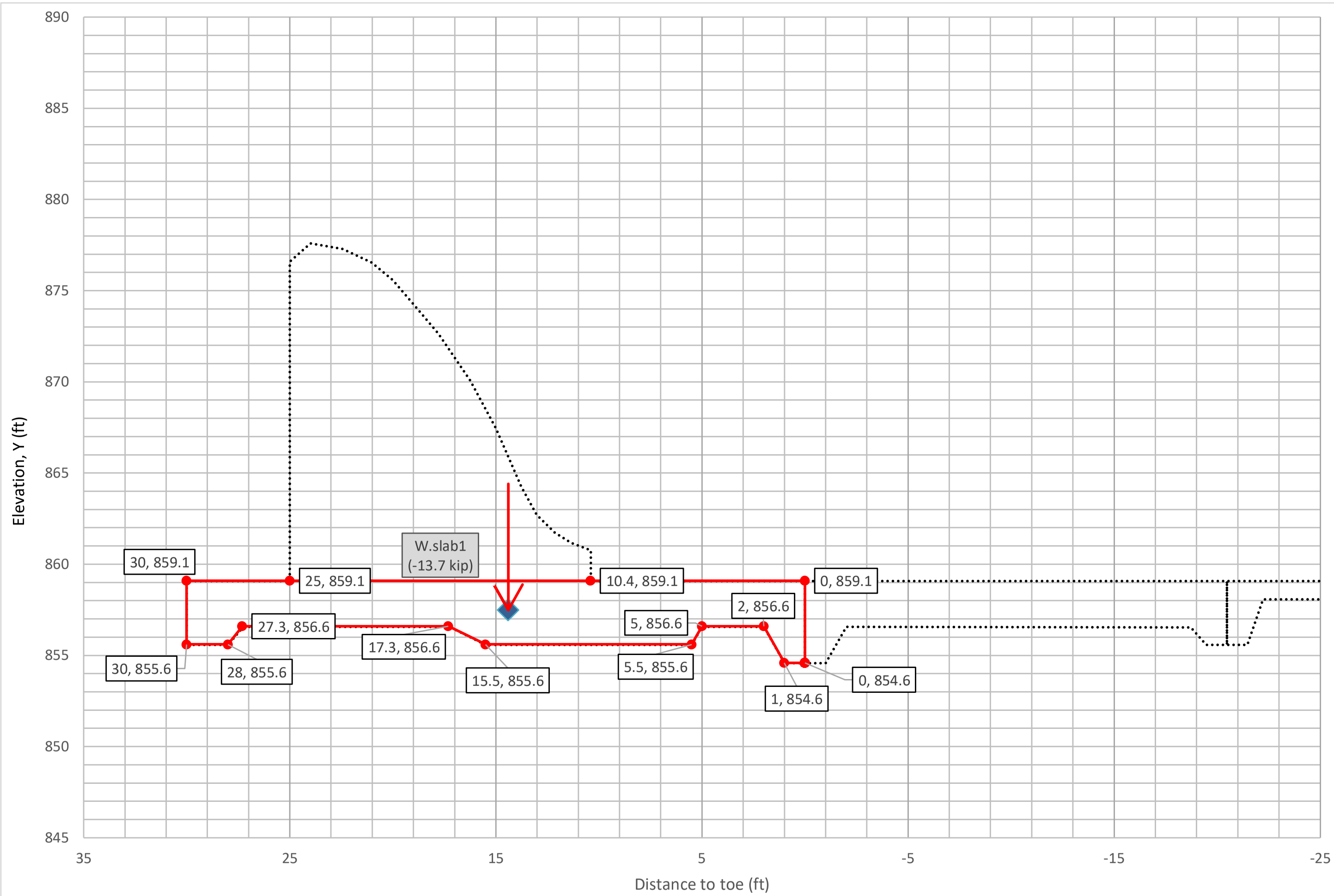
Section Geometry, Centroid Location, and Weight

Force Label: W.slab1 **Description:** Weight of Concrete Slab Beneath Ogee

Area Type:	**SECTION**		**Total Area (sf):**	91.5	**Centroid:** X:	14.4	(Horiz. Dist. To Toe, ft)
Near Face:	0	ft (Meas. Into Page)	**Volume (CF):**	91.5	Y:	857.5	(Elevation, ft)
Far Face:	1	ft (Meas. Into Page)	**Unit Weight:**	-0.15 kcf	Z:	0.5	(Perp. To Analysis Section, ft)
Thickness:	1	ft (Meas. Into Page)	**Total Force (kip):**	-13.7			



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Project No.	2204052	Document No.	N/A
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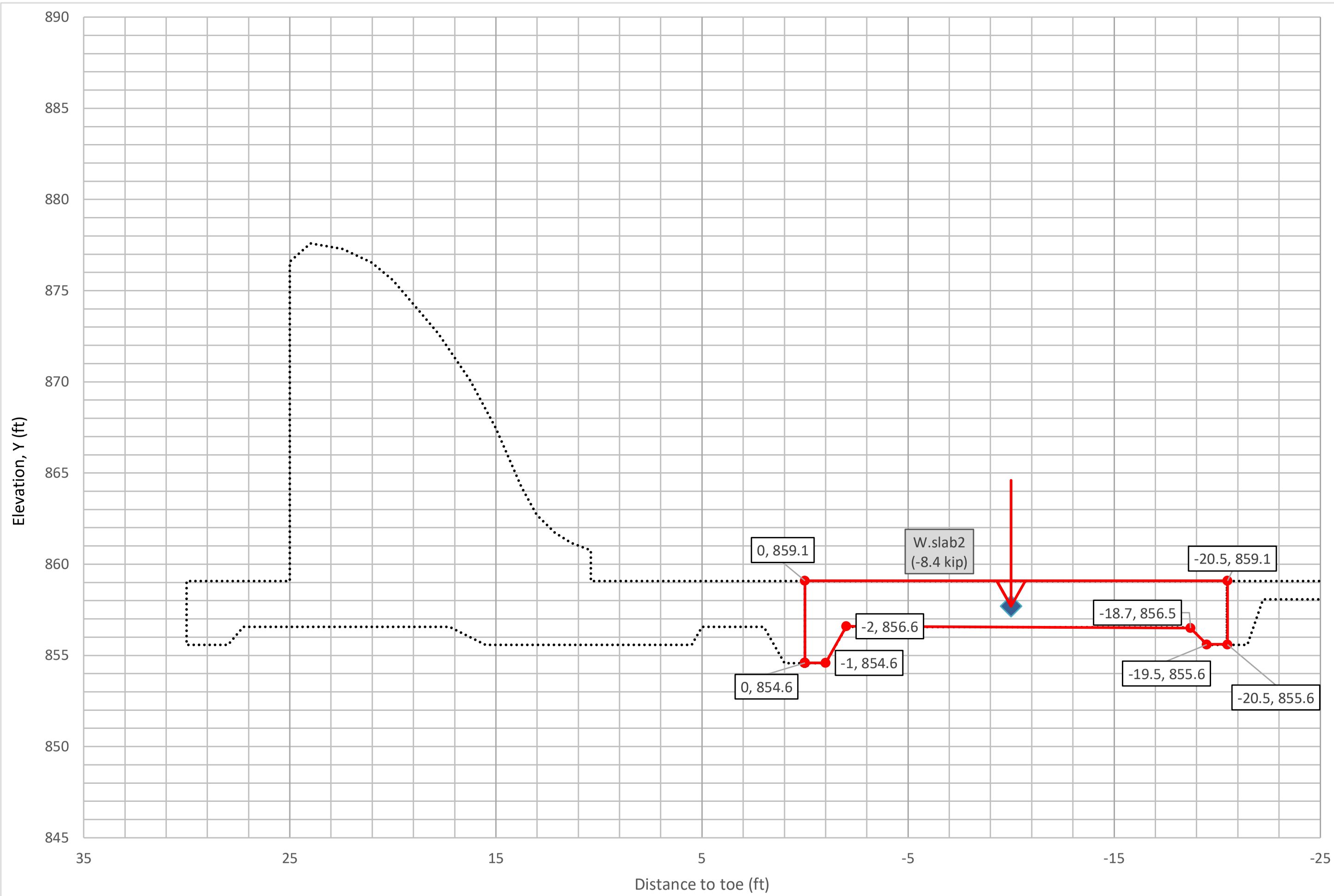
Section Geometry, Centroid Location, and Weight

Force Label: W.slab2 **Description:** Weight of Concrete Slab DS of Ogee

Area Type:	SECTION		**Total Area (sf):**	55.9	**Centroid:** X:	-10.0	(Horiz. Dist. To Toe, ft)
Near Face:	0	ft (Meas. Into Page)	**Volume (CF):**	55.9	Y:	857.7	(Elevation, ft)
Far Face:	1	ft (Meas. Into Page)	**Unit Weight:**	-0.15 kcf	Z:	0.5	(Perp. To Analysis Section, ft)
Thickness:	1	ft (Meas. Into Page)	**Total Force (kip):**	-8.4			



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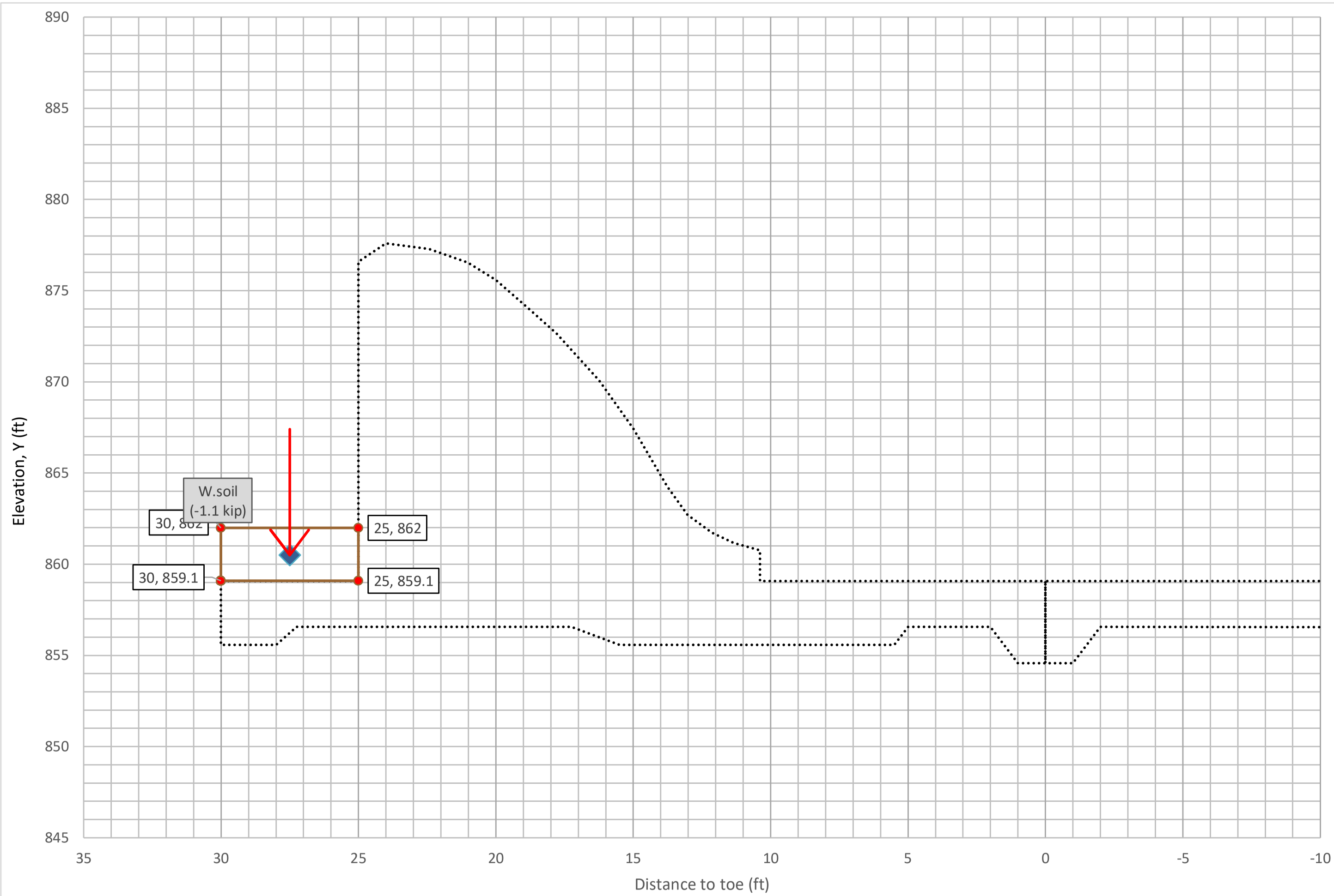
Section Geometry, Centroid Location, and Weight

Force Label: W.soil **Description:** Buoyant Weight of Upstream Soil

Area Type:	SECTION		**Total Area (sf):**	14.7	**Centroid:** X:	27.5	(Horiz. Dist. To Toe, ft)
Near Face:	0	ft (Meas. Into Page)	**Volume (CF):**	14.7	Y:	860.5	(Elevation, ft)
Far Face:	1	ft (Meas. Into Page)	**Unit Weight:**	-0.0776 kcf	Z:	0.5	(Perp. To Analysis Section, ft)
Thickness:	1	ft (Meas. Into Page)	**Total Force (kip):**	-1.1			



CENTROID - NORMAL AND NORMAL + ICE POOL CONDITIONS

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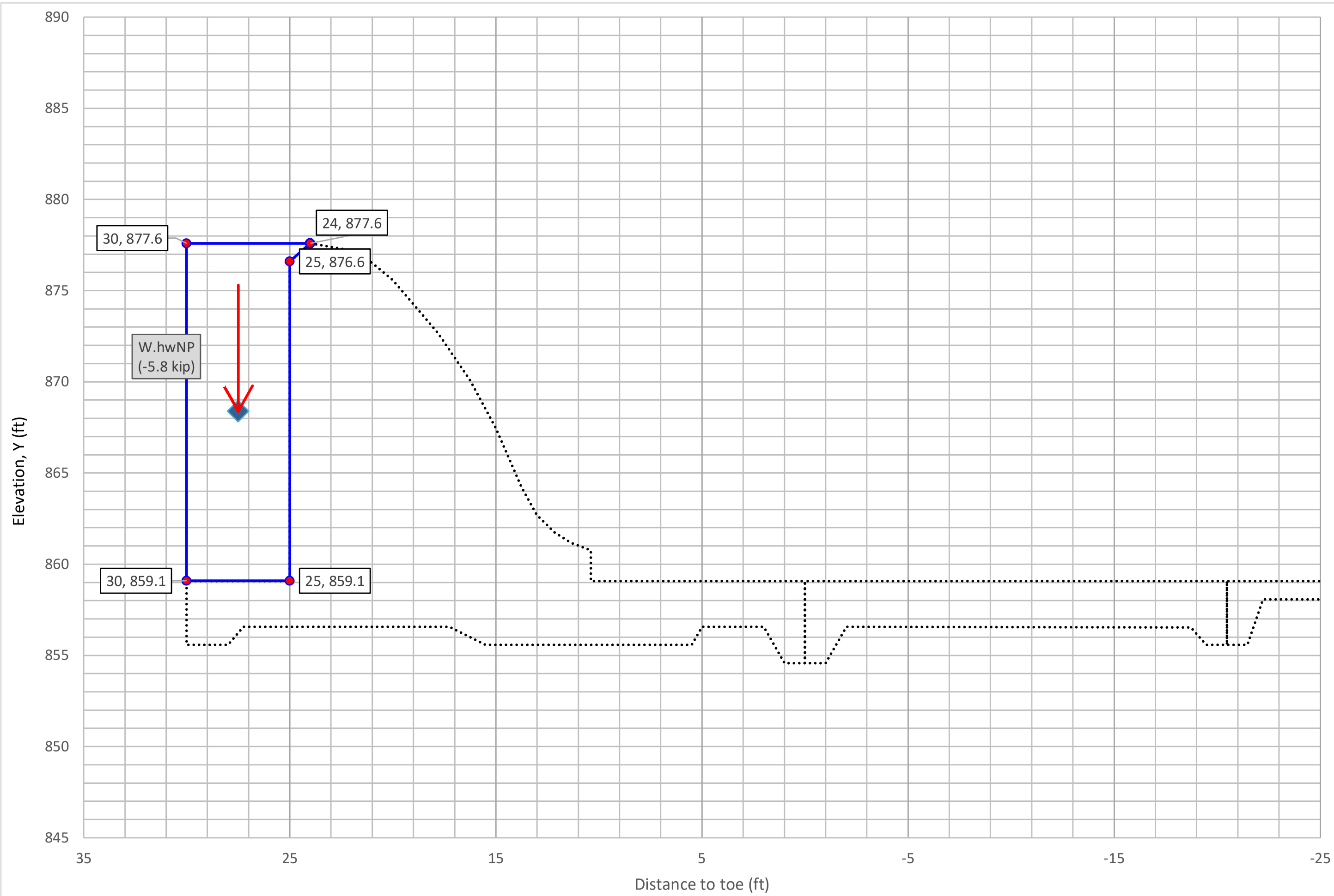
Section Geometry, Centroid Location, and Weight

Force Label: W.hwNP **Description:** Weight of Water Upstream

Area Type:	SECTION		**Total Area (sf):**	93.1	**Centroid:** X:	27.5	(Horiz. Dist. To Toe, ft)
Near Face:	0	ft (Meas. Into Page)	**Volume (CF):**	93.1	Y:	868.4	(Elevation, ft)
Far Face:	1	ft (Meas. Into Page)	**Unit Weight:**	-0.0624 kcf	Z:	0.5	(Perp. To Analysis Section, ft)
Thickness:	1	ft (Meas. Into Page)	**Total Force (kip):**	-5.8			



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	Project	Ford Manchester Dam		Pg. Rev.	0
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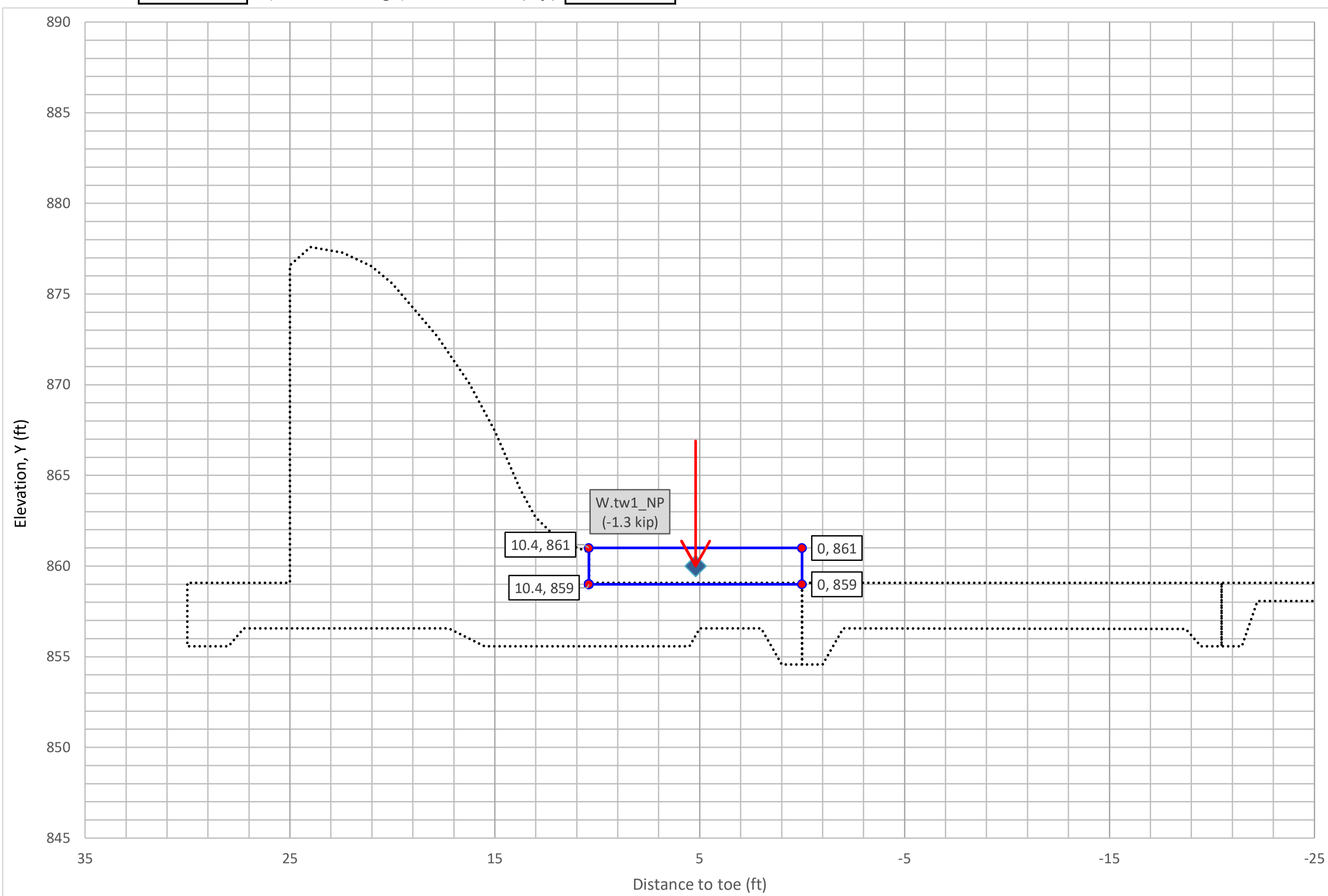
Project No.	2204052	Document No.	N/A		
Subject	Overflow Spillway Stability Analyses				

Section Geometry, Centroid Location, and Weight

Force Label: W.tw1_NP **Description:** Weight of tailwater above Slab1

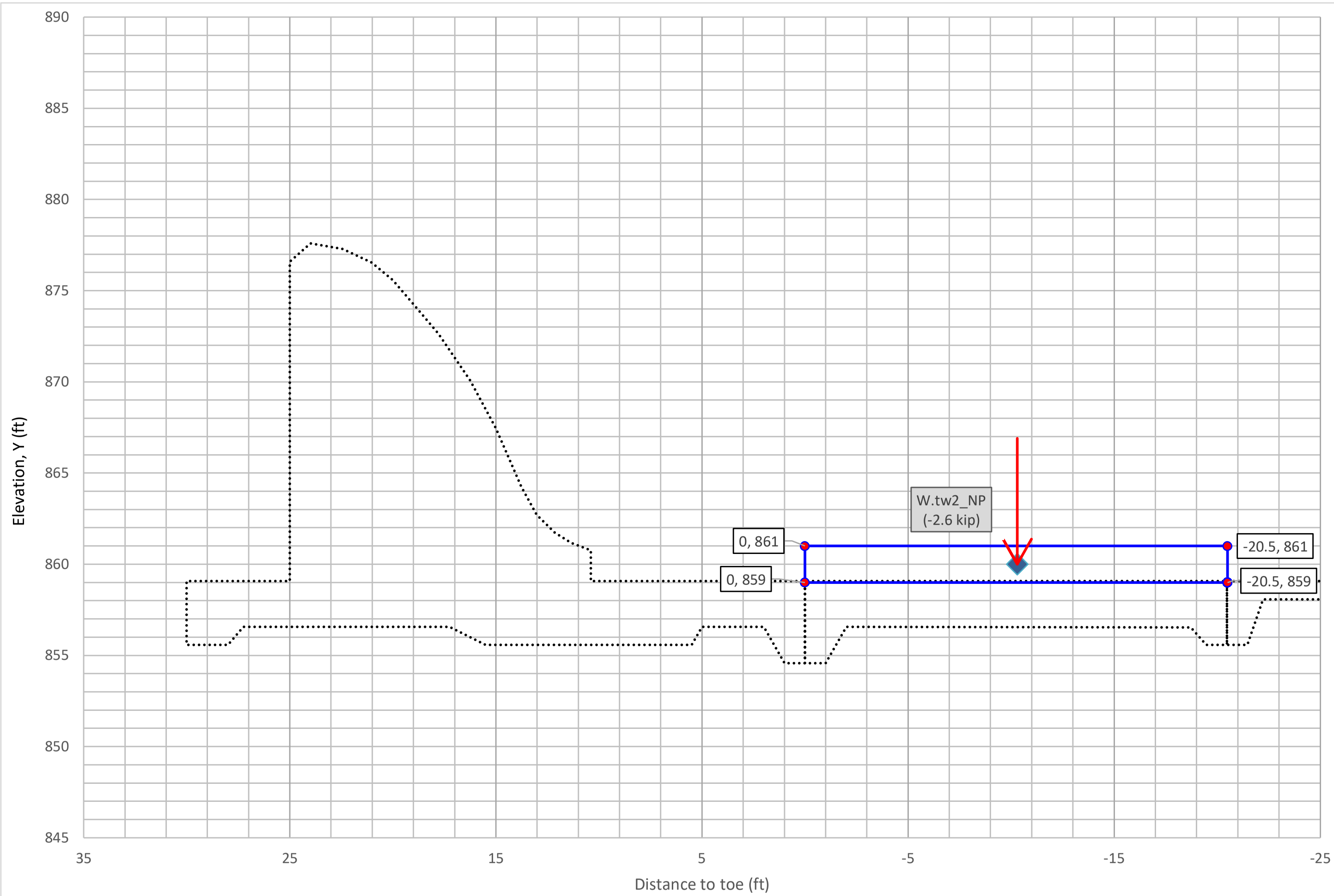
Area Type:	SECTION		**Total Area (sf):**	20.8	**Centroid:** X:	5.2	(Horiz. Dist. To Toe, ft)
Near Face:	0	ft (Meas. Into Page)	**Volume (CF):**	20.8	Y:	860.0	(Elevation, ft)
Far Face:	1	ft (Meas. Into Page)	**Unit Weight:**	-0.0624 kcf	Z:	0.5	(Perp. To Analysis Section, ft)
Thickness:	1	ft (Meas. Into Page)	**Total Force (kip):**	-1.3			



Section Geometry, Centroid Location, and Weight

Force Label: W.tw2_NP **Description:** Weight of tailwater above Slab 2

Area Type:	SECTION		**Total Area (sf):**	41	**Centroid:** X:	-10.3	(Horiz. Dist. To Toe, ft)
Near Face:	0	ft (Meas. Into Page)	**Volume (CF):**	41	Y:	860.0	(Elevation, ft)
Far Face:	1	ft (Meas. Into Page)	**Unit Weight:**	-0.0624 kcf	Z:	0.5	(Perp. To Analysis Section, ft)
Thickness:	1	ft (Meas. Into Page)	**Total Force (kip):**	-2.6			



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CENTROID - FLOOD POOL CONDITION

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Project No.	2204052	Document No.	N/A			
Subject	Overflow Spillway Stability Analyses					

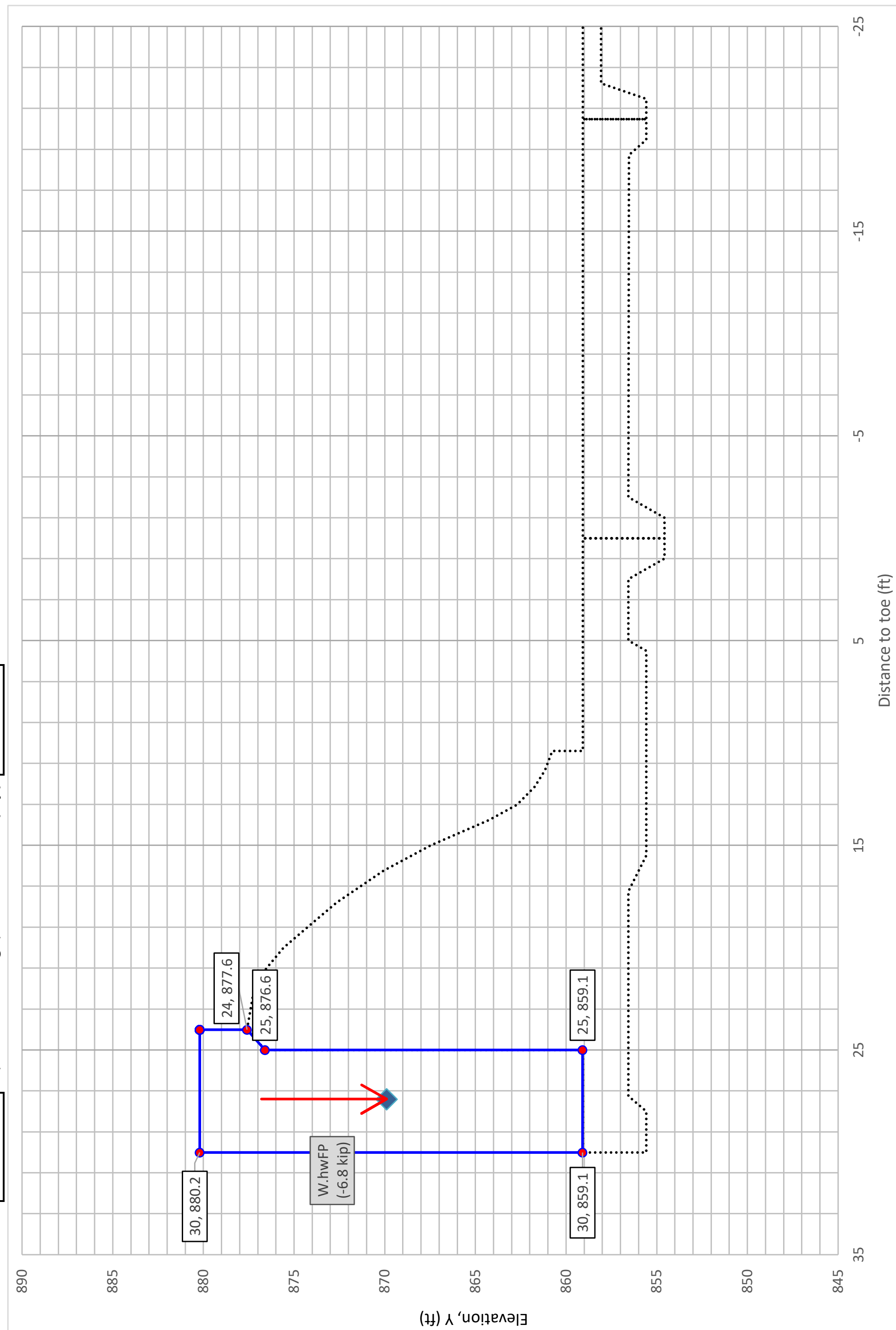
Section Geometry, Centroid Location, and Weight

Force Label: W.hwFP **Description:** Weight of Water Upstream

Area Type:	SECTION	
Near Face:	0	ft (Meas. Into Page)
Far Face:	1	ft (Meas. Into Page)
Thickness:	1	ft (Meas. Into Page)

Total Area (sf):	108.6
Volume (CF):	108.6
Unit Weight:	-0.0624 kcf
Total Force (kip):	-6.8

Centroid:		
X:	27.4	(Horiz. Dist. To Toe, ft)
Y:	869.9	(Elevation, ft)
Z:	0.5	(Perp. To Analysis Section, ft)



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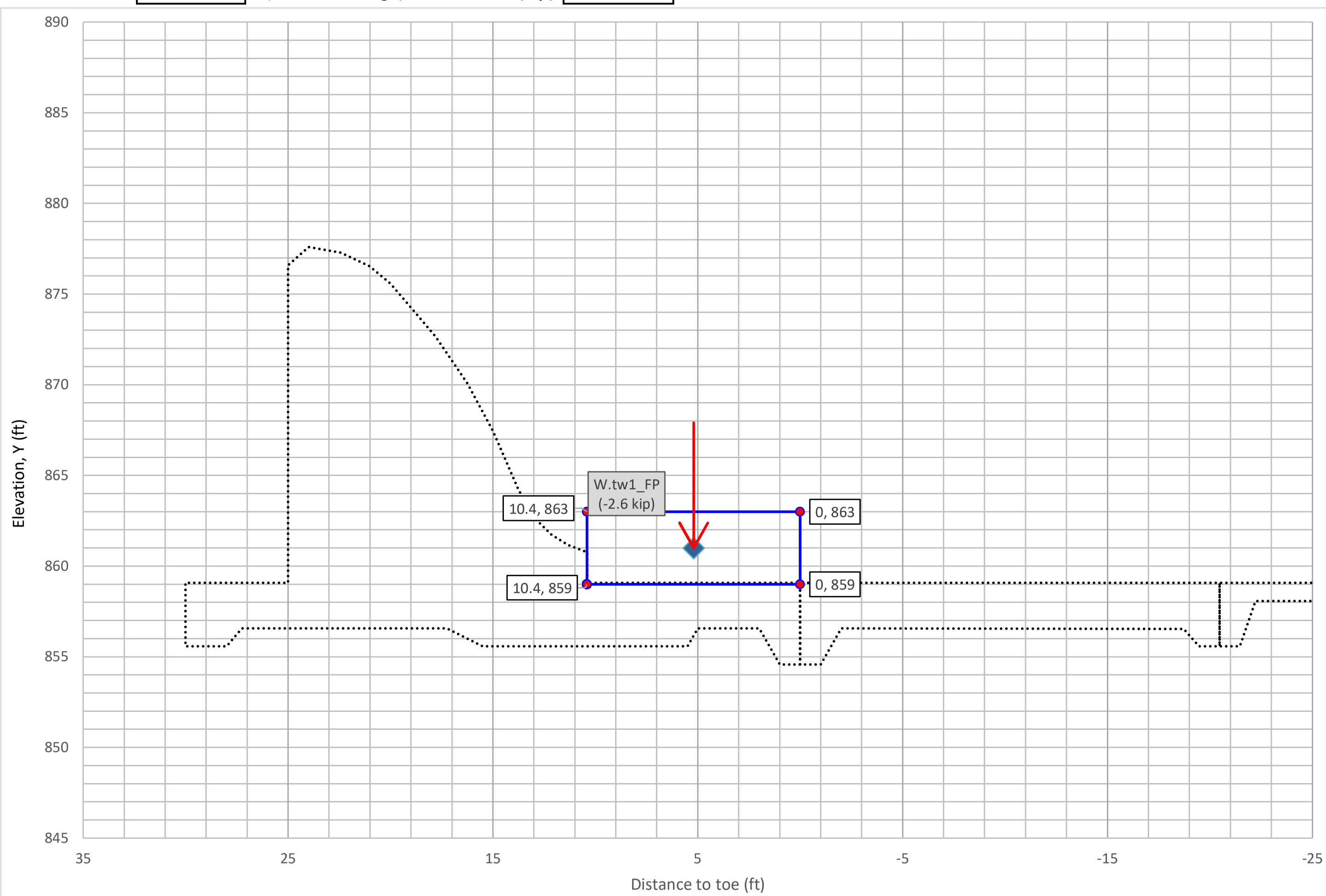
Section Geometry, Centroid Location, and Weight

Force Label: W.tw1_FP **Description:** Weight of tailwater above Slab1

Area Type:	SECTION		**Total Area (sf):**	41.6	**Centroid:**	X:	5.2	(Horiz. Dist. To Toe, ft)
Near Face:	0	ft (Meas. Into Page)	**Volume (CF):**	41.6		Y:	861.0	(Elevation, ft)
Far Face:	1	ft (Meas. Into Page)	**Unit Weight:**	-0.0624 kcf		Z:	0.5	(Perp. To Analysis Section, ft)
Thickness:	1	ft (Meas. Into Page)	**Total Force (kip):**	-2.6				



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Subject	Overflow Spillway Stability Analyses					

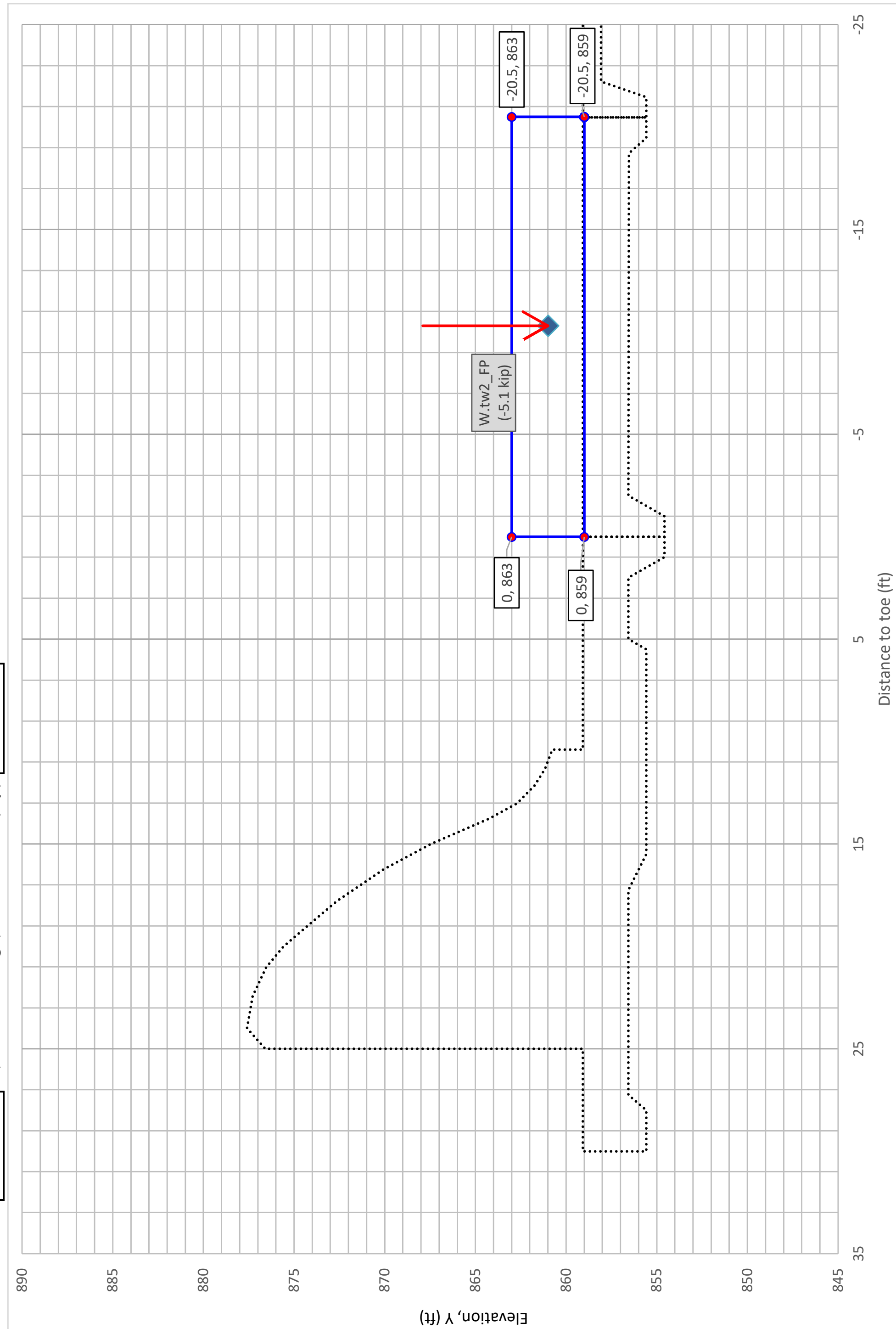
Section Geometry, Centroid Location, and Weight

Force Label:	W.tw2_FP	Description:	Weight of tailwater above Slab 2

Area Type:	SECTION	
Near Face:	0	ft (Meas. Into Page)
Far Face:	1	ft (Meas. Into Page)
Thickness:	1	ft (Meas. Into Page)

Total Area (sf):	82
Volume (CF):	82
Unit Weight:	-0.0624 kcf
Total Force (kip):	-5.1

Centroid:			
	X:	-10.3	(Horiz. Dist. To Toe, ft)
	Y:	861.0	(Elevation, ft)
	Z:	0.5	(Perp. To Analysis Section, ft)



HEADWATER AND TAILWATER LEVELS

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Subject	Overflow Spillway Stability Analyses					

WATER LEVEL ELEVATION SOURCES

Description	ELEVATION NGVD29 (USGS)	ELEVATION NAVD88	SOURCE
HEADWATER LEVELS			
FEMA 500 HW	880.3	879.9	FEMA FIS (April 3, 2012)
200 YEAR HW	880.17	879.77	2022 EGLE Inspection Report = 2.65ft +877.52 crest el. = 880.17 (NGVD)
FEMA 100 YEAR HW	879.8	879.4	FEMA FIS (April 3, 2012)
NORMAL HW	877.52	877.12	= Spillway Crest Elevation, 2019 EAP
TAILWATER LEVELS			
FEMA 500 TW	863.1	862.7	FEMA FIS (April 3, 2012)
200 YEAR TW	Unknown, See following computations		
FEMA 100 YEAR TW	862.6	862.2	FEMA FIS (April 3, 2012)
NORMAL TW	861	860.6	= Top of weir. Design drawings.

Notes: - NGVD = NVD88 + 0.4 (Ref. FIS, 2012)
- 2019 EAP references USGS datum as dam crest elevation 877.5 feet.

Source Values

	∆ HEAD (FT)	
FEMA 500 YEAR:	17.2	<-- UNIFORM HEADLOSS ACROSS DAM FOR BOTH EVENTS.
FEMA 100 YEAR:	17.2	<--
200 YR HW - 100 YR HW:	0.4	
200 YEAR TW:	863.0	= 100 YEAR TW + 0.4 FT

WATER ELEVATIONS FOR ANALYSES

WSEL NORMAL OPERATION CONDITIONS FOR ANALYSIS (NGVD29):

HW:	877.52	EQUAL TO CREST OF SPILLWAY
TW:	861.0	EQUAL TO TOP OF WEIR IN SPILLWAY APRON. TW AT POWERHOUSE = 857.7 FT

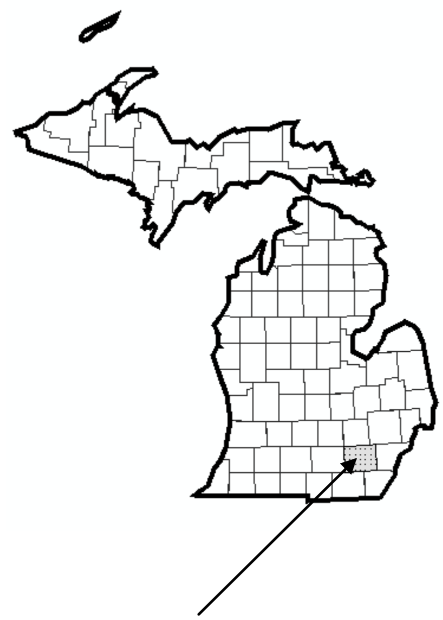
WSEL FOR 200 YEAR FLOOD ANALYSIS (NGVD29):

HW:	880.2	2022 EGLE REPORT
TW:	863.0	100 YEAR TW + 0.4 FT

EXCERPT FROM:

FLOOD INSURANCE STUDY

WASHTENAW COUNTY, MICHIGAN
(ALL JURISDICTIONS)



VOLUME 2 OF 2

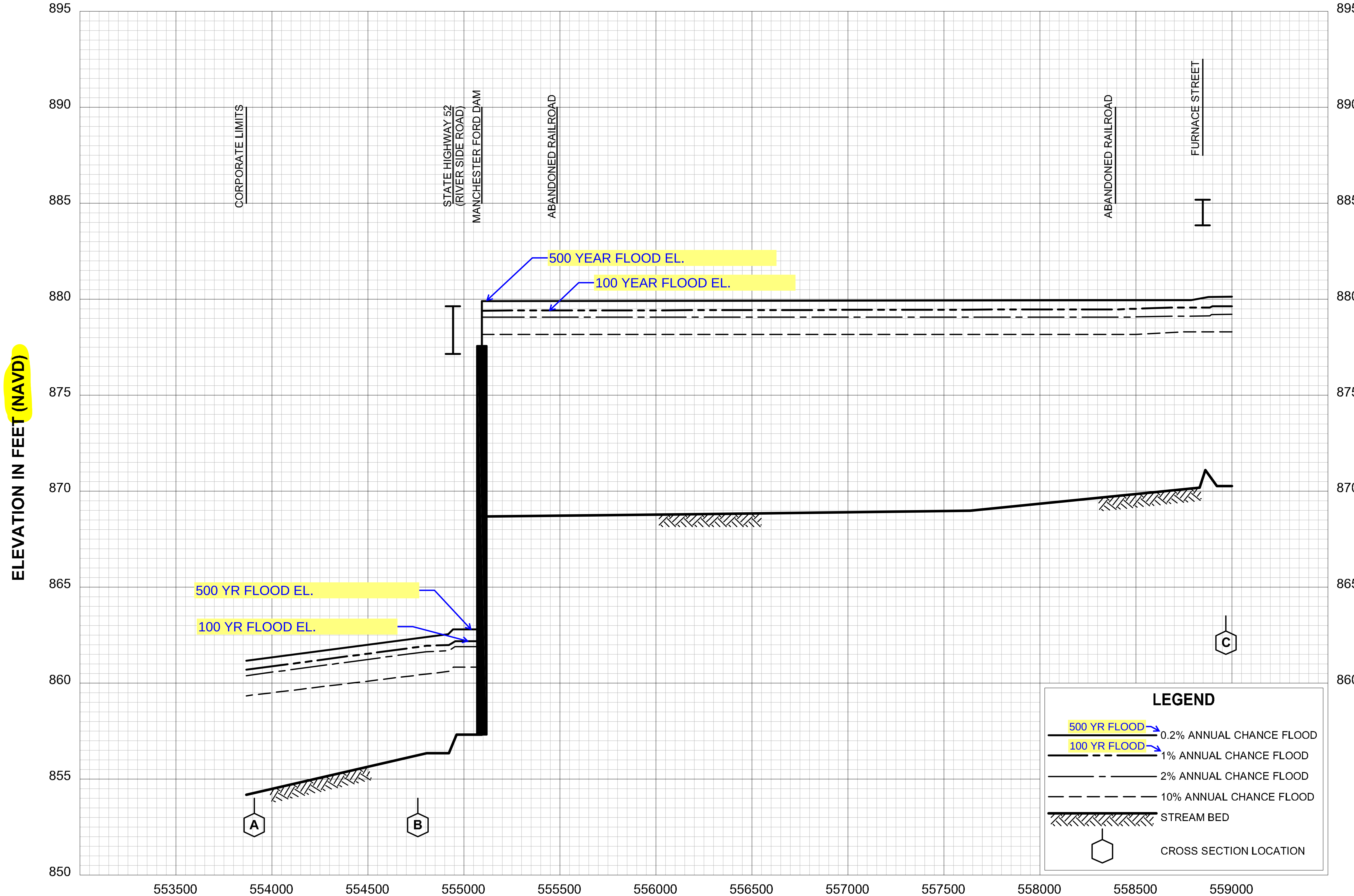
Community Name	Community Number	Community Name	Community Number
Ann Arbor, Charter Township of	260535	Milan, City of	260151
Ann Arbor, City of	260213	Northfield, Township of	260635
Augusta, Township of	260627	Pittsfield, Charter Township of	260623
Barton Hills, Village of	261154	Salem, Township of	260636
Bridgewater, Township of*	261786	Saline, City of	260215
Chelsea, City of	260599	Saline, Township of	261792
Dexter, Township of	260536	Scio, Township of	260537
Dexter , Village of	260600	Sharon, Township of*	260538
Freedom, Township of*	261787	Superior, Township of	260540
Lima, Township of	261788	Sylvan, Township of	261793
Lodi, Township of	261789	Webster, Township of	261785
Lyndon, Township of	261790	York, Charter Township of	260541
Manchester, Township of	261791	Ypsilanti, Charter Township of	260542
Manchester, Village of	260316	Ypsilanti, City of	260216

* No Special Flood Hazard Areas Identified

Effective
April 3, 2012

Federal Emergency Management Agency

FLOOD INSURANCE STUDY NUMBER
26161CV002A



BY: P. Grodecki 11/27/2023
CHK: E. Baffoe 11/30/2023
APP: M. Guirguis 12/01/2023

FLOOD PROFILES

RIVER RAISIN

FEDERAL EMERGENCY MANAGEMENT AGENCY

WASHTENAW COUNTY, MI

ALL JURISDICTIONS

DAM SAFETY INSPECTION REPORT
FORD MANCHESTER DAM – DAM ID NO. 391
RIVER RAISIN
WASHTENAW COUNTY – SECTION 1, T 04S, R 03E

OWNER(S)/OPERATOR(S):	Village of Manchester 912 City Road PO Box 485 Manchester, MI 48158 (734) 428-7877
HAZARD POTENTIAL CLASSIFICATION:	High
INSPECTION DATE:	May 17, 2022
REPORT DATE:	August 4, 2022

PREPARED AND INSPECTED BY:

Thomas Horak, E.I.T.
Dam Safety Unit
Water Resources Division
Dept. of Environment, Great Lakes, and Energy
P.O. Box 30458
Lansing, Michigan 48909
517-231-8594

Lucas A. Trumble, P.E.
Dam Safety Unit
Water Resources Division
Department of Environment, Great Lakes, and Energy
P.O. Box 30458
Lansing, Michigan 48909
517-420-8923

STATE OF MICHIGAN
LUCAS A. TRUMBLE
ENGINEER NO. 58295
LICENSED PROFESSIONAL ENGINEER

analysis of the spillway structure or major dam repairs have been completed, so it is recommended that such an analysis be completed within the next year and any necessary repairs be implemented as recommended in that report.

HYDROLOGY AND HYDRAULICS

The contributing drainage area to the River Raisin at the Ford Manchester Dam is approximately 149 square miles. The design discharge for this high hazard potential dam is the 0.5-percent annual chance (200-year) flood discharge, which is estimated to be 1,300 cubic feet per second (cfs).

2.65+877.52 = 880.17 ft (NGVD)

Using the weir equation with an ogee weir coefficient of 3.8, the 80.5-foot long spillway can pass the design flood inflow with approximately 2.65 feet of head. This leaves approximately 1.85 feet of freeboard at the earthen embankments. Therefore, the dam is considered to have adequate spillway capacity to safely convey the design flood.

Copies of the hydraulic calculations used to make this determination are on file with the Dam Safety Program.

OPERATION AND MAINTENANCE

Operation of the dam is by staff of the Village of Manchester. According to our records, a written O&M Plan has never been prepared for this dam. An O&M Plan should be prepared that addresses day-to-day operation, as well as operation during flood conditions. This plan should be reviewed regularly, with updated copies provided to the Dam Safety Program.

EMERGENCY ACTION PLAN

The Ford Manchester Dam has been assigned a high hazard potential rating. As such, the owner is required under Part 315 to prepare, and keep up to date, an Emergency Action Plan (EAP) for the dam. A written EAP was originally prepared in 1995. An updated copy of the EAP was provided to this office on July 10, 2019. The owner shall review, and update as necessary, the dam's EAP in coordination with Washtenaw County Emergency Management. The results of this review, and any updates, should be provided to the Dam Safety Program by December 31, 2022.

APPENDICES

A location map, inspection photographs, hydraulic calculations, and 2022 EGLE estimated flood flows are attached.

OTHER COMPUTATIONS

Slab 1

Representative Base Elevation for Analysis:

U/S Edge Dist. To Toe (ft)	Elev. (ft)	Length (ft)	Elev * Length
30	855.5	2.4	2053.2
27.6	856.5	11.2	9592.8
16.4	855.5	11.2	9538.8
5.25	856.5	3.8	3211.9
1.5	854.5	1.5	1281.8
0	-	-	-
TOTALS:		30	25678.45

Weighted Base El. = **855.9** ft = 25678.5 / 30

Slab 2

Representative Base Elevation for Analysis:

U/S Edge Dist. To Toe (ft)	Elev. (ft)	Length (ft)	Elev * Length
20.5	855.5	1.4	1197.7
19.1	856.5	17.6	15074.4
1.5	854.5	1.5	1281.8
0			
TOTALS:		20.5	17553.85

Weighted Base El. = **856.3** ft = 17553.9 / 20.5

ULTIMATE BEARING CAPACITY

Bearing Capacity

Reference:

- USACE EM 1110-1-1905 - Bearing Capacity of Soils. October 30, 1992.

Bearing Capacity Equation: *Assumes cohesionless soil. No cohesive parameters.*

$$q_{ult} := 0.5 \cdot B \cdot \gamma_b \cdot N_\gamma \cdot \zeta_{\gamma s} \cdot \zeta_{\gamma d} + (\gamma_b \cdot D_f) \cdot N_q \cdot \zeta_{qs.} \cdot \zeta_{qd}$$

$\gamma_b := (135 - 62.4)pcf = 72.60 \cdot pcf$ Buoyant Unit Weight of Soil Assumed

$B := 30ft$ Width of Base

$D_f := 2ft$ Depth of bo2om of base below grade

$\phi := 35deg$ Founda5on Soil Internal Fric5on Angle

Meyerhof Bearing Capacity Factors

$$N_\phi := \tan\left(45deg + \frac{\phi}{2}\right)^2 = 3.69$$

$$N_q := N_\phi \cdot e^{\pi \cdot \tan(\phi)} = 33.30$$

$$N_c := (N_q - 1) \cdot \cot(\phi) = 46.12$$

$$N_\gamma := (N_q - 1) \cdot \tan(1.4 \cdot \phi) = 37.15$$

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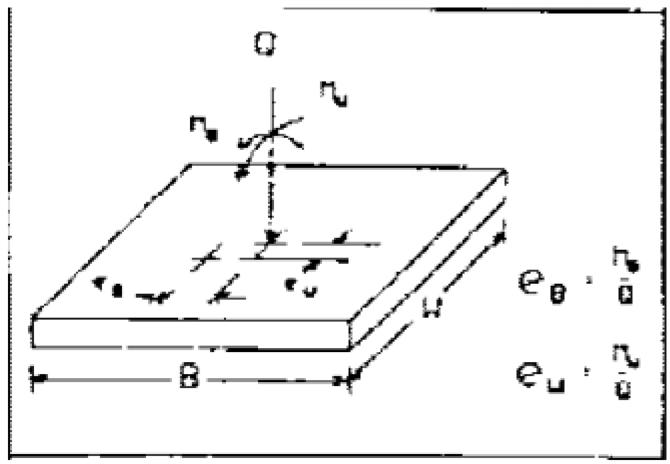
Modification Factors - Foundation Shape and Eccentricity:

Base Geometry:

$B = 30.00\,ft$ Base Width

$W := 80ft$ Side Length (into page)



Eccentricity (Meas. from base center):

$e_B := 0ft$

$e_W := 0ft$

Assume no eccentric loading in either direc5on

Effective Base:

$B' := B - 2 \cdot e_B = 30.00\,ft$

$W' := W - 2 \cdot e_W = 80.00\,ft$

Wedge Modification Factor:

$$\zeta_{\gamma s} := 1 + 0.1 \cdot N_\phi \cdot \frac{B'}{W'} = 1.14$$

Surcharge Load Modification Factor:

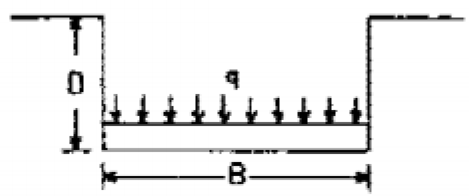
$$\zeta_{qs} := 1 + 0.1 \cdot N_\phi \cdot \frac{B'}{W'} = 1.14$$

Modification Factors - Foundation Depth:

Base Geometry:

$B = 30.00\,ft$ Base Width

$D_f = 2.00\,ft$ Bo2om of foo5ng depth



Wedge Modification Factor:

$$\zeta_{\gamma d} := 1 + 0.1 \cdot N_\phi^{0.5} \cdot \frac{D_f}{B} = 1.01$$

Surcharge Load Modification Factor:

$$\zeta_{qd} := 1 + 0.1 \cdot N_\phi^{0.5} \cdot \frac{D_f}{B} = 1.01$$

Computed Bearing Capacity

$$q_{ult} := 0.5 \cdot \gamma_b \cdot B \cdot N_\gamma \cdot \zeta_{\gamma s} \cdot \zeta_{\gamma d} + \gamma_b \cdot D_f \cdot N_q \cdot \zeta_{qs} \cdot \zeta_{qd} = 52.22 \cdot ksf$$

CECW-EG Engineer Manual 1110-1-1905	Department of the Army U.S. Army Corps of Engineers Washington, DC 20314-1000	EM 1110-1-1905 30 October 1992
	Engineering and Design BEARING CAPACITY OF SOILS	
	Distribution Restriction Statement Approved for public release; distribution is unlimited.	

a. **General Equation.** The ultimate bearing capacity of the foundation shown in Figure 1-6 can be determined using the general bearing capacity Equation 1-1

$$q_u = cN_c\zeta_c + \frac{1}{2}B'\gamma'_H N_\gamma \zeta_\gamma + \sigma'_D N_q \zeta_q \tag{4-1}$$

where

- q_u = ultimate bearing capacity, ksf
- c = unit soil cohesion, ksf
- B' = minimum effective width of foundation $B - 2e_B$, ft
- e_B = eccentricity parallel with foundation width B, M_B/Q, ft
- M_B = bending moment parallel with width B, kips-ft
- Q = vertical load applied on foundation, kips
- γ'_H = effective unit weight beneath foundation base within the failure zone, kips/ft^3
- σ'_D = effective soil or surcharge pressure at the foundation depth D, $\gamma'_D \cdot D$, ksf
- γ'_D = effective unit weight of soil from ground surface to foundation depth, kips/ft^3
- D = foundation depth, ft
- N_c, N_γ, N_q = dimensionless bearing capacity factors of cohesion c, soil weight in the failure wedge, and surcharge q terms
- $\zeta_c, \zeta_\gamma, \zeta_q$ = dimensionless correction factors of cohesion c, soil weight in the failure wedge, and surcharge q accounting for foundation geometry and soil type

c. **Meyerhof Model.** This solution considers correction factors for eccentricity, load inclination, and foundation depth. The influence of the shear strength of soil above the base of the foundation is considered in this solution. Therefore, beneficial effects of the foundation depth can be included in the analysis. Assumptions include use of a shape factor ζ_q for surcharge, soil at plastic equilibrium, and a log spiral failure surface that includes shear above the base of the foundation. The angle $\psi = 45 + \phi/2$ was used for determination of N_γ. Table 4-3 illustrates the Meyerhof dimensionless bearing capacity and correction factors required for solution of Equation 4-1 (Meyerhof 1963).

(1) **Bearing Capacity Factors.** Table 4-4 provides the bearing capacity factors in 2-degree intervals.

(2) **Correction Factors.** Correction factors are given by

Cohesion: $\zeta_c = \zeta_{cs} \cdot \zeta_{ci} \cdot \zeta_{cd}$
Wedge: $\zeta_\gamma = \zeta_{\gamma s} \cdot \zeta_{\gamma i} \cdot \zeta_{\gamma d}$
Surcharge: $\zeta_q = \zeta_{qs} \cdot \zeta_{qi} \cdot \zeta_{qd}$

where subscript s indicates shape with eccentricity, subscript i indicates inclined loading, and d indicates foundation depth.

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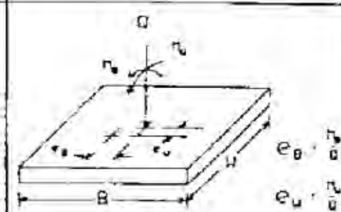
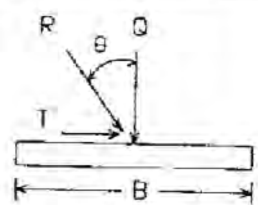
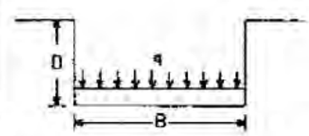
TABLE 4-3

Meyerhof Dimensionless Bearing Capacity and Correction Factors (Data from Meyerhof 1953; Meyerhof 1963)

FACTOR			COHESION (c)	WEDGE (γ)	SURCHARGE (q)	DIAGRAM
BEARING CAPACITY N			N_c	N_γ	N_q	
		$\phi = 0$	5.14	0.00	1.00	
		$\phi > 0$	$(N_q-1)\cot\phi$	$(N_q-1)\tan(1.4\phi)$	$N_\phi e^{\pi\tan\phi}$	
CORRECTION ζ	FOUNDATION SHAPE WITH ECCENTRICITY s		ζ_{cs}	$\zeta_{\gamma s}$	ζ_{qs}	
		$\phi = 0$	$1 + 0.2N_\phi \frac{B'}{W'}$	1.0	1.0	
		$\phi > 10$	"	$1 + 0.1N_\phi \frac{B'}{W'}$	$1 + 0.1N_\phi \frac{B'}{W'}$	
		$0 < \phi \leq 10$	"	Linear Interpolation Between $\phi = 0$ and $\phi = 10$ Degrees		
	INCLINED LOADING i		ζ_{ci}	$\zeta_{\gamma i}$	ζ_{qi}	
		$\phi = 0$	$\left[1 - \frac{\theta}{90}\right]$	1.0	$\left[1 - \frac{\theta}{90}\right]$	
		$\phi > 0$	$\left[1 - \frac{\theta}{90}\right]^2$	$\theta \leq \phi$ $\left[1 - \frac{\theta}{\phi}\right]^2$	$\left[1 - \frac{\theta}{90}\right]^2$	
				$\theta > \phi$ 0.0		
	FOUNDATION DEPTH d		ζ_{cd}	$\zeta_{\gamma d}$	ζ_{qd}	
		$\phi = 0$	$1 + 0.2(N_\phi)^{1/2} \cdot \frac{D}{B}$	1.0	1.0	
		$\phi > 0$	"	$1 + 0.1(N_\phi)^{1/2} \cdot \frac{D}{B}$	$1 + 0.1(N_\phi)^{1/2} \cdot \frac{D}{B}$	
		$0 < \phi \leq 10$	"	Linear Interpolation Between $\phi = 0$ and $\phi = 10$ Degrees		

Note: Eccentricity and inclined loading correction factors may not be used simultaneously; factors not used are unity

Nomenclature:
- ϕ = angle of internal friction, degrees
- N_ϕ = $\tan^2(45 + \phi/2)$
- B' = effective width of foundation, $B - 2e_B$, ft
- W' = effective lateral length of foundation, $W - 2e_W$, ft
- B = foundation width, ft
- W = foundation lateral length, ft
- D = foundation depth, ft
- Q = vertical load on foundation, qBW, kips
- q = bearing pressure on foundations, ksf
- T = horizontal load on foundation, right +, kips
- R = resultant load on foundation, $(Q^2 + T^2)^{1/2}$
- θ = angle of resultant load with vertical axis, $\cos^{-1}(Q/R)$, degrees
- e_B = eccentricity parallel with B, M_B/Q
- e_W = eccentricity parallel with W, M_W/Q
- M_B = bending moment parallel with B, kips-ft
- M_W = bending moment parallel with W, kips-ft

TABLE 4-4

Meyerhof, Hansen, and Vesic Dimensionless Bearing Capacity Factors

				N_γ		
ϕ	N_ϕ	N_c	N_q	Meyerhof	Hansen	Vesic
0	1.00	5.14	1.00	0.00	0.00	0.00
2	1.07	5.63	1.20	0.01	0.01	0.15
4	1.15	6.18	1.43	0.04	0.05	0.34
6	1.23	6.81	1.72	0.11	0.11	0.57
8	1.32	7.53	2.06	0.21	0.22	0.86
10	1.42	8.34	2.47	0.37	0.39	1.22
12	1.52	9.28	2.97	0.60	0.63	1.69
14	1.64	10.37	3.59	0.92	0.97	2.29
16	1.76	11.63	4.34	1.37	1.43	3.06
18	1.89	13.10	5.26	2.00	2.08	4.07
20	2.04	14.83	6.40	2.87	2.95	5.39
22	2.20	16.88	7.82	4.07	4.13	7.13
24	2.37	19.32	9.60	5.72	5.75	9.44
26	2.56	22.25	11.85	8.00	7.94	12.54
28	2.77	25.80	14.72	11.19	10.94	16.72
30	3.00	30.14	18.40	15.67	15.07	22.40
32	3.25	35.49	23.18	22.02	20.79	30.21
34	3.54	42.16	29.44	31.15	28.77	41.06
36	3.85	50.59	37.75	44.43	40.05	56.31
38	4.20	61.35	48.93	64.07	56.17	78.02
40	4.60	75.31	64.19	93.69	79.54	109.41
42	5.04	93.71	85.37	139.32	113.95	155.54
44	5.55	118.37	115.31	211.41	165.58	224.63
46	6.13	152.10	158.50	328.73	244.64	330.33
48	6.79	199.26	222.30	526.44	368.88	495.99
50	7.55	266.88	319.05	873.84	568.56	762.85

Appendix E

Concrete Repair Drawings

FORD MANCHESTER DAM

MODIFICATIONS AND CONCRETE REPAIRS
EGLE DAM ID NO. 391
MANCHESTER, MI

STATE or COUNTY MAP
(NOT TO SCALE)

SOURCE:
{2023 MICROSOFT CORPORATION 2023 MAXAR CNES 2023 DISTRIBUTION AIRBUS DS}

SITE LOCATION MAP
(NOT TO SCALE)

SHEET LIST

Sheet Number	Sheet Title	Sheet Description
1	S-0	COVER
2	S-1.0	DESIGN NOTES AND SYMBOLOGY
3	S-2.0	EXISTING CONDITIONS
4	S-3.0	LEFT WALL REPAIR
5	S-3.1	RIGHT WALL REPAIR
6	S-3.2	UPSTREAM SPILLWAY REPAIRS
8	S-3.3	INTAKE REPAIRS
9	S-3.4	SEATING WALL REPAIRS
10	S-4.0	DAM MODIFICATIONS
11	S-5.0	REPAIR DETAILS
12	S-6.0	SPECIFICATIONS

PREPARED FOR:

VILLAGE OF MANCHESTER
912 CITY ROAD
MANCHESTER, MI 48158
(734) 428-7877

PREPARED BY:

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GEI Consultants

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NO.	DATE	ISSUE/REVISION	APP
0	2/12/2024	FOR REVIEW	MG

SHEET NO.
S-0

REV NO.
0

GEI PROJECT NO. 2204052

SURVEY DATUM INFORMATION

VERTICAL DATUM: NATIONAL GEODETIC VERTICAL DATUM OF 1929 (NGVD 1929)?
HORIZONTAL DATUM: NORTH AMERICAN DATUM OF 1983 (NAD83), STATE PLANE MICHIGAN
EXISTING SITE CONDITIONS DEVELOPED PRIMARILY FROM HISTORICAL DRAWINGS PROVIDED BY VILLAGE OF MANCHESTER.
CONTROL MONUMENTS ON-SITE SHALL BE REFERRED TO CONFIRM HORIZONTAL AND VERTICAL MEASUREMENTS.

DESIGN PARAMETERS

- NORMAL RESERVOIR ELEVATION 877.52
- NORMAL TAILWATER ELEVATION 857.7
- INFLOW DESIGN FLOOD (IDF) HEADWATER ELEVATION 880.2
- INFLOW DESIGN FLOOD (IDF) TAILWATER ELEVATION 863.0

REFERENCE DRAWINGS DOCUMENTS & DRAWINGS

REPORTS

- 1978 USACE DAM INSPECTION REPORT
- 2022 EGLE DAM INSPECTION REPORT
- 2019 EMERGENCY ACTION PLAN
- 2023 J.F. BRENNAN COMPANY, INC. DIVE INSPECTION REPORT

DRAWINGS

- CONSTRUCTION DRAWINGS, 1939
  - DA-2
  - DA-8
  - 1939 CONSTRUCTION DRAWINGS FIGURES 6 THROUGH 10

1 / G-02 DETAIL
SCALE: NTS

DETAIL TITLE. THE NUMBER "1" REFERS TO THE DETAIL DESIGNATION. THE NUMBER "G-02" REFERS TO THE DRAWING NUMBER WHERE THE DETAIL IS CALLED OUT.

A / G-02 SECTION
SCALE: NTS

SECTION TITLE. THE LETTER "A" REFERS TO THE SECTION DESIGNATION. THE NUMBER "G-02" REFERS TO THE DRAWING NUMBER WHERE THE SECTION IS CALLED OUT.

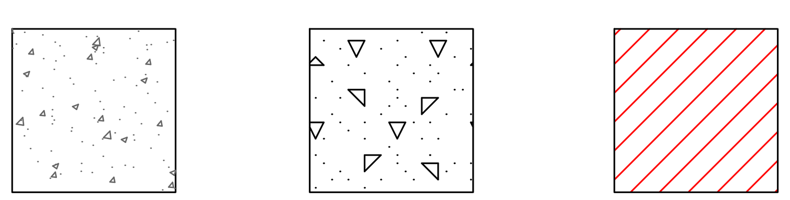
A / G-02

SECTION LOCATION. THE LETTER "A" REFERS TO THE SECTION DESIGNATION. THE NUMBER "G-02" REFERS TO THE DRAWING NUMBER WHERE THE SECTION IS SHOWN.

1 / G-02

DETAIL LOCATION. THE NUMBER "1" REFERS TO THE DETAIL DESIGNATION. THE NUMBER "G-02" REFERS TO THE DRAWING NUMBER WHERE THE DETAIL IS SHOWN.

HATCH LEGEND:



DESIGN REFERENCE STANDARDS

- (USBR, 1987) UNITED STATES DEPARTMENT OF THE INTERIOR, BUREAU OF RECLAMATION, "DESIGN OF SMALL DAMS", 1987.
- (USACE, 1995) UNITED STATES ARMY CORPS OF ENGINEERS, ENGINEERING AND DESIGN, "CONSTRUCTION CONTROL FOR EARTH AND ROCK-FILL DAMS", EM 1110-2-1911, 1995.
- (USACE, 2016) UNITED STATES ARMY CORPS OF ENGINEERS, ENGINEERING AND DESIGN, "STRENGTH DESIGN FOR REINFORCED CONCRETE HYDRAULIC STRUCTURES", EM 1110-2-2104, 2016.
- (USACE, 2017) UNITED STATES ARMY CORPS OF ENGINEERS, ENGINEERING AND DESIGN, "GROUTING TECHNOLOGY", EM 1110-2-3506
- (ACI, 2001) AMERICAN CONCRETE INSTITUTE, "CONTROL OF CRACKING IN CONCRETE STRUCTURES" (ACI 224), 2001.
- (USACE, 2004) UNITED STATES ARMY CORPS OF ENGINEERS, ENGINEERING AND DESIGN, "GENERAL DESIGN AND CONSTRUCTION CONSIDERATIONS FOR EARTH AND ROCK-FILL DAMS", EM 1110-2-2300, 2004.
- (ACI, 2006) AMERICAN CONCRETE INSTITUTE, "CODE REQUIREMENTS FOR ENVIRONMENTAL ENGINEERING CONCRETE STRUCTURES" (ACI 350), 2006.
- (ACI, 2019) AMERICAN CONCRETE INSTITUTE, "BUILDING CODE REQUIREMENTS FOR STRUCTURAL CONCRETE" (ACI 318), 2019.
- (USBR, 2012) UNITED STATES DEPARTMENT OF THE INTERIOR, BUREAU OF RECLAMATION, DESIGN STANDARD NO. 13 - EMBANKMENT DAMS, "CHAPTER 2 - EMBANKMENT DESIGN", 1992.
- (USBR, 2012) UNITED STATES DEPARTMENT OF THE INTERIOR, BUREAU OF RECLAMATION, DESIGN STANDARD NO. 13 - EMBANKMENT DAMS, "CHAPTER 9 - STATIC DEFORMATION ANALYSIS", 1992.
- (FERC, 2016) FEDERAL ENERGY REGULATORY COMMISSION, ENGINEERING GUIDELINES FOR EVALUATION OF HYDROPOWER PROJECTS (MOST RECENT VERSIONS)

SCHULTZ, ALEX B:\Working\VILLAGE OF MANCHESTER\2204052 Ford Manchester Dam Structural Analysis\00_CAD\Design\Sheets\S-1.0.dwg - 2/12/2024

NOT FOR CONSTRUCTION

Attention:
0 1"
If this scale bar does not measure 1" then drawing is not original scale.

DRAFT

Designed:	MEC
Drawn:	AFS
Checked:	MG
Approved:	SE
P.E. No:	####
GEI Project	2204052

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49158

FORD MANCHESTER DAM

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912 CITY ROAD
MANCHESTER, MI 48158

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SHEET NAME: **DESIGN NOTES AND SYMBOLOGY**

SHEET NO.: S-1.0

DWG. NO.: 2 OF ?

EXISTING CONDITIONS

SCALE: 1" = 30'

NOT FOR CONSTRUCTION

APPROX 766 SQ FT OF CONCRETE REPAIR

Attention:

0 1"

If this scale bar does not measure 1" then drawing is not original scale.

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912 CITY ROAD
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SHEET NAME

EXISTING CONDITIONS

SHEET NO.

S-2.0

DWG. NO.

3 OF ?

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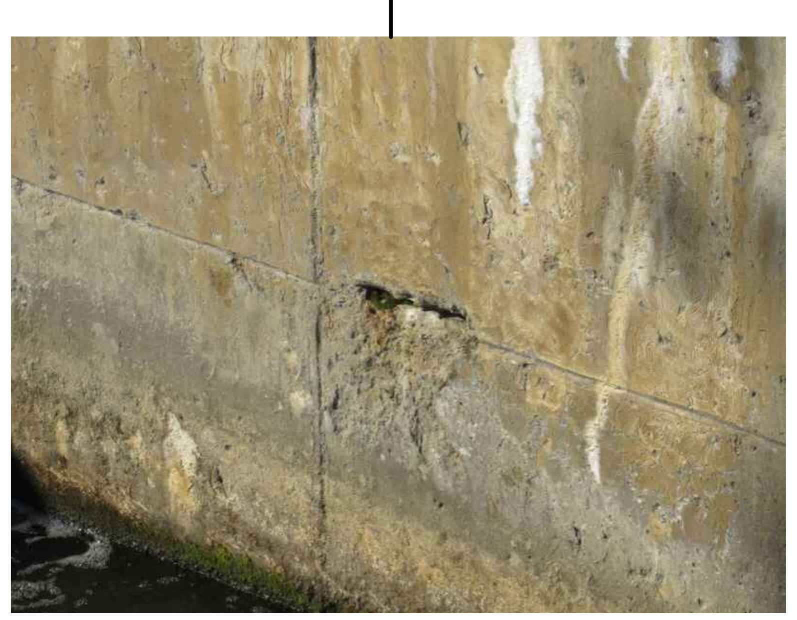
A S-2.0 LEFT WALL ELEVATION SCALE: 1/8" = 1'



NOT FOR CONSTRUCTION

Attention: 0 1" If this scale bar does not measure 1" then drawing is not original scale.

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SHEET NAME: **LEFT WALL REPAIR**

SHEET NO. S-3.0

DWG. NO. 4 OF ?

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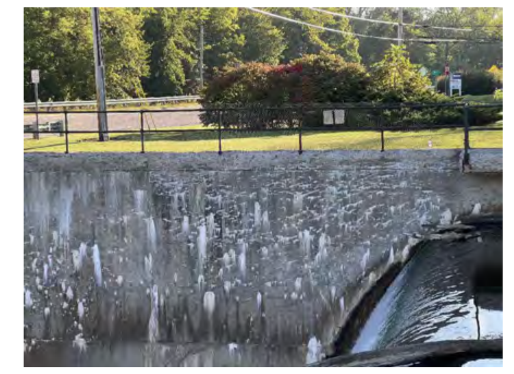

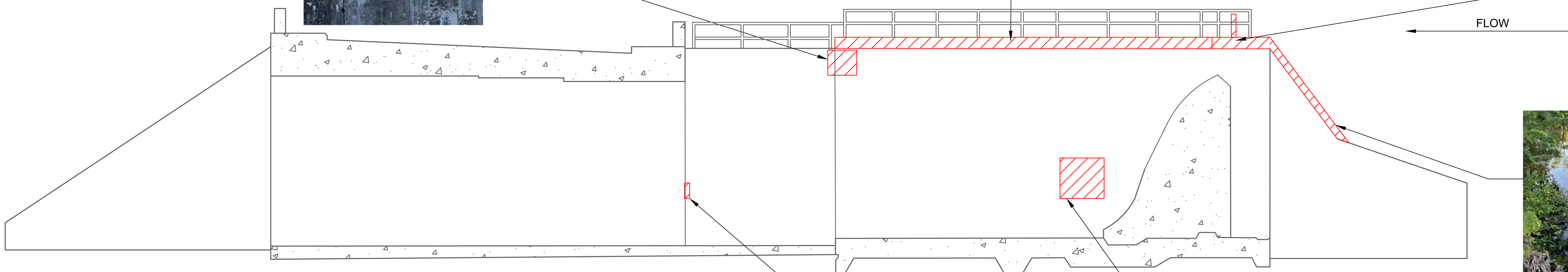

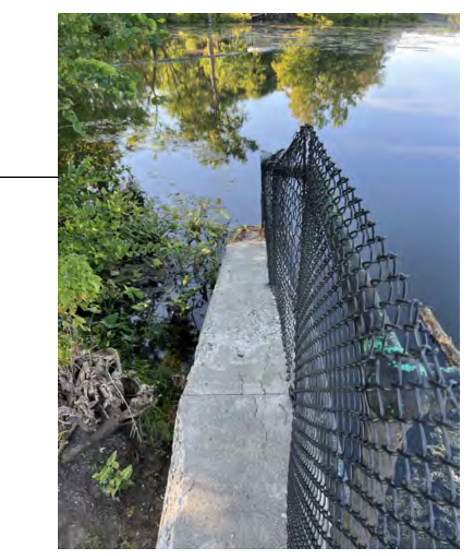

B
S-2.0

RIGHT WALL
ELEVATION

SCALE: 1/8" = 1'

NOT FOR CONSTRUCTION

Attention: 0 1" If this scale bar does not measure 1" then drawing is not original scale.	DRAFT	*Designed:* MEC *Drawn:* AFS *Checked:* MG *Approved:* SE *P.E. No:* #### *GEI Project* 2204052	GEI Consultants GEI CONSULTANTS, INC. 8615 W. BRYN MAWR AVE. SUITE 406 CHICAGO, IL 60631 (312)985-0365	VILLAGE OF MANCHESTER 912 CITY ROAD MANCHESTER, MI 49158	**FORD MANCHESTER DAM** VILLAGE OF MANCHESTER 912 CITY ROAD MANCHESTER, MI 48158	SHEET NAME **RIGHT WALL REPAIR**	SHEET NO. S-3.1 DWG. NO. 5 OF ?

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C
S-2.0
UPSTREAM SPILLWAY ELEVATION
SCALE: 1/4" = 1'

NOT FOR CONSTRUCTION

Attention:
0 1"
If this scale bar does not measure 1" then drawing is not original scale.

DRAFT

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Drawn:	AFS
Checked:	MG
Approved:	SE
P.E. No:	####
GEI Project	2204052

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MANCHESTER, MI 48158

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SHEET NAME
UPSTREAM SPILLWAY REPAIRS

SHEET NO.
S-3.2

DWG. NO.
6 OF ?

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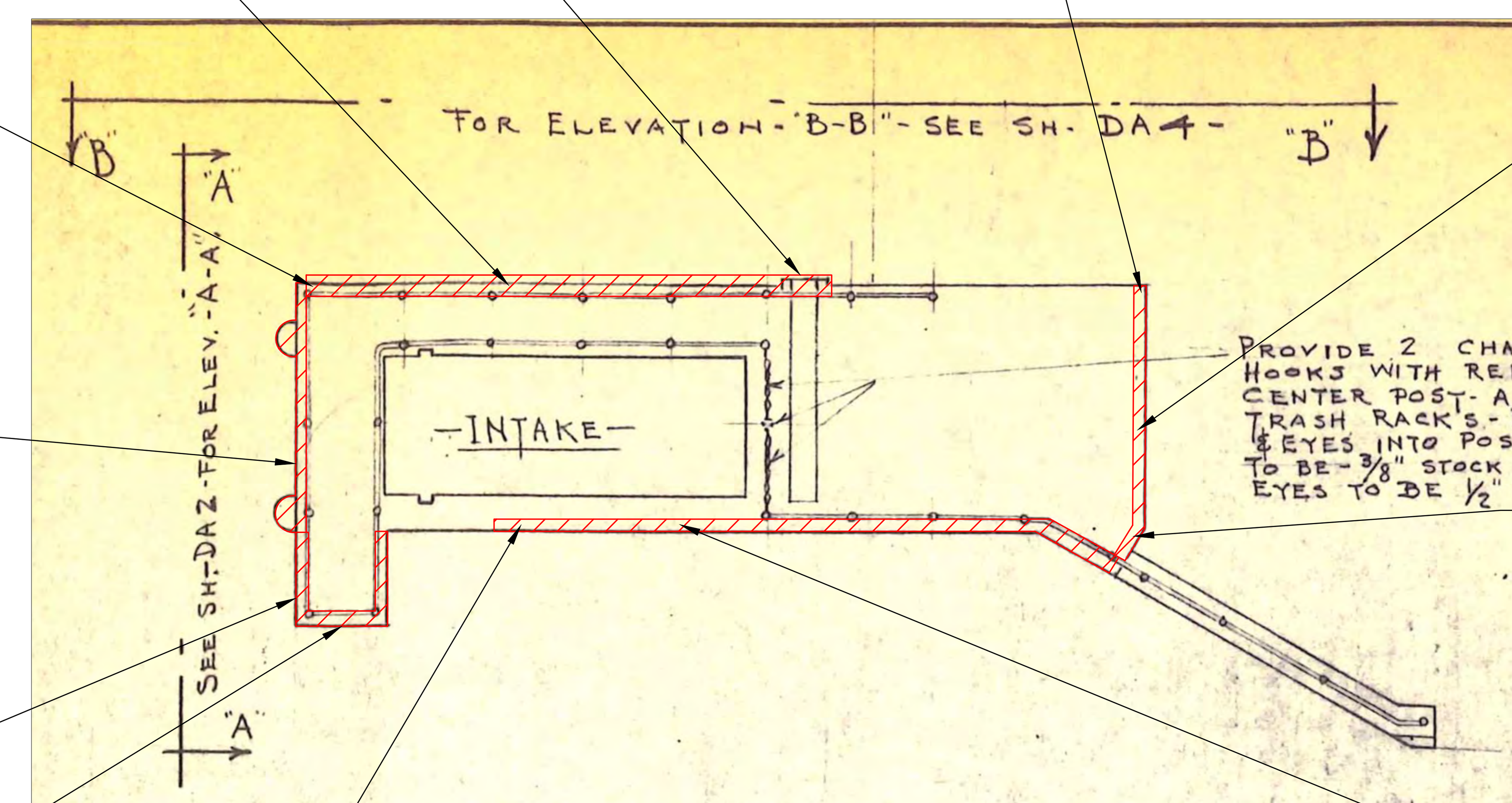
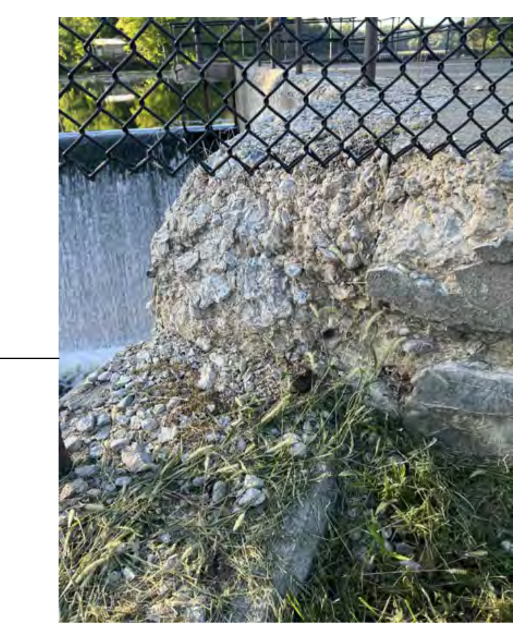
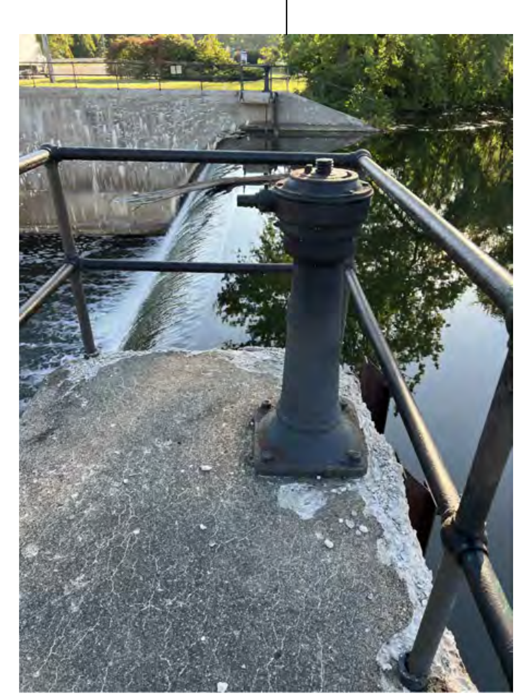


E
S-2.0
PENSTOCK INTAKE
PLAN
SCALE: 1/8" = 1'





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Attention: 0 1" If this scale bar does not measure 1" then drawing is not original scale.	DRAFT	Designed: MEC Drawn: AFS Checked: MG Approved: SE P.E. No: #### GEI Project 2204052	GEI Consultants GEI CONSULTANTS, INC. 8615 W. BRYN MAWR AVE. SUITE 406 CHICAGO, IL 60631 (312)985-0365	VILLAGE OF MANCHESTER 912 CITY ROAD MANCHESTER, MI 49158	**FORD MANCHESTER DAM** VILLAGE OF MANCHESTER 912 CITY ROAD MANCHESTER, MI 48158	SHEET NAME **INTAKE REPAIRS**	SHEET NO. S-3.3 DWG. NO. 8 OF ?

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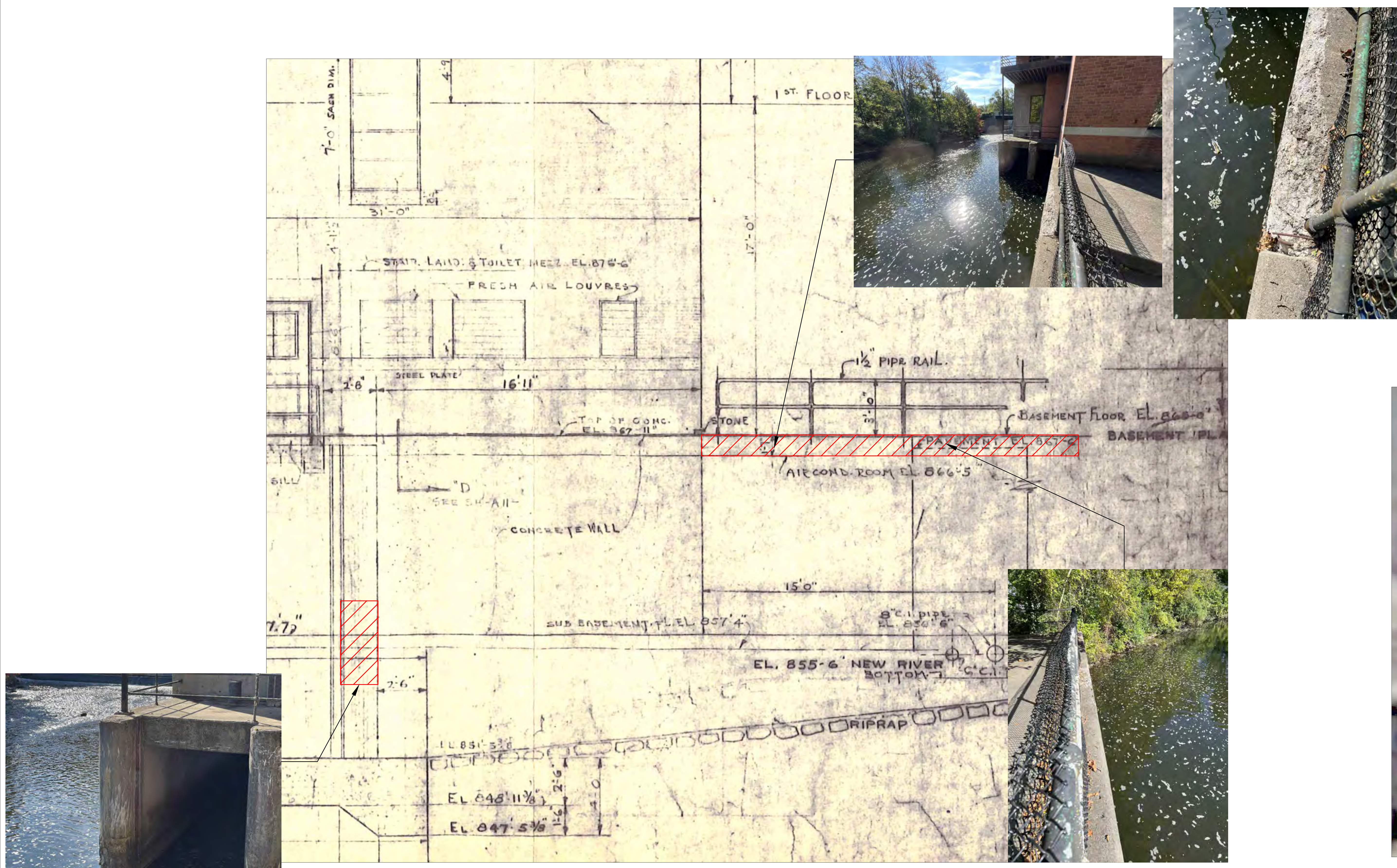
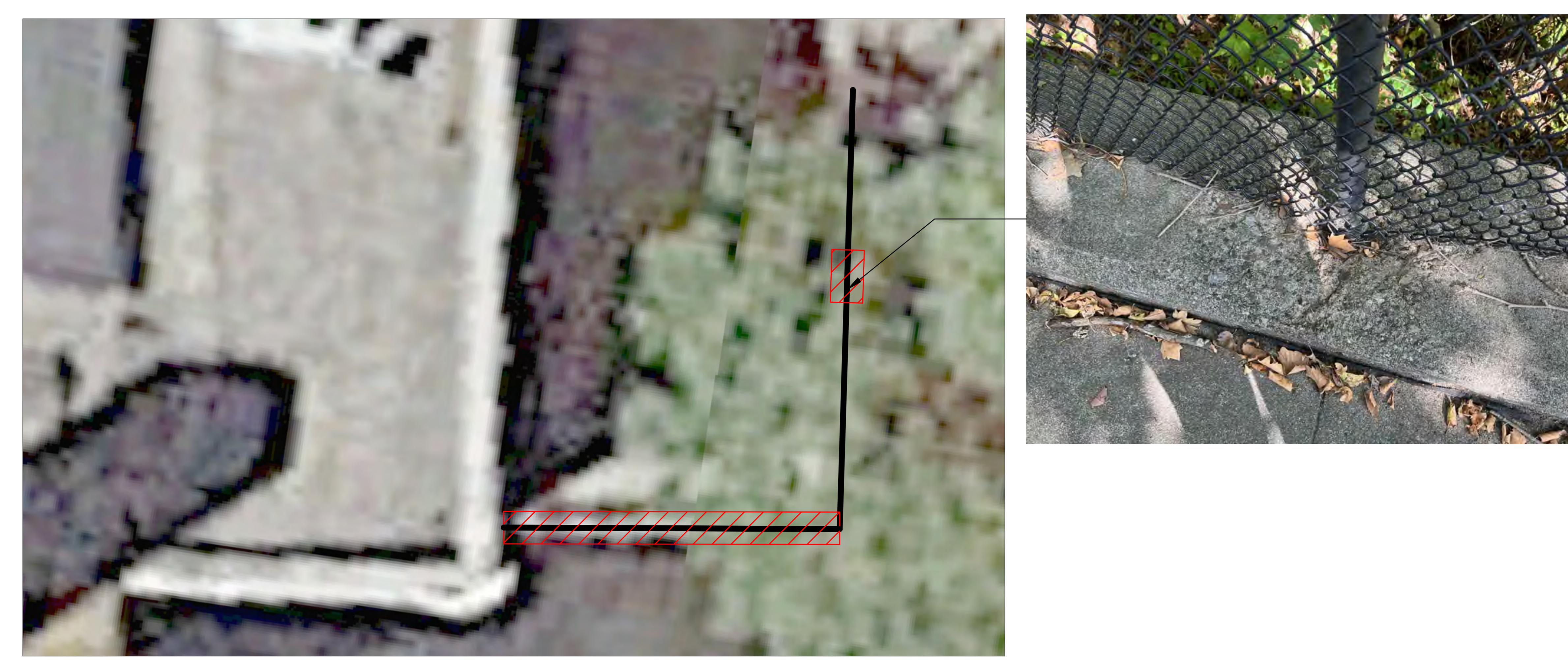


F / S-2.0 SEAT WALL
ELEVATION
SCALE: 1/4" = 1'



H / S-2.0 SEAT WALL
PLAN
SCALE: 3/16" = 1'

NOT FOR CONSTRUCTION

Attention:
0 1"
If this scale bar does not measure 1" then drawing is not original scale.

DRAFT

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Drawn:	AFS
Checked:	MG
Approved:	SE
P.E. No:	####
GEI Project	2204052

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912 CITY ROAD
MANCHESTER, MI 49158

FORD MANCHESTER DAM

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912 CITY ROAD
MANCHESTER, MI 48158

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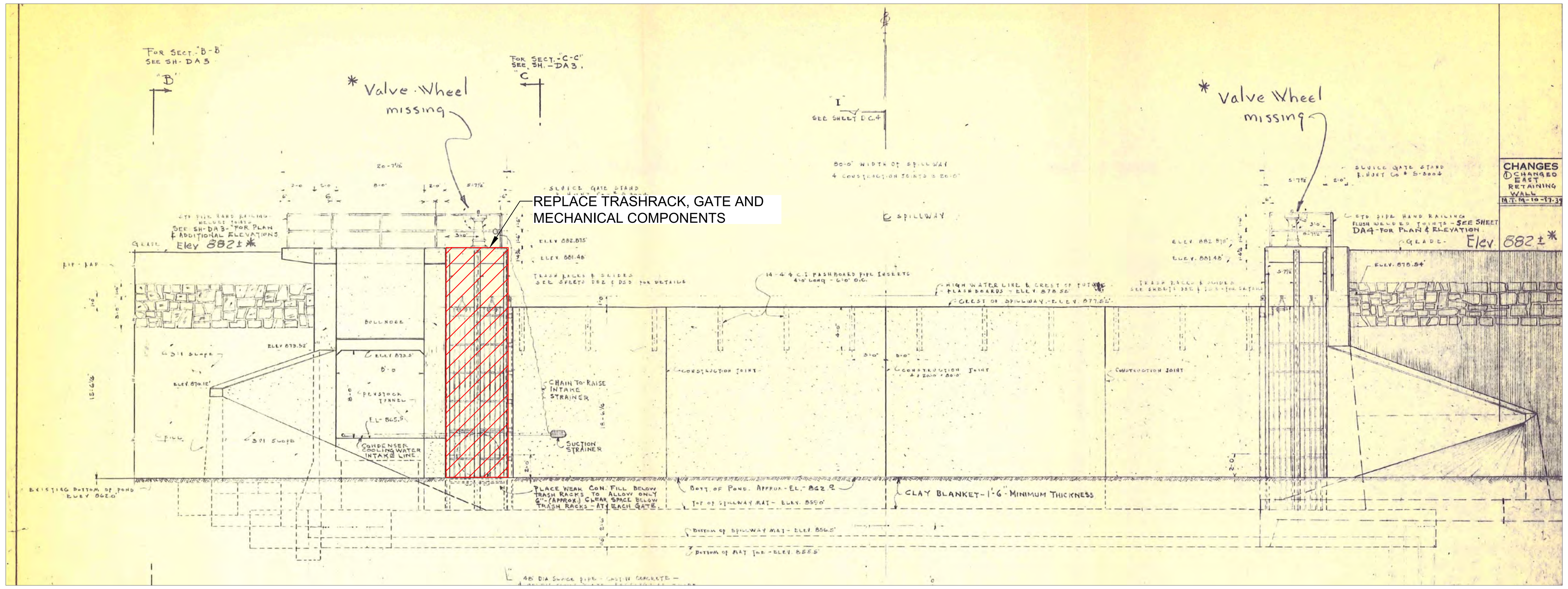
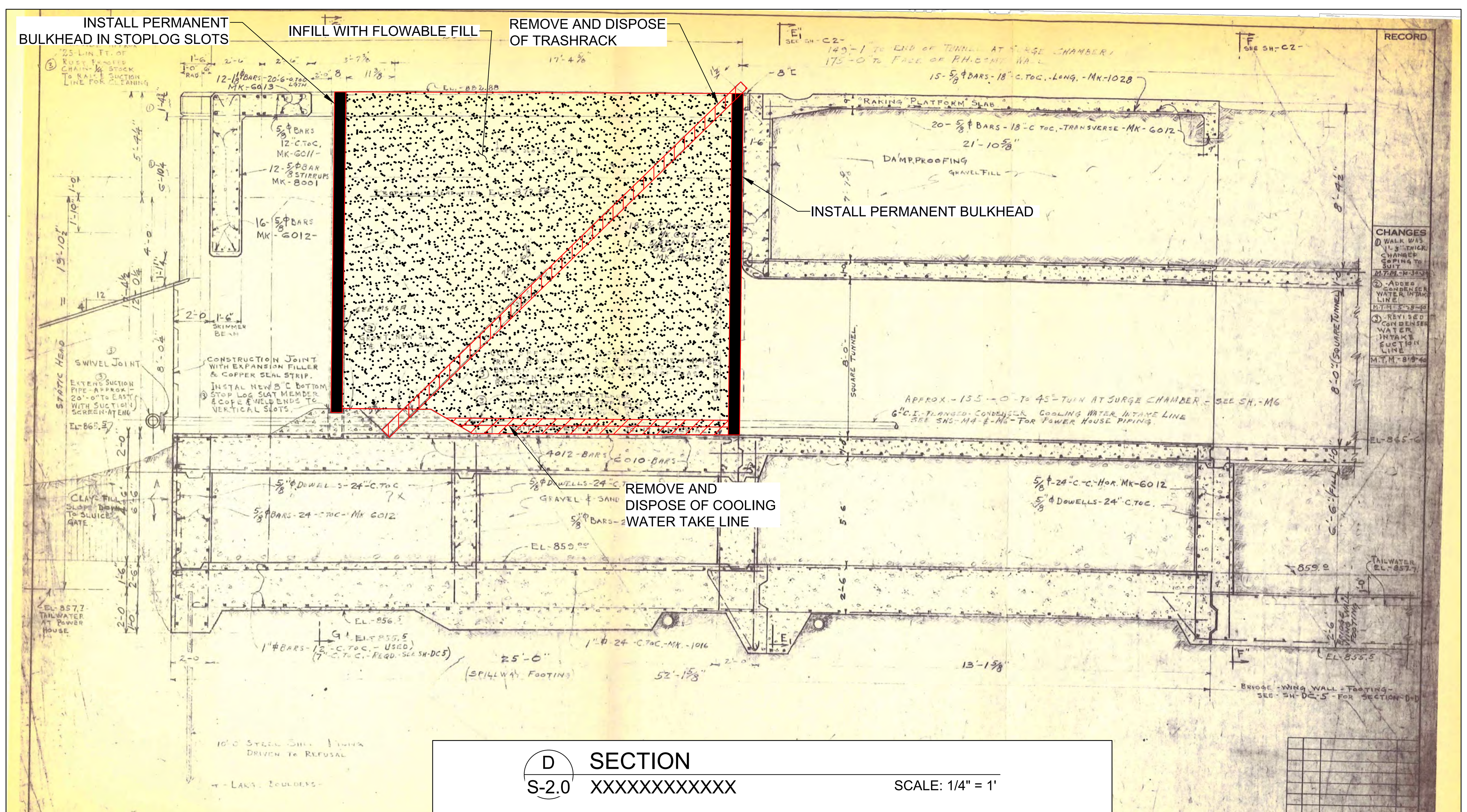
SHEET NAME: **INTAKE REPAIRS**

SHEET NO. S-3.4

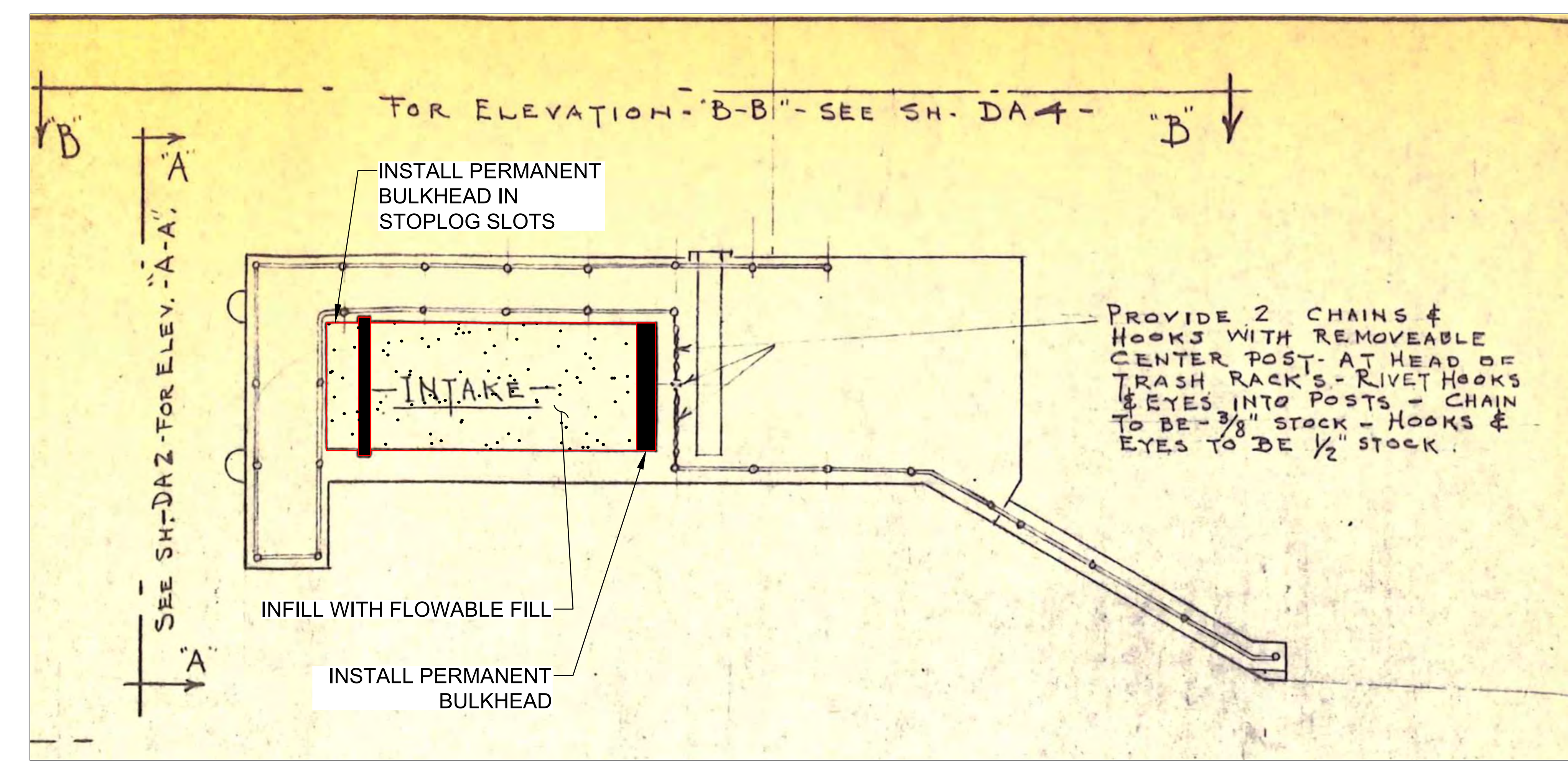
DWG. NO. 9 OF ?



G SECTION
S-2.0 XXXXXXXXXXXX SCALE: 1/8" = 1'



D SECTION
S-2.0 XXXXXXXXXXXX SCALE: 1/4" = 1'



E INTAKE PLAN
S-2.0 XXXXXXXXXXXX SCALE: 1/8" = 1'

NOT FOR CONSTRUCTION

Attention: 0 1" If this scale bar does not measure 1" then drawing is not original scale.	DRAFT	Designed: MEC Drawn: AFS Checked: MG Approved: SE P.E. No: #### GEI Project 2204052	GEI Consultants GEI CONSULTANTS, INC. 8615 W. BRYN MAWR AVE. SUITE 406 CHICAGO, IL 60631 (312)985-0365	VILLAGE OF MANCHESTER 912 CITY ROAD MANCHESTER, MI 49158	**FORD MANCHESTER DAM** VILLAGE OF MANCHESTER 912 CITY ROAD MANCHESTER, MI 48158	SHEET NAME: **DAM MODIFICATIONS** SHEET NO. S-4.0 DWG. NO. 10 OF ?

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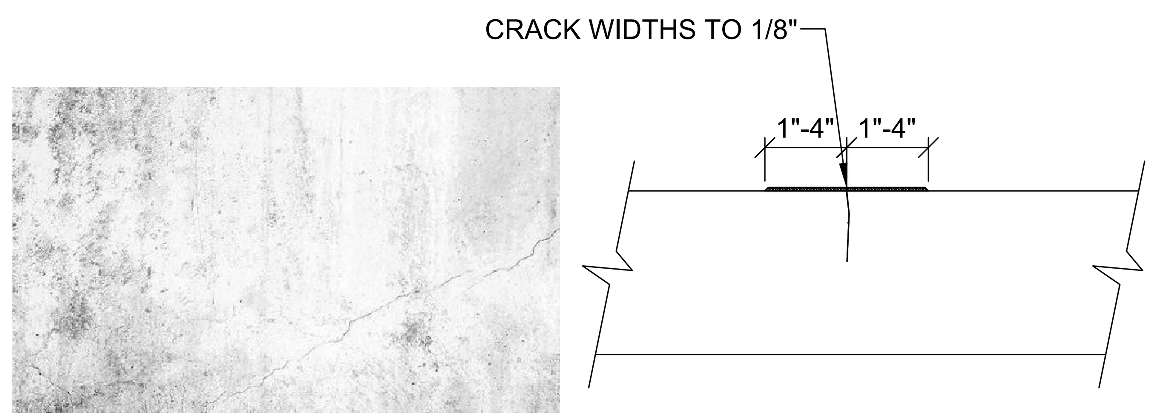
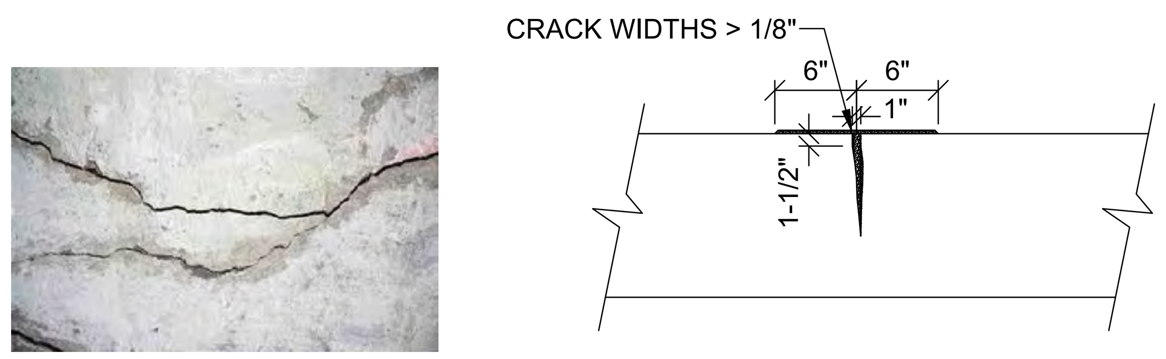


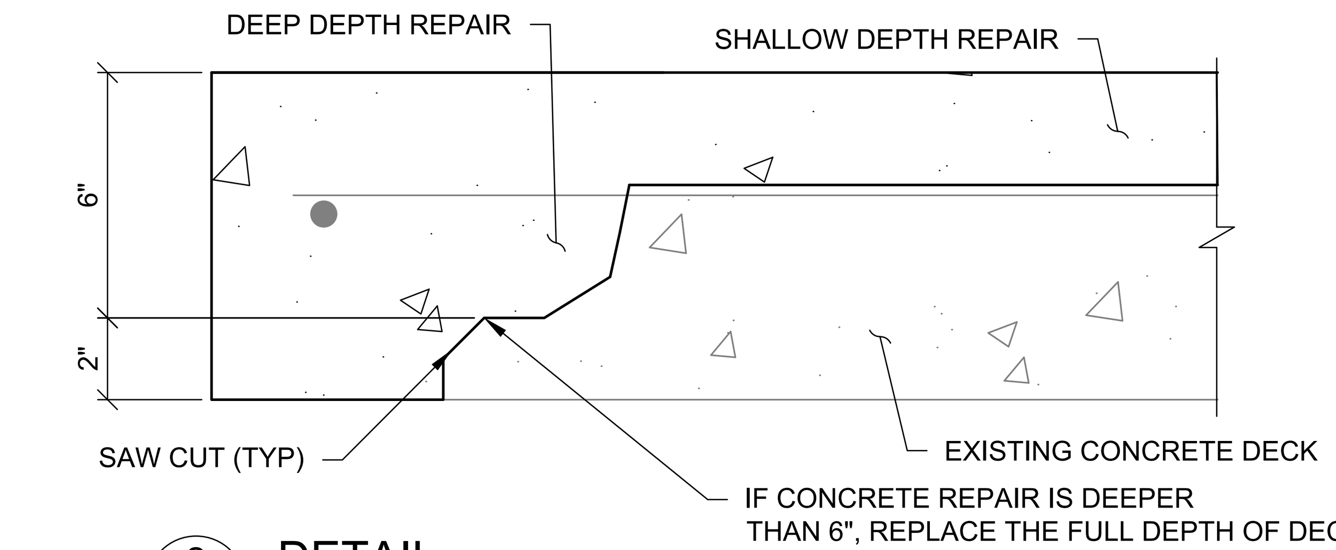
1 - TYPE 1 REPAIR - MINOR CRACKING
NO SCALE



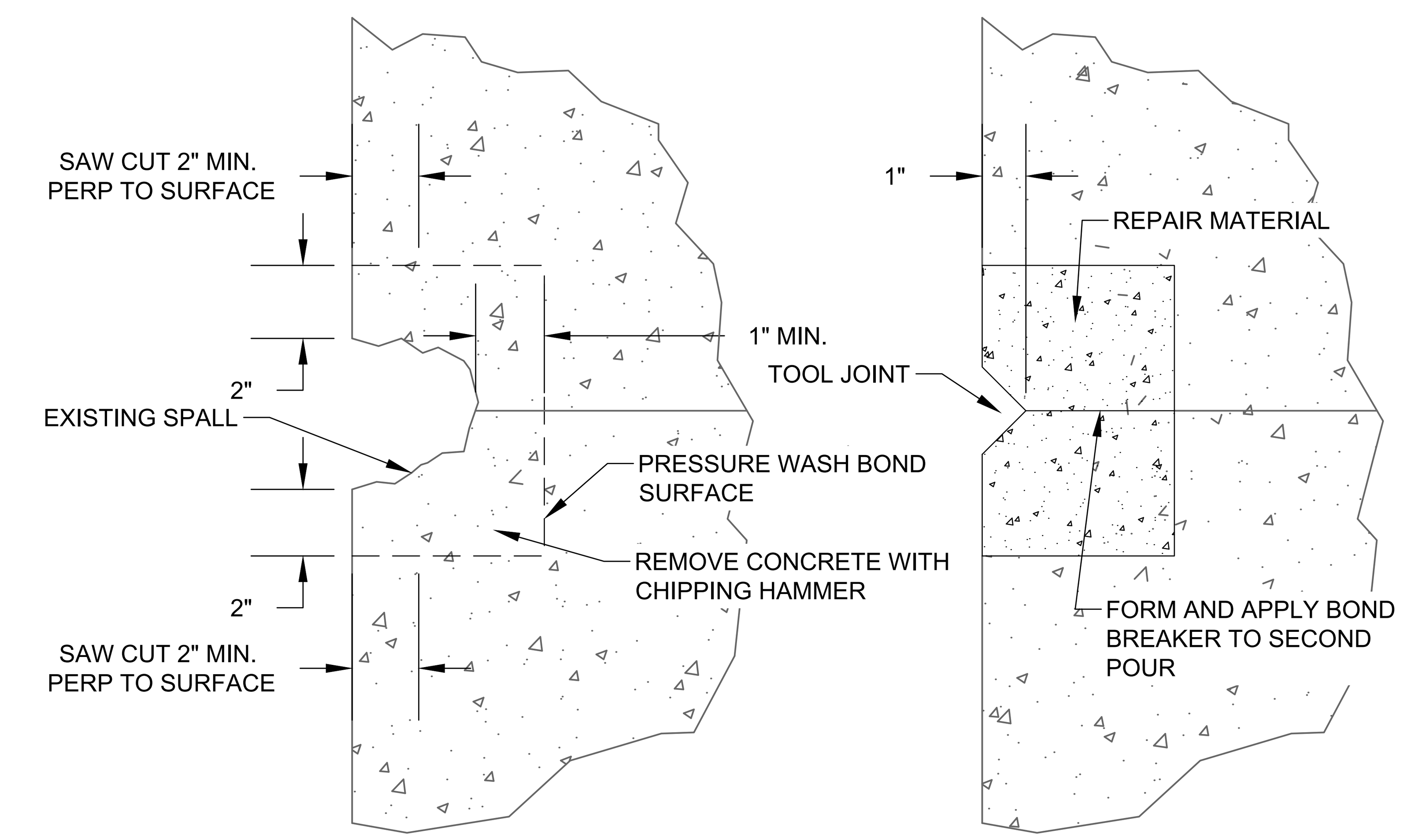
2 - TYPE 2 REPAIR - MAJOR CRACKING
NO SCALE

PREPARATION AND APPLICATION NOTES:

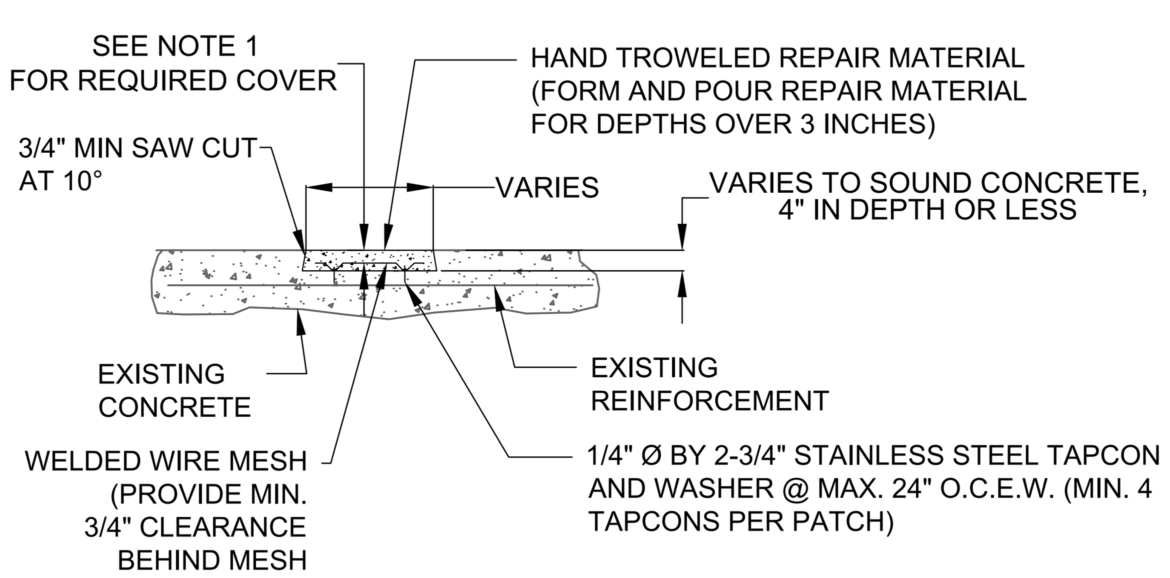
1. CLEAN ALL JOINTS/AREAS TO RECEIVE XYPEX THOROUGHLY. REMOVE ALL LOOSE OR DELAMINATED CONCRETE, LAITANCE, DIRT, OIL, PAINT OR OTHER FOREIGN SUBSTANCES THAT CAN INHIBIT BOND.
2. TYPE 1 REPAIR: APPLY SIKAFLEX-1a TO JOINT SURFACES IN ACCORDANCE WITH MANUFACTURER INSTRUCTIONS. EXTEND SIKAFLEX 1 TO 4-INCHES ON EITHER SIDE OF THE CRACK.
3. TYPE 2 REPAIR: ROUTE CRACK BY CHIPPING OR SAWCUTTING TO A DEPTH OF 1.5-IN AND A WIDTH OF 1-IN. SLOT SHALL NOT BE A "V" SHAPED SLOT. PLACE XYPEX PATCH AND PLUG IN CRACK CAVITY PRESSING FIRMLY UNTIL PATCH AND PLUG MATERIAL IS HARDENED.APPLY XYPEX CONCENTRATE SLURRY TO JOINT SURFACES AT THE RATE OF 2.0 LB./SQ. YD. (1.0 KG/M²). EXTEND SLURRY COAT 6-INCHES ON EITHER SIDE OF THE CRACK.
4. TYPE 3 REPAIR: SAWCUT SLAB AT 10° ANGLE TO THE GREATEST DEPTH OF THE SPALL. CLEAN PER NOTE 1. PLACE BONDING AGENT AND REPAIR MORTAR AND PLUG PER MANUFACTURER'S REQUIREMENTS AND TROWEL FINISH FLUSH WITH FACE OF CONCRETE. 1
5. ALL REPAIRS SHALL MATCH THE SURFACE OF EXISTING CONCRETE. ALL EXISTING JOINTS SHALL BE MAINTAINED. IF A REPAIR EXTENDS PAST AN EXISTING JOINT, A 1/2" DEEP SAWCUT SHALL BE PROVIDED.



? - DETAIL
CONCRETE DECK JOINT
SCALE: 3" = 1'-0"



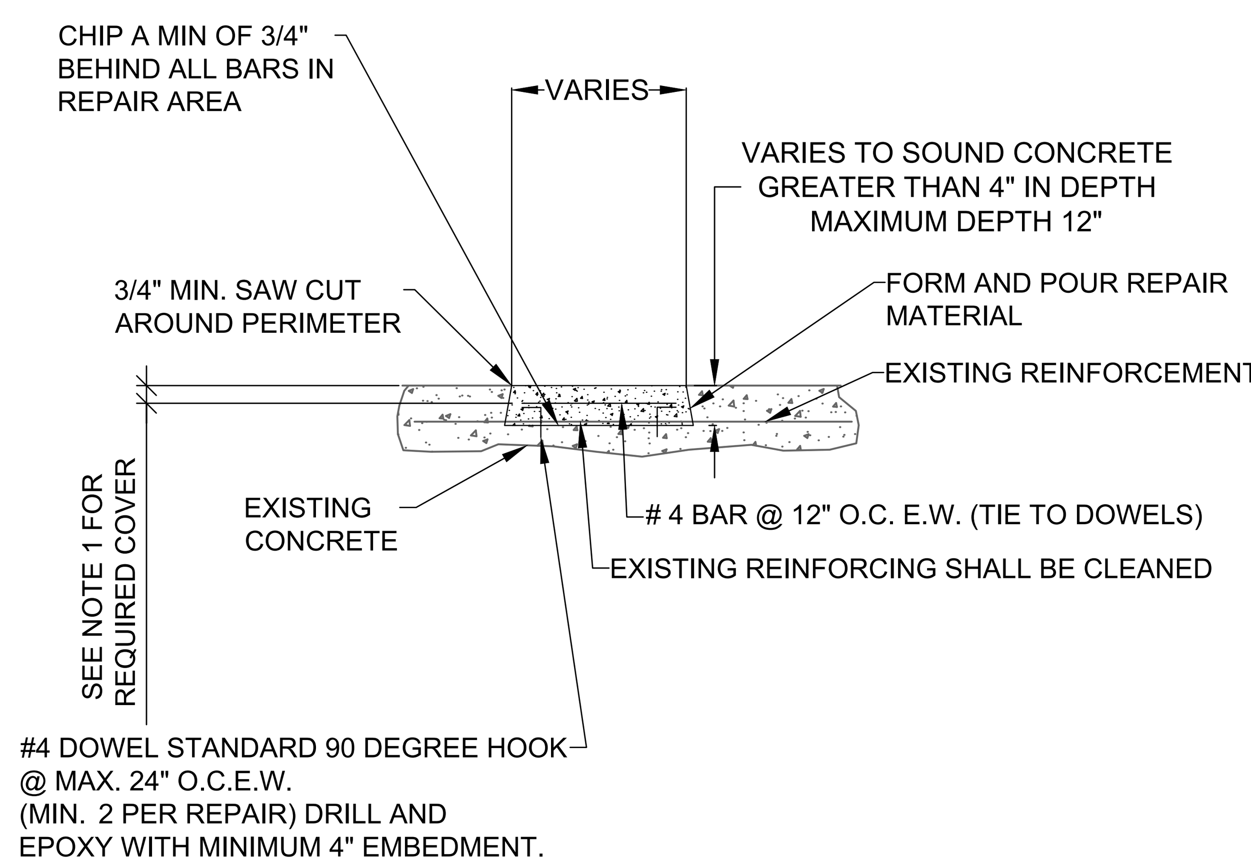
? - DETAIL
JOINT REPAIR - 2 SIDES
NOT TO SCALE



NOTES:

1.) PROVIDE MINIMUM REINFORCEMENT COVER OF 2".
2.) IF ANY REINFORCEMENT IS EXPOSED, PLEASE INITIATE DEEP DEPTH REPAIR PER DETAIL 2.
3.) EXISTING SUBSTRATE MATERIALS SHALL BE COATED WITH A BONDING AGENT BY SIKA OR APPROVED EQUAL.

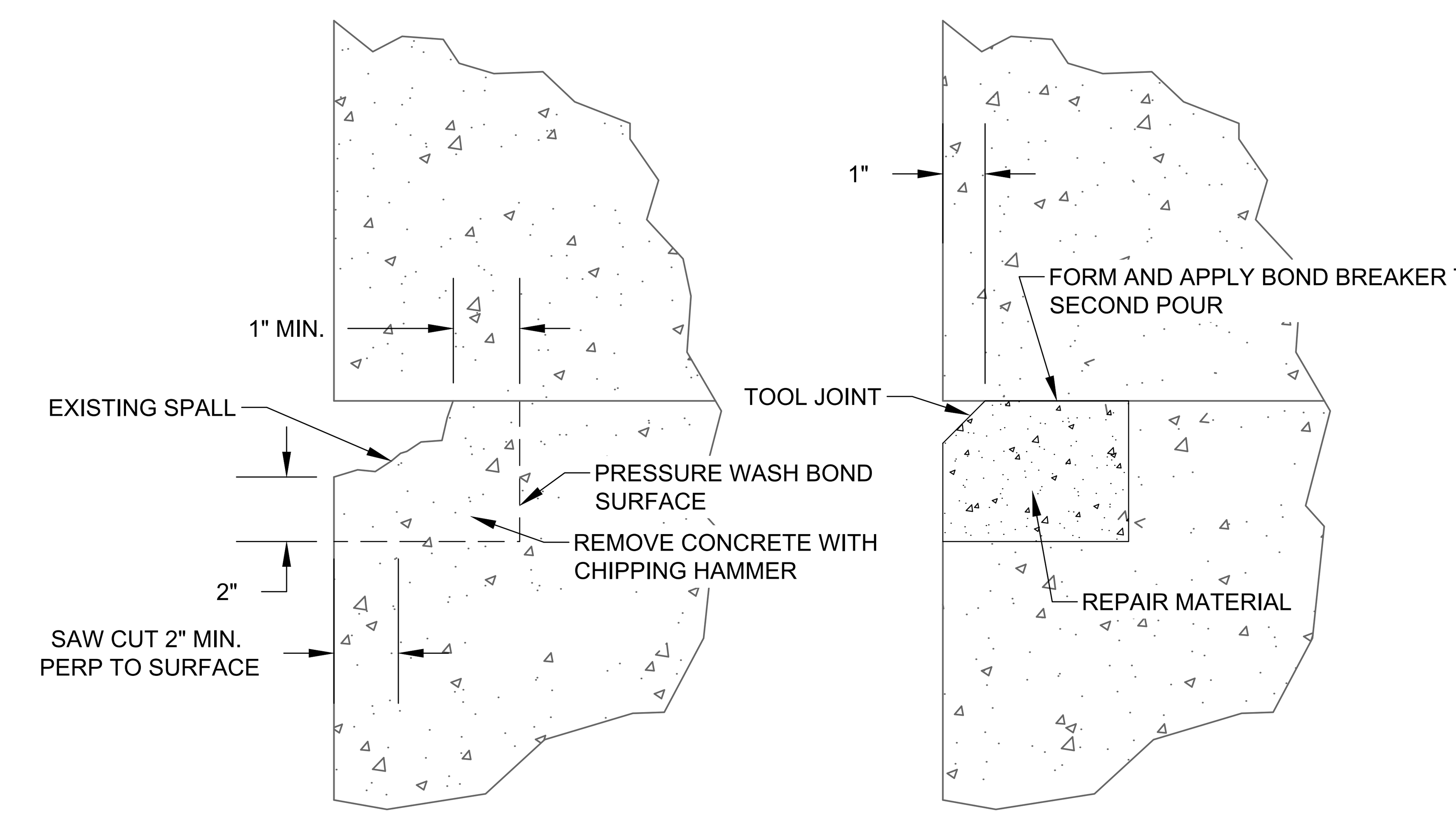
3.1 - TYPE 3 REPAIR - SPALL REPAIR
SHALLOW DEPTH REPAIR (4" OR LESS)
NO SCALE



NOTES:

1.) PROVIDE MINIMUM REINFORCEMENT COVER OF 3".
2.) BOTH SHALLOW AND DEEP REPAIR MAY BE POURED AS A CONTINUOUS POUR.
3.) EXISTING SUBSTRATE MATEIAL SHALL BE COATED WITH A BONDING AGENT BY SIKA OR APPROVED EQUAL.

3.2 - TYPE 3 REPAIR - SPALL REPAIR
DEEP DEPTH REPAIR (GREATER THAN 4")
NO SCALE



? - DETAIL
JOINT REPAIR - SINGLE SIDE
NOT TO SCALE

NOT FOR CONSTRUCTION

Attention: 0 — 1" If this scale bar does not measure 1" then drawing is not original scale.	DRAFT	Designed: MEC Drawn: AFS Checked: MG Approved: SE P.E. No: #### GEI Project 2204052	GEI Consultants GEI CONSULTANTS, INC. 8615 W. BRYN MAWR AVE. SUITE 406 CHICAGO, IL 60631 (312)985-0365	VILLAGE OF MANCHESTER 912 CITY ROAD MANCHESTER, MI 49158

FORD MANCHESTER DAM

VILLAGE OF MANCHESTER
912 CITY ROAD
MANCHESTER, MI 48158

NO	DATE	ISSUE/REVISION	APP
0	2/12/2024	FOR REVIEW	MG

SHEET NAME: **REPAIR DETAILS**

SHEET NO. S-5.0

DWG. NO. 11 OF ?

SCHULTZ, ALEX B:\Working\VILLAGE OF MANCHESTER\2204052 Ford Manchester Dam Structural Analysis\00_CAD\Design\Sheets\S-5.0.dwg - 2/12/2024

1.0 GENERAL NOTES

1. THESE NOTES SUPPLEMENT THE SPECIFICATIONS. ANY DISCREPANCY FOUND AMONG THE DRAWINGS, SPECIFICATIONS, THESE NOTES, AND THE SITE CONDITIONS SHALL BE REPORTED TO THE OWNER'S REPRESENTATIVE (GEI), WHO SHALL CORRECT SUCH DISCREPANCY IN WRITING. ANY WORK DONE BY THE CONTRACTOR AFTER DISCOVERY OF SUCH DISCREPANCY SHALL BE DONE AT CONTRACTOR'S RISK. THE CONTRACTOR SHALL VERIFY AND COORDINATE THE DIMENSIONS AMONG ALL DRAWINGS PRIOR TO PROCEEDING WITH ANY WORK OF FABRICATION.

2. CONSTRUCTION IS SUBJECT TO THE RULES AND REGULATIONS OF THE FEDERAL ENERGY REGULATORY COMMISSION (FERC). SUBSTANTIVE CHANGES TO THE APPROVED DRAWINGS AND SPECIFICATIONS MAY REQUIRE FERC APPROVAL.

3. PRIMARY SITE ACCESS TO THE PROJECT IS PROVIDED FROM JEFFERSON BOULEVARD TO TWIN LAKES DRIVE FROM THE NORTH AND FROM LINCOLN WAY TO POWER DRIVE FROM THE SOUTH.

4. THE SUBSURFACE UTILITY INFORMATION IN THIS PLAN IS UTILITY LEVEL C. THIS QUALITY LEVEL WAS DETERMINED ACCORDING TO THE GUIDELINES OF CI/ASCE 38-2, ENTITLED "STANDARD GUIDELINES FOR THE COLLECTION AND DEPICTION OF EXISTING SUBSURFACE UTILITY DATA".

5. THE CONTRACTOR IS RESPONSIBLE FOR ALL TEMPORARY BRACING AND SHORING (SHOWN AND NOT SHOWN ON THESE DRAWINGS) DURING CONSTRUCTION.

6. SHOP DRAWINGS FOR REINFORCING STEEL AND STRUCTURAL STEEL SHALL BE SUBMITTED TO THE OWNER'S REPRESENTATIVE (GEI) FOR REVIEW PRIOR TO FABRICATION. OWNER'S REPRESENTATIVE (GEI) SHOP DRAWING REVIEW IS FOR GENERAL CONFORMANCE ONLY OF THE DESIGN CONCEPT AND CONTRACT DOCUMENTS. OWNER'S REPRESENTATIVE (GEI) COMMENTS SHALL NOT BE CONSTRUED AS RELIEVING THE CONTRACTOR FROM COMPLIANCE WITH THE PROJECT PLANS AND SPECIFICATIONS. THE CONTRACTOR REMAINS RESPONSIBLE FOR DETAILS AND ACCURACY FOR CONFORMING AND CORRELATING ALL QUANTITIES AND DIMENSIONS AND FOR PERFORMING THE WORK IN SAFE MANNER.

7. CONTRACTOR IS RESPONSIBLE FOR DEVELOPING AND IMPLEMENTING A SEQUENCE AND SCHEDULE FOR COMPLETION OF THE WORK IN ACCORDANCE WITH APPLICABLE PROVISIONS OF THE CONTRACT DOCUMENTS. A GENERAL CONSTRUCTION SEQUENCE IS PROVIDED BELOW. HOWEVER, THE CONTRACTOR WILL NEED TO SUBMIT THEIR WORK PLAN WHICH SHALL INCLUDE THEIR PROPOSED SEQUENCE OF WORK.

8. RESERVOIR MUST REMAIN IN SERVICE, FULLY OPERATIONAL, AND ACCESSIBLE BY OWNER WITHOUT INTERRUPTION THROUGHOUT THE PROJECT.

1.1 COORDINATION

1. SCHEDULE, COORDINATE, AND PERFORM THE WORK TO ALLOW NORMAL FACILITY OPERATIONS, INCLUDING OWNER ACTIVITIES RELATED TO OPERATION, MAINTENANCE, MONITORING, AND INSPECTION OF THE PROJECT.

2. COORDINATE WITH THE OWNER FOR DESIGNATION OF SPECIFIC WORK AREAS FOR RESTRICTED ACCESS DURING PARTICULAR PERIODS OF THE WORK, AS WELL AS NEED FOR FENCING, SIGNAGE, AND OTHER MEASURES NECESSARY TO PROTECT THE PUBLIC, COMPLETED WORK, WORK IN PROGRESS, AND CONTRACTOR'S EQUIPMENT AND PROPERTY ON SITE.

3. MAINTAIN FULL AND COMPLETE ACCESS TO WORK TO OWNER, ENGINEER, AND REGULATORY PERSONNEL.

1.2 PERMITTING AND ENVIRONMENTAL

1. COMPLY WITH ALL REQUIREMENTS OF OWNER-OBTAINED PROJECT PERMITS AND THE SPECIAL CONDITIONS FOR REQUIREMENTS OF OWNER-OBTAINED PERMITS. CONTRACTOR SHALL OBTAIN ALL OTHER PERMITS NECESSARY FOR CONSTRUCTION OF THE WORK.

2.0 MATERIALS

2.1 REINFORCEMENT

A. REINFORCING STEEL: ASTM A615, 60 KSI YIELD GRADE BILLET STEEL DEFORMED BARS, UNCOATED FINISH.

B. DOWELS: ASTM A615, 60 KSI YIELD GRADE BILLET STEEL DEFORMED BARS, UNCOATED FINISH.

C. WELDED WIRE MESH: 12 OR 14 GA., FLAT SHEETS, 4" X 4" SQUARE OPENING, ANSI/ASTM A185, 304 STAINLESS STEEL.

D. STAINLESS STEEL TAPCON: GRADE 410 STAINLESS STEEL 1/4" DIAMETER BY 2-3/4" LENGTH HEX HEAD SCREW WITH STAINLESS STEEL WASHER.

2.2 CURING COMPOUND

A. ASTM C 309, TYPE 1, LIQUID MEMBRANE

B. BASF MASTERKURE CC 1315WB OR APPROVED EQUIVALENT

2.3 REPAIR MORTARS

A. PRODUCT PERFORMANCE SHALL MEET OR EXCEED THAT SPECIFIED FOR R1 MATERIALS PER ASTM C 928.

a. REPAIR MORTARS (LESS THAN 4 INCHES THICK): MASTEREMACO S 488 CI, S 466 CT, BASF OR EQUIVALENT.

b. REPAIR MORTARS (GREATER THAN 4 INCHES THICK): MASTEREMACO S 466 CT, BASF OR EQUIVALENT.

3.0 EXECUTION

3.1 WELDED WIRE MESH INSTALLATION

A. MESH SHALL BE FREE FROM LOOSE RUST AND SCALE, DIRT, OIL OR OTHER DELETERIOUS COATING THAT COULD REDUCE BOND WITH THE CONCRETE.

B. CONCRETE COVER SHALL BE AS REQUIRED BY ACI 301 AND AS FOLLOWS:

a. SURFACES EXPOSED TO WEATHER - 2 INCHES

C. MECHANICALLY ANCHOR THE MESH TO THE EXISTING CONCRETE WITH STAINLESS STEEL TAPCON SCREWS AND WASHERS AS INDICATED ON THE PROJECT DRAWINGS.

D. ADJACENT MESH SHEETS SHALL BE OVERLAPPED ONE MESH WIDTH AND SHALL BE TIED FIRMLY TOGETHER WITH 304 STAINLESS STEEL WIRE AT INTERVALS NOT EXCEEDING 8 INCHES.

E. WIRE MESH IS NOT NECESSARY IN AREAS WHERE EXISTING REINFORCEMENT WILL PROVIDE ADEQUATE RESTRAINT.

3.2 REPAIR MORTAR/READY MIX CONCRETE APPLICATION

A. PATCHING MORTAR REPAIRS LESS THAN 3 INCHES THICK - NO FORMS. UNLESS OTHERWISE RECOMMENDED BY MORTAR MANUFACTURER, APPLY AS FOLLOWS:

a. THE SUBSTRATE SHALL BE KEPT WET FOR THE FIRST 12 HOURS DURING THE 24-HOUR PERIOD PRIOR TO PLACING MORTAR TO ASSURE SATURATED SURFACE DRY CONDITIONS.

b. EXISTING SUBSTRATE MATERIAL SHALL BE COATED WITH A BONDING AGENT BY SIKA OR APPROVED EQUAL. THE SURFACE SHALL BE VACUUMED COMPLETELY CLEAN AS THE LAST OPERATION PRIOR TO PLACING MORTAR.

c. WHERE POSSIBLE SCRUB A SLURRY OF NEAT PATCHING MORTAR INTO SUBSTRATE, FILLING PORES AND VOIDS.

d. PLACE PATCHING MORTAR BY TROWELING TOWARD EDGES OF PATCH TO FORCE INTEGRAL CONTACT WITH EDGE SURFACES. FOR LARGE PATCHES, FILL EDGES FIRST AND THEN WORK TOWARD CENTER, ALWAYS TROWELING TOWARD EDGES OF PATCH. AT FULLY EXPOSED REINFORCING BARS, FORCE PATCHING MORTAR TO FILL SPACE BEHIND BARS BY COMPACTING WITH TROWEL FROM SIDES OF BARS.

e. FOR VERTICAL PATCHING, PLACE MORTAR IN LIFTS OF NOT MORE THAN 3 INCHES NOR LESS THAN 1/4 INCH. DO NOT FEATHER EDGE.

f. FOR OVERHEAD PATCHING, PLACE MORTAR IN LIFTS OF NOT MORE THAN 1 INCH NOR LESS THAN 1/4 INCH. DO NOT FEATHER EDGE.

g. AFTER EACH LIFT IS PLACED, CONSOLIDATE MATERIAL AND SCREED SURFACE.

h. WHERE MULTIPLE LIFTS ARE USED, SCORE SURFACE OF LIFTS TO PROVIDE A ROUGH SURFACE FOR APPLICATION OF SUBSEQUENT LIFTS. ALLOW EACH LIFT TO REACH FINAL SET BEFORE PLACING SUBSEQUENT LIFTS. SURFACE SHALL BE KEPT CONTINUALLY MOIST BETWEEN APPLICATIONS.

i. ALLOW SURFACES OF LIFTS THAT ARE TO REMAIN EXPOSED TO BECOME FIRM AND THEN APPLY BROOM FINISH.

j. WET-CURE MORTAR FOR NOT LESS THAN SEVEN DAYS BY WATER-FOG SPRAY OR WATER-SATURATED ABSORPTIVE COVER OR CURING COMPOUND APPLIED IN ACCORDANCE WITH MANUFACTURER'S RECOMMENDATIONS. IF DAILY HIGH TEMPERATURES ARE ABOVE 90 DEGREES FAHRENHEIT, WET-CURE METHODS SHALL BE USED FOR A MINIMUM OF FOUR DAYS IN ADDITION TO THE USE OF A CURING COMPOUND.

B. CAST-IN-PLACE PATCHING MORTAR REPAIRS GREATER THAN 3 INCHES THICK. UNLESS OTHERWISE RECOMMENDED BY MORTAR MANUFACTURER, PLACE AS FOLLOWS:

a. THE SUBSTRATE SHALL BE KEPT WET FOR THE FIRST 12 HOURS DURING THE 24-HOUR PERIOD PRIOR TO PLACING MORTAR TO ASSURE SATURATED SURFACE DRY CONDITIONS.

b. EXISTING SUBSTRATE MATERIAL SHALL BE COATED WITH A BONDING AGENT BY SIKA OR APPROVED EQUAL.THE SURFACE SHALL BE VACUUMED COMPLETELY CLEAN AS THE LAST OPERATION PRIOR TO PLACING MORTAR.

c. USE VIBRATORS TO CONSOLIDATE MORTAR AS IT IS PLACED.

d. AT UNFORMED SURFACES, SCREED MORTAR TO PRODUCE A SURFACE THAT WILL MATCH REQUIRED PROFILE AND SURROUNDING CONCRETE.

e. WHEN PLACING MORTAR BY FORM AND POUR METHOD.

i. DESIGN AND CONSTRUCT FORMS TO RESIST WEIGHT OF WET MORTAR. SEAL JOINTS AND SEAMS IN FORMS AND JUNCTIONS OF FORMS WITH EXISTING CONCRETE.

ii. POUR MORTAR INTO PLACE, RELEASING AIR FROM FORMS AS MORTAR IS INTRODUCED. VIBRATE TO CONSOLIDATE AND REMOVE AIR POCKETS.

f. WET-CURE MORTAR FOR NOT LESS THAN SEVEN DAYS BY LEAVING FORMS IN PLACE OR KEEPING SURFACES CONTINUOUSLY WET BY WATER-FOG SPRAY OR WATER-SATURATED ABSORPTIVE COVER OR CURING COMPOUND APPLIED IN ACCORDANCE WITH MANUFACTURER'S RECOMMENDATIONS. IF DAILY HIGH TEMPERATURES ARE ABOVE 90 DEGREES FAHRENHEIT, WET-CURE METHODS SHALL BE USED FOR A MINIMUM OF FOUR DAYS IN ADDITION TO THE USE OF A CURING COMPOUND.

C. COLD-WEATHER REQUIREMENTS: PROCEDURES SHALL CONFORM TO ACI 306. SPECIAL PROTECTIVE MEASURES, APPROVED BY THE ENGINEER, SHALL BE USED WHEN THE AMBIENT AIR TEMPERATURE IS BELOW 35F OR IF THE AMBIENT AIR TEMPERATURE IS BELOW 40F AND FALLING. SUITABLE COVERING AND OTHER MEANS, AS APPROVED, SHALL BE PROVIDED FOR MAINTAINING THE CONCRETE AT A TEMPERATURE OF AT LEAST 50F FOR NOT LESS THAN 72 HOURS AFTER PLACING AND AT A TEMPERATURE ABOVE FREEZING FOR THE REMAINDER OF THE CURING PERIOD. SALT, CHEMICALS OR OTHER FOREIGN MATERIALS SHALL NOT BE MIXED WITH THE CONCRETE TO PREVENT FREEZING.

D. HOT-WEATHER REQUIREMENTS:

a. WHEN CLIMATIC OR OTHER CONDITIONS ARE SUCH THAT TEMPERATURE OF CONCRETE MAY REASONABLY BE EXPECTED TO EXCEED 85 DEGREES F AT TIME OF DELIVERY AT WORK SITE, DURING PLACEMENT, OR DURING FIRST 24 HOURS AFTER PLACEMENT, PERFORM WORK IN ACCORDANCE WITH ACI 305R – HOT WEATHER CONCRETING.

b. CONTRACTOR SHALL MAINTAIN TEMPERATURE OF STRUCTURAL CONCRETE BELOW SPECIFIED MAXIMUM PLACEMENT TEMPERATURES DURING MIXING, CONVEYING, AND PLACING. COOL INGREDIENTS BEFORE MIXING TO MAINTAIN CONCRETE TEMPERATURE AT TIME OF PLACEMENT. MIXING WATER MAY BE CHILLED, OR CHOPPED ICE MAY BE USED TO CONTROL TEMPERATURE PROVIDED WATER EQUIVALENT OF ICE IS CALCULATED IN TOTAL AMOUNT OF MIXING WATER.

c. CONCRETE SHALL BE PLACED IMMEDIATELY AFTER MIXING. TRUCK MIXING SHALL BE DELAYED UNTIL ONLY TIME ENOUGH REMAINS TO ACCOMPLISH MIXING BEFORE CONCRETE IS PLACED.

d. EXPOSED CONCRETE SURFACES WHICH TEND TO DRY OR SET TOO RAPIDLY SHALL BE CONTINUOUSLY MOISTENED BY MEANS OF FOG SPRAYS OR OTHERWISE PROTECTED FROM DRYING DURING THE TIME BETWEEN PLACEMENT AND FINISHING, AND AFTER FINISHING.

e. FINISHING OF SLABS AND OTHER EXPOSED SURFACES SHALL BE STARTED AS SOON AS CONDITION OF CONCRETE ALLOWS AND SHALL BE COMPLETED WITHOUT DELAY.

f. CONCRETE SURFACES EXPOSED TO AIR SHALL BE COVERED AS SOON AS CONCRETE HAS HARDENED SUFFICIENTLY AND SHALL BE KEPT CONTINUOUSLY WET FOR AT LEAST FIRST 24 HOURS OF CURING PERIOD, AND FOR ENTIRE CURING PERIOD UNLESS CURING COMPOUND IS APPLIED AS SPECIFIED BELOW.

g. FORMED SURFACES SHALL BE KEPT COMPLETELY AND CONTINUOUSLY WET FOR THE DURATION OF CURING PERIOD (PRIOR TO, DURING AND AFTER FORM REMOVAL) OR UNTIL CURING COMPOUND IS APPLIED AS SPECIFIED BELOW.

h. IF MOIST CURING IS DISCONTINUED BEFORE THE END OF THE CURING PERIOD, CURING COMPOUND SHALL BE APPLIED IMMEDIATELY, ACCORDING TO MANUFACTURER'S RECOMMENDATIONS. THIS DOES NOT APPLY TO STRUCTURAL CONCRETE.

NOT FOR CONSTRUCTION

Attention:
0 1"
If this scale bar does not measure 1" then drawing is not original scale.

DRAFT

Designed:	MEC
Drawn:	AFS
Checked:	MG
Approved:	SE
P.E. No:	####
GEI Project	2204052

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GEI CONSULTANTS, INC.
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SUITE 406
CHICAGO, IL 60631
(312)985-0365

VILLAGE OF MANCHESTER
912 CITY ROAD
MANCHESTER, MI 49158

FORD MANCHESTER DAM

VILLAGE OF MANCHESTER
912 CITY ROAD
MANCHESTER, MI 48158

NO	DATE	ISSUE/REVISION	APP
0	2/12/2024	FOR REVIEW	MG

SHEET NAME: **SPECIFICATIONS**

SHEET NO. S-6.0

DWG. NO. 12 OF ?

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Appendix F

Preliminary Engineer’s Opinion of Probable Construction Cost Estimate

OPINION OF PROBABLE COST - CONCEPTUAL REPAIRS

Project: Ford Manchester Dam
Client: Village of Manchester

Project No.: 2204052
Date: 12/1/2023
Estimated by: M. Carden
Checked by: M. Guirguis

Item	Description	Quantity	Units	Unit Price	Total Cost	Notes
1.00	**Intake Abandonment**					
1.01	Abandonment	1	LS	$ 125,000	$ 125,000	Includes moblizabtion, removal of old gate, equipment, disposal, demoblization
1.02	Bulkhead Intake	1	LS	$ 25,000	$ 25,000	
1.03	Infill Intake Strucutre - CLSM	2	LS	$ 25,000	$ 50,000	
				Subtotal	**$ 200,000**	
2.00	**New Gate and Trashrack**					
2.01	Supplemental Dive	1	LS	$ 20,000	$ 20,000	Includes dive to take final measurements and detailed insepction of area around the new gate and trashrack
2.02	Installation	1	LS	$ 190,000	$ 190,000	Includes moblizabtion, removal of old gate and trashrack, installtion of new gate and traskrack, rentals, demoblization
2.03	New Gate	1	LS	$ 100,000	$ 100,000	
2.04	New Trashrack	1	LS	$ 10,000	$ 10,000	
				Subtotal	**$ 320,000**	
3.00	**Concrete Repair**					
3.01	Concrete Repair	1	ea	$ 550,000	$ 550,000	766 square feet, includes mobilizabtion, repairs, equipment, demoblizabtion
				Subtotal	**$ 550,000**	
	Construction Subtotal				**$ 1,070,000**	
4.00	Unknown Scope Items			30%	**$ 321,000**	
5.00	Engineering Design and Permitting			10%	**$ 107,000**	
6.00	Engineering and Construction Observation			10%	**$ 107,000**	
	Total Estimated Cost				**$ 1,605,000**	

Information presented on this sheet represents our opinion of probable costs in 2023 dollars. Unit and lump-sum prices are based on costs for similar projects, engineering judgment, and/or published cost data. Client administrative/engineering costs and regulatory fees not included. Actual bids and total project costs may vary based on contractor's perceived risk, site access, season, market conditions, etc. No warranties concerning the accuracy of costs presented herein are expressed or implied.

Appendix G

Operations and Maintenance

Consulting
Engineers and
Scientists

Operation, Maintenance, and Inspection Manual
Ford Manchester Dam

Manchester, Michigan
EGLE Dam ID No. 391

Submitted to:
Village of Manchester
P.O Box 485
Manchester, Michigan 48158

Submitted by:
GEI Consultants of Michigan, P.C.
230 N. Washington Square
Lansing, Michigan 49503

GEI Project No. 2204052

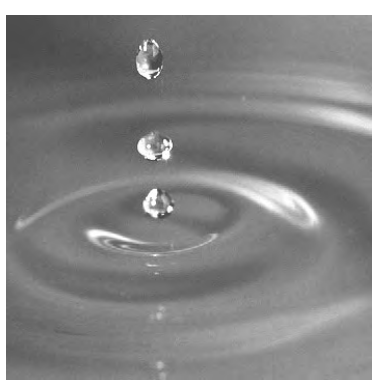

Table of Contents

REVISION SHEET

No.	Description of Revision Made	By	Date
#		by	Date
#		by	Date
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1. General Information

1.1 Operation, Maintenance, and Inspection Manual Introduction

This document is the Operation, Maintenance, and Inspection Manual (Manual) for Ford Manchester Dam. The document provides procedures, guidance, and standard forms for the normal operation, maintenance, monitoring, and inspection of the dam.

If your dam is failing or is experiencing an unusual condition that may lead to failure, immediately activate the Emergency Action Plan (EAP). At a minimum, take the following actions:

- ***Call 911 and let the operator know what roads or buildings downstream of the dam may need to be blocked or evacuated.***
- ***Call the EGLE Dam Safety Emergency Number (800-292-4706)***
- ***Call your Dam Safety Engineer (GEI Consultants, Mark Guirguis (314) 609-3824)***

1.2 Description of Dam and Reservoir

The Ford Manchester Dam impoundment has a surface area of approximately 45 acres at normal lake level, a structural height of approximately 26.5 feet and a hydraulic height of approximately 24.6 feet. During normal conditions, the dam has approximately 20-feet of head with 3.5 to 4-feet of freeboard. Normal headwater is i elevation 877.5 feet and a tailwater elevation of 857.7 feet (USGS Datum). The dam has no auxiliary spillway. The nearest upstream dam is the Manchester Mill Dam, located approximately 1 mile upstream of the Ford Manchester Dam. The nearest downstream dam is the Atles Mill Dam approximately 8.5 miles downstream of the Ford Manchester Dam in Clinton, Michigan.

The Ford Manchester Dam was constructed on the River Raisin in Manchester, Michigan in 1940 by the Henry Ford Motor Company to generate hydroelectric power. Since then, the use of hydropower generation has been abandoned. The dam and powerhouse were purchased in 2000 by the Village of Manchester and the powerhouse was reconfigured into the village offices. As of 2004 the dam is regulated and inspected by the Department of Environment, Great Lakes, and Energy (EGLE) Dam ID No. 391 and is rated as a High Hazard Dam. Prior to 2004 the dam was inspected by numerous other companies with the oldest provided inspection report dating back to 1978 prepared for the U.S. Army Corp of Engineers.

The dam structures consist of, from left to right looking downstream, a 540-foot-long left earth embankment, an abandoned intake and powerhouse, an 80.5-foot-wide concrete spillway, and a 190-foot-long right earth embankment.

The concrete structures making up the dam (including the powerhouse, spillway, walls, and intake) are founded on native hard sand gravel clay and boulder foundation. The concrete structures are supported by a slab-on-grade. The spillway slab-on-grade has a steel sheet pile (SSP) cutoff wall integral with the slab upstream of the spillway and three (3) seepage drains beneath the downstream spillway slab. The downstream spillway slab has a weir approximately 15 feet downstream of the M-52 bridge. The dam structures are depicted in Figure 1.

Figure 1: Dam Structure Locations

1.2.1 Spillway

The spillway consists of a single fixed crest weir approximately 80.5-foot-wide concrete spillway with two (2) 4-foot-diameter sluice gates. The sluice gates are currently in the closed potion and as of December 2023, there have been no documented events where the sluice gates would have been operated (raised and/or lowered). Once a gate is repaired or replaced, it will stay in the closed position.

The drawdown sluice gates are situated on the left and right sides of the spillway with a trash rack directly upstream. Between the spillway and the M-52 bridge there is a left retaining wall that also functions as the right side of the penstock and a right retaining wall that support the upstream side of the right embankment.

1.2.2 Powerhouse and Penstock

The intake structure for the Powerhouse is situated directly to the left of the spillway looking downstream and consists of an 8-foot-square concrete penstock with an angled trash rack. The penstock feeds two (2) twin turbines located at the powerhouse. The head gates at the powerhouse are currently in the closed position and there have been no documented events where the head gates would have been operated (raised and/or lowered). Therefore, the condition of the components can be observed but the operability is questionable since there are no records of the head gate having been operated.

1.2.3 Earthen Embankment

The earthen embankment is approximately 540 feet long to the left of the spillway and approximately 190 feet long to the right of the spillway. The earthen embankments have crest widths of approximately 35-feet and serve as the roadbed for M-52. A bridge is situated over the river channel directly downstream of the spillway. The upstream and downstream slopes are approximately 3 horizontal to 1 vertical.

1.2.4 Retaining walls and Appurtenant Structures

The walls consist of a combination of earth retaining walls, bridge abutment walls, powerhouse superstructure support, intake, and draft bay walls. Downstream of the M-52 bridge there are left and right retaining walls. The left retaining wall supports the downstream side of the left embankment (grassy area in front of the powerhouse) and the right side of the penstock. The right retaining wall supports the downstream side of the right embankment. At the left downstream retaining wall abutment at the powerhouse there are two (2) vault areas that are divided by the head gates. The operating equipment for the head gates are located within the vaults. At the left retaining wall abutment to the powerhouse the wall transitions into a structural wall that supports the superstructure of the powerhouse, and the powerhouse intake including the turbine and the draft bay. To the left of the draft bay section of the structural wall is a basement wall that supports the superstructure of the powerhouse. Downstream of the powerhouse the basement wall transitions into a retaining wall that supports an outdoor seating area for the village staff.

The walls consist of a combination of earth retaining walls, bridge abutment walls, powerhouse superstructure support, intake, and draft bay walls.

The upstream walls consist of a left-wing wall that abuts the penstock intake and a right-wing wall that abuts the spillway.

1.3 Assignment of Responsibility

The Ford Manchester Dam is currently owned and maintained by the Village of Manchester. The dam is regulated under Part 315, Dam Safety, of the Natural Resources and Environmental Protection Act, Public Act 451 of 1994, as amended.

1.4 Record Keeping

Documenting the current and past condition of the dam is necessary to assess the adequacy of operation, maintenance, surveillance, and proposed corrective actions. Dam records should be kept at a designated location at the Village of Manchester Office. The following records should be maintained at a minimum:

- Design and construction documents
- Documentation of major repair work
- Routine maintenance activities
- Maintenance and repair activities triggered by inspections
- Dam safety inspection reports
- Completed checklists from routine inspections
- Photo documentation from inspections
- Dam measurements
- Recorded reservoir levels and rain events

Inspections and maintenance should be completed in accordance with Section 3 and Section 5, respectively.

Immediately following an inspection, observations should be compared with previous records to see if there are any trends that may indicate developing problems. If a questionable change or trend is noted, and/or failure is imminent, the owner should consult a professional engineer experienced in dam safety. Reacting quickly to questionable conditions will ensure the safety and long life of a dam and possibly prevent costly repairs.

2. Operation Procedures

2.1 Reservoir Operations

The Dam has four major structural components: the spillway; penstock and powerhouse; and earthen embankment. The spillway is a fixed-crest weir approximately 80 feet long with an elevation of 792.63 feet. The dam is a passive spillway system. Once The Village has completed work recommended in 2023, the spillway will have a sluice gate that will remain in the closed position and used for lowering the impoundment level during inspections or maintenance.

The dam operator will track general weather trends and forecasts on a regular basis to provide forewarning for events that may result in heavy inflows into the reservoir. If significant rainfall is predicted, the operator should remove debris from the upstream areas of the spillway to minimize any reduction in spillway capacity.

When operating in flood conditions, visits to the dam should be made at least twice daily and the dam should be inspected during each visit for indications of distress. The dam does not have any active controls that can be adjusted based on flooding conditions. Therefore, the operator should refer to the Emergency Action Plan if there appears to be a potential for hazardous conditions. This may include conditions such as:

- Loss of earthen embankment material
- Loss of concrete abutment material
- Structural failure of the spillway
- High water levels
- Structural failure of the powerhouse

3. Monitoring Inspection

3.1 Types and Frequency of Inspections

An effective inspection program is essential to identification of problems at the dam that require maintenance, repair, or further evaluation. The program should involve four types of inspections:

- Periodic technical inspections
- Periodic regulatory inspections
- Monthly maintenance inspections
- Informal observations by project personnel as they operate the dam

Periodic technical inspections are comprehensive inspections and reviews of the dam's design and construction performed by engineering specialists engaged by the dam owner. These comprehensive inspections and reviews are recommended to take place at least once every ten (10) years or more frequently depending on the condition of the dam, the hazard potential, and the results of previous findings.

Periodic regulatory inspections are visual inspections with limited review of the dam design/construction/maintenance history and are performed by the owner and its qualified engineer or EGLE Dam Safety Division personnel (if ordered by EGLE). These inspections are completed in accordance with Part 315 of NREPA, typically on a recurring schedule once every three (3) years. A Dam Safety Inspection Report is prepared and submitted to EGLE.

Monthly maintenance inspections are visual inspections completed by the dam owner once per month. The inspection should include, at a minimum, a review of any potential new downstream development that may change the hazard potential, a visual inspection of the dam using the Ford Manchester Dam Inspection Checklist (Appendix A), and photographs of the dam.

Informal Observations can occur year-round at any time by any personnel that are operating or maintaining the dam. These personnel should feel empowered to check for deficiencies or unusual conditions and report them to the appropriate personnel. In addition, informal observations are recommended following certain events such as prior to a major storm event or heavy snowmelt, during or after a severe storm, or after an earthquake. If emergency conditions are observed, the staff should refer to the EAP for appropriate actions.

3.2 Performing an Inspection

Monthly maintenance inspections and informal observations should be documented using the checklist provided in Appendix A. During each inspection, the following are required:

- Record the weather (current weather and notable weather conditions from past week), the date of the inspection, and the persons in attendance.
- Document observations on the checklist form, including condition rating and comments.
- If conditions requiring maintenance are observed during the current inspection, perform the maintenance at the conclusion of the inspection, or make note to schedule the maintenance in a reasonable period of time.
- Inspection forms should be reviewed by appropriate Village personnel for noted changes with the dam or its appurtenances.

The entire structure and adjacent areas should be inspected regularly. During periods of extreme low flow over the spillway, observe downstream sill of the spillway slab and wall to identify if any visible erosion or scour that is occurring, or any other areas that may not otherwise be visible. It is important during these inspections to record measurements and photographs of observed deficiencies for future comparisons.

All individuals responsible for operating, inspecting, and maintaining the dam should receive proper training. These individuals include dam owners, dam operators, and DPW supervisors and personnel. An example of proper training can be found from the Association of State Dam Safety Officials (ASDSO). The ASDSO have a robust training center designed for dam owners and municipalities and can be used as a training tool.

3.2.1 Recommended Inspection Equipment/Materials

The inspectors should use the appropriate equipment to perform the inspection. Suggested equipment for performing inspections include:

- **Notebook and pencil** – should be available so that observations can be written down at the time they are made, reducing mistakes, and avoiding the need to return to the site to refresh the inspector's memory.
- **Inspection checklist** – serves as a reminder of all important conditions to be examined.
- **Digital camera** – can be used to photograph field conditions. Photographs should be taken from the same vantage points as previous photographs to allow for

comparison of past and present conditions. GPS enabled devices with timestamps are recommended.

- **Small erasable board** – can be used to note date, time, location, and pertinent information in a photograph.
- **Measuring tape** – allows for accurate measurements so that meaningful comparisons can be made of movements. (A clear plastic crack gauge is also recommended.)
- **Flashlight** – may be needed to inspect the interior of an outlet in a small dam.
- **Tapping device** – is used to determine the condition of support material behind concrete or asphalt faced dams by firmly tapping the surface of the facing material. Concrete fully supported by fill material produces a “click” or “bink” sound, while facing material over a void or hole produces a “clonk” or “bonk” sound. The device can be made from a 1-inch hardwood dowel with a metal tip firmly fixed to the tapping end or a length of reinforcing steel.
- **Binoculars** – useful for inspecting limited-access areas, especially on concrete dams.
- **Volume container and timer** – used to make accurate measurements of the rate of leakage. Various container sizes may be required, depending on the flow rates. (If seepage is observed).
- **Stakes, flagging tape, grease pencils** – used to mark areas requiring future attention and to stake the limits of existing conditions, such as wet areas, for future comparison.
- **Watertight boots or waders** – recommended for inspecting areas of the site where water is standing.
- **Personal protective equipment (PPE)** - Insect repellent, sunscreen, snake protection, other PPE as conditions dictate (e.g., air meters, harnesses, fall protection, personal floatation devices).

4. Dam Instrumentation

4.1 General

Dam Instrumentation refers to a variety of devices installed within, on, or near the dam to monitor structural behavior during construction, initial filling, and subsequent operation. Instruments provide a means for detecting and analyzing abnormal conditions that could lead to major problems.

This section describes the possible instrumentation to be installed at Ford Manchester Dam, the methods and frequency of data collection, transmittal of data, and procedures to evaluate the data. Timely evaluation of instrumentation readings is critical if an abnormal condition is to be detected, defined, and to allow for effective corrective action.
The following devices are the most common monitoring devices found:

1. ***Reservoir Staff Gage***: A graduated marker mounted on a structure within the reservoir or on a pole that is used to measure the water level in the reservoir.
2. ***Survey Monuments or Measurement Points***. A set of defined points (to be surveyed during the dam's life) from which the displacements that the dam undergoes may be measured.
3. ***Piezometers/Observation Wells.*** Used to measure the height of the water surface or hydrostatic pressure in the embankment.
4. ***Weirs and Seepage Outfalls***. Measures the quantity of leakage occurring through the embankment and/or foundation.

Instrumentation and proper monitoring and evaluation are extremely valuable in determining the performance of a dam. Specific information that instrumentation can provide includes:

- Warning of a problem (i.e., settlement, movement, seepage, instability)
- Definition and analysis of a problem, such as locating areas of concern
- Proof that behavior is as expected
- Evaluating remedial actions.

Currently, there is no instrumentation installed at the Ford Manchester Dam. Installation of instrumentation may be recommended at a future date.

5. Maintenance

5.1 Critical Conditions

The following conditions are critical and require immediate repair or maintenance under the direction of a qualified engineer retained by the dam owner. Critical conditions should trigger a response as outlined in the EAP.

- Erosion, slope failure or other conditions that are endangering the integrity of the dam
- Piping or internal erosion as evidenced by increasingly cloudy seepage or other symptoms
- Spillway blockage or restriction
- Excessive or rapidly increasing seepage appearing anywhere near the dam site.

5.2 Periodic Maintenance

The following items should be noted during normal inspections and added to the work schedule for maintenance/repair as soon as possible:

- Remove bushes and trees from the embankment and abutments
- Repair erosion gullies
- Repair deteriorated concrete or metal components
- Maintain riprap or other erosion protection.

5.3 Routine Maintenance

The following maintenance should be performed at the dam on a routine basis:

1. Control vegetation on the left and right abutment of the spillway, the needle section, the area surrounding the powerhouse, and on the upstream and downstream slope of the earthen embankment on a periodic basis.
2. Keep monthly inspection forms on file at the plant for reference. Inspection forms should be reviewed by appropriate plant personnel for noted changes with the dam or its appurtenances.

3. Monitor flow through the intake structure of the former powerhouse, including the removal of debris from the trash racks, to prevent a potential unsafe condition from developing within this structure.

4. Inspect bulkheaded penstock to ensure no water is present.

5. Monitor and photograph any present erosion and compare with previous erosion to monitor for any additional erosion.

6. If water levels are low enough, the Village should inspect and document the downstream toe and spillway walls for continued scour, erosion and/or undermining of the concrete structure.

7. Continue removing debris from the upstream areas of the spillway to prevent loss or reduction of spill capacity.

Additional recommended maintenance is described below.

5.3.1 Tree and Brush

Trees and brush should not be permitted on embankment surfaces or in vegetated earth spillways. A general rule of thumb is that no trees or woody vegetation should be allowed within 15 feet of the dam or appurtenant structures. Tree and brush growth adjacent to concrete walls and structures may eventually cause damage and should be removed.

5.3.2 Erosion

Erosion is a natural process, and its continuous forces will eventually wear down almost any surface or structure. Periodic and timely maintenance is essential in preventing continuous deterioration and possible failure. Prompt repair of vegetated areas that develop erosion is required to prevent more serious damage to the embankment. Not only should the eroded areas be repaired, but also the cause of the erosion should be addressed to prevent a continuing maintenance problem. Erosion might be aggravated by improper drainage, animal burrows, or other forces. The cause of the erosion will have a direct bearing on the type of repair needed.

5.3.3 Upstream Slope Protection

Effective slope protection must prevent soil from being removed from the embankment. When erosion occurs and benching develops on the upstream slope of a dam, repairs should be made as soon as possible. Riprap or other protection such as concrete bags should be monitored for deterioration from weathering. Freezing and thawing, wetting, and drying, abrasive wave action, and other natural processes can break down the material.

Maintenance may require repositioning any material that becomes displaced, replacing any material that becomes deteriorated or is missing, and removing vegetation.

5.3.4 Concrete

Repair of deteriorated concrete should be discussed with an engineer. Any vegetation observed growing from cracks in the concrete should be removed.

Over time, concrete surfaces will weather, leaving the concrete rough to the touch, or will hold moisture on the surface. When this occurs, consider applying a protective coating to the concrete to help prevent moisture from entering the structure. By applying a protective coating to the concrete surface and sealing the cracks the chances of freeze/thaw damage will be greatly reduced, increasing the life expectancy of the structure. Prior to the application of a concrete sealer, the structure should be cleaned, existing cracks should be sealed with a flexible sealant, and any spalling repaired. Any sealer chosen for the concrete should be a water or solvent-based acrylic protective coating, which may be either clear or colored, and may be textured.

Periodic maintenance should be performed on all concrete surfaces to repair deteriorated areas in coordination with the engineer. Repair deteriorated concrete as soon as possible when noted; it is most easily repaired in its early stages. Deterioration can accelerate and, if left unattended, can result in serious problems. Consult an experienced engineer to determine both the extent of deterioration and the proper method of repair. Seal joints and cracks in concrete structures to avoid damage beneath the concrete.

More serious damage such as spalling should be repaired as soon as it is identified, especially if steel reinforcing has been exposed. All surfaces to be patched need to be structurally sound, clean, and free of loose debris, oils, vegetation, paints, sealants, and other contaminants. Remove all deteriorated concrete to depth sufficient to avoid delamination of the repair (consult your engineer). Cut edges should be square with the concrete surface, and not feathered. Surfaces should be sufficiently rough to ensure a good bond. Any existing reinforcing bars should be thoroughly cleaned. If required, existing concrete should be removed to fully expose the reinforcing bar. Sandblasting may be required to clean them thoroughly. All surfaces should be fully saturated and freestanding excess water should be removed before applying the repair material.

Visible cracking, scaling, or spalling are signs of concrete movement and stresses within the concrete. Cracks in concrete walls that are not repaired are subject to freeze/thaw damage, which widens the gap and leads to additional spalling of the concrete. When examining any concrete structures, spalling, scaling, or cracking should be minimal.

5.3.5 Rodent Control

If rodent burrowing is occurring at or near the dam, a program to trap nuisance animals should be developed and implemented. This program should be extended until such time that there is no evidence of new burrowing activities in the dam embankments. Creating conditions inhospitable to the rodents should be a goal of the program by ensuring that tall grasses, trees, vegetation at the water line are maintained.

The recommended method of backfilling a burrow in an embankment is mud-packing. This method can be accomplished by placing one or two lengths of metal stove or vent pipe in a vertical position over the entrance of the den. Making sure that the pipe connection to the den does not leak, the mud-pack mixture is then poured into the pipe until the burrow and pipe are filled with the earth-water mixture. The pipe is removed and dry earth is tamped into the entrance. The mud-pack is made by adding water to a 90 percent earth and 10 percent cement mixture until a slurry or thin cement consistency is attained. All entrances should be plugged with the well-compacted earth and vegetation re-established. Dens should be eliminated without delay because damage from just one hole can lead to failure of a dam or levee.

Large active or collapsed burrows should be excavated to remove loose soil, and then filled with compacted lifts of the excavated soil or a new compatible borrow material. Prior to making any excavations into a dam embankment, the Dam Safety Division should be contacted to discuss permitting and engineering controls. Excavations should be conducted when water levels in the lake/reservoir are at a seasonal low.

Additional methods for preventing burrowing include the installation of graded rip-rap “barriers.” A properly constructed rip-rap filter and filter layer will discourage burrowing. The filter and rip-rap should extend at least three (3) feet below the water line. As an animal attempts to construct a burrow, the sand and gravel of the filter layer caves in and thus discourages den building. Heavy wire fencing laid flat against the slope and extending above and below the water line can also be effective. Eliminating or reducing aquatic vegetation along the shoreline will also discourage habitation.

5.3.6 Access Equipment

The Ford Manchester Dam has two operator gates and a platform above the intake structure for the penstock upstream of the M-52 bridge. The concrete platforms are surrounded by safety handrails. Safety handrails are also installed along the spillway. This equipment should be monitored and repaired as needed to maintain safe access.

Appendix A

Inspection Checklist

Ford Manchester Dam – Inspection Checklist
Village of Manchester

Inspected By	Inspection Date	Weather Conditions

	Item	Yes	No	N/A	[1]Condition (No Change – Maintenance –Monitor – Investigation)	Remarks
1	**General Condition of Dam**					
A	Alterations to the dam?					
B	Development in downstream floodplain?					
C	Grass cover adequate?					
D	Settlements, misalignments, or cracks?					
E	Recent high water marks?					elevation
2	**Upstream Slope of Dam**					
A	Erosion?					
B	Trees/woody vegetation?					
C	Rodent holes?					
D	Cracks, settlement, or bulges?					
E	Adequate and sound rip-rap? (if present)					
3	**Downstream Slope of Dam**					
A	Erosion?					
B	Trees/woody vegetation?					
C	Rodent holes?					
E	Cracks, settlement, or bulges?					
F	Seepage or boils?					Estimated gpm
4	**Retaining walls and Appurtenant Structures**					
A	Erosion, cracks, or slides?					
B	Seepage?					Estimated gpm
C	Cracking or spalling?					
D	Outfalls working?					
E	Corrosion or deterioration?					
5	**Primary Spillway and Operator Decks**					
A	Spalling, cracking, or scaling?					
B	Exposed reinforcement?					
C	Joints displaced or offset?					
D	Joint material lost?					
E	Leakage?					
F	Earth erosion?					
6	**Penstock (Exterior and Interior) and Former Powerhouse**					
A	Spalling, cracking, or scaling?					
B	Exposed reinforcement?					
C	Joints displaced or offset?					
D	Joint material lost?					
E	Leakage?					
F	Earth erosion?					
7	**Sluice Gate Operation (Semi-annually, Spring/Fall)**					
A	Operable?					
B	Proper Lubrication of mechanisms?					
C	Rust, damage, deterioration?					

REMARKS:

[1]**Condition**: The goal of the owner performed inspections is to identify potential adverse changes in the dam.

- **No Change:** Specific feature is consistent with observations during previous inspection.
- **Maintenance:** Maintenance required. Provide specific description of necessary maintenance activities (tree removal, mowing, debris removal, etc.).
- **Monitor:** A minor deficiency (crack, seepage, etc.) has been observed and ongoing monitoring will be performed to assess progression.
- **Investigation** – Concerning issue has been identified (boils, settlement, scour, etc.) and engineering investigation is required.

Appendix H

Disposition Study

Consulting Engineers and Scientists

The Ford Manchester Dam Disposition Study

Manchester, Michigan

Submitted to:
The Village of Manchester
912 City Road
Manchester, Michigan 48158

Submitted by:
GEI Consultants of Michigan, P.C.
4472 Mt. Hope Road
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December 15th, 2023
Project 2204052

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LRH/DD:lmc

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1. Introduction

1.1 Purpose

This study conducted by GEI Consultants of Michigan, P.C. (GEI) aims to evaluate three long-term alternatives for the Ford Manchester Dam: (1) maintaining under current regulations, (2) maintaining under potential regulatory changes, and (3) dam removal. By identifying key factors influencing the costs and feasibility of these alternatives, the study provides valuable insights to aid the Village of Manchester (the Village) in future planning and decision-making.

1.2 Site History and Dam Classification

The Ford Manchester Dam, located on the River Raisin, is owned by the Village. Originally built in 1940 by the Henry Ford Motor Company to supply hydroelectric power to a Ford assembly plant, the dam's hydropower generation has since been decommissioned. The former powerhouse now serves as the Village's office space. Designated as a high-hazard dam, the dam poses a substantial risk to human life and downstream property and infrastructure in the event of failure.

1.3 Dam Inspection Summary

The most recent dam inspection was completed on May 17, 2022, by the Department of Environment, Great Lakes, and Energy (EGLE) Dam Safety Unit. EGLE inspectors classified the dam to be in ‘fair’ condition. A dam in 'fair' condition is defined as 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. The recommendations provided in the EGLE inspection report are considered in the context of Alternative 1 – maintaining the dam and Alternative 2 – maintaining dam under potential regulatory changes.

1.4 Social and Community Considerations

This study included an examination of non-economic factors associated with each alternative. These factors may significantly influence decision-making but lack easily quantifiable costs. These considerations encompass the dam's impact on recreational activities, natural resources, and adherence to the Village Master Plan, among others. This evaluation considered the goals outlined in the Manchester Community Joint Master Plan (2017), The Manchester Joint Parks and Recreation Master Plan (2022), and the River Raisin Watershed

Management Plan by the River Raisin Watershed Council (2009) that apply to each alternative. The applicable goals evaluated are:

Manchester Community Joint Master Plan (2017)

- Conserve and enhance the community's natural resources, including lakes, rivers, wetlands, woodlands, and topography.
- Protect and enhance the River Raisin, tributaries, and watershed. Collaborate for improved water quality with Washtenaw County and the River Raisin Watershed Council.

Manchester Joint Parks and Recreation Master Plan (2022)

- Promote and develop a continuous River Raisin Greenway.
- Focus on future land acquisition and parkland development along the river to provide public access, connect with nature, and offer opportunities for physical activity.
- Serve both recreation and ecological goals by safeguarding riverfront habitat and biodiversity.
- Work towards acquiring and developing parkland and open spaces along the River Raisin.

River Raisin Watershed Management Plan (2009)

- Rehabilitate rare high-gradient habitats by removing dams no longer used for their original purpose, such as retired hydroelectric facilities.
- Address issues associated with dams creating small, shallow, and silt-laden impoundments.

These goals collectively emphasize the importance of environmental conservation, watershed protection, and the development of recreational spaces along the River Raisin. Given these considerations, the dam removal alternative would best meet the goals of these plans.

1.5 Dam Alternatives Evaluated

The three possible long-term alternatives identified by the Village include:

- Maintain the dam under current regulations.
- Maintain the dam under potential regulatory changes, and
- Dam removal.

Each section evaluates the benefits and limitations of the alternative and the associated costs.

2. Maintain the Dam under Current Regulations

This proposed alternative (Alternative 1) evaluates maintaining the Ford Manchester Dam and implementing essential repairs based on GEI's structural analysis recommendations. Additionally, this option encompasses long-term improvements aimed at sustaining the dam's functionality for a minimum of 50 years.

2.1 Considerations

This alternative primarily addressed the recommendations outlined in the EGLE dam safety inspection report dated May 17th, 2022. Following the inspection, the Village engaged GEI to perform a comprehensive structural assessment of the main spillway and powerhouse structure. The initial structural repairs necessary for dam stability, as identified in the GEI Stability Analysis Report, are incorporated into this alternative.

2.1.1 Initial Structural Repairs

The initial recommended structural repairs to the dam include the following:

- Replace at least one existing gate, operating it according to the latest Operation and Maintenance (O&M) program.
- Repair deteriorated concrete on the spillway and surrounding structures.
- Install a bulkhead on the upstream side of the penstock intake to obstruct flow into the penstock and powerhouse.
- Continue vegetation removal from embankments and address animal burrows.
- Monitor runoff and erosion on the downstream right embankment and reinforce as needed.

2.1.2 Ongoing Costs

After initial repairs are completed, ongoing financial commitments will be necessary for the dam. If not initially addressed, in the coming years other issues at the dam include the restoration or replacement of at least one sluice gate to ensure operational functionality. This replacement requires either a complete dewatering of the spillway or the installation of a temporary cofferdam.

Additional ongoing costs involve the operation and maintenance of the dam. Village personnel will need to regularly assess the dam's condition, conduct routine mowing, and ensure embankment slopes remain free from woody vegetation. They will also be responsible for keeping the spillway clear of debris and conducting regular checks to verify the functionality of all components.

If the Village chooses to maintain the dam, long-term structural retrofits similar to the current recommended repairs will be necessary. These continued repairs are essential to prevent failure, given the typical 50 to 100-year lifespan of dams.

2.1.3 Other Benefits and Drawbacks

In addition to action items needed to maintain the dam discussed above, Table 1 outlines other benefits and drawbacks of for this alternative.

Table 1: Benefits and Drawbacks of Maintaining Dam

Maintain Dam Alternative	
Benefit	Drawback
- Initial repairs for maintaining the dam could be less than dam removal. - Current recreational use maintained.	- Water quality issues and ecosystem disruption. - Disrupt fish passage. - Continued expense for the life of the dam. - Maintenance costs and aging infrastructure. - Continued sediment buildup.

2.2 Initial Cost Estimate

The cost for Alternative 1 is estimated at $1.6 million. This estimation is for the repairs to the concrete, bulkheading the penstock and replacing one gate on the dam. These estimates draw upon comparable project costs, engineering expertise, and published cost data. It is important to note that the actual bids and overall project expenses may vary, influenced by factors such as the contractor's perceived risks, site accessibility, seasonal conditions, market dynamics, and other related considerations. More detail regarding this cost estimate can be found in the Structural Analysis Report.

2.3 50-year Life Cycle Cost Estimate

Given the lifespan of a dam and the requirement for ongoing repairs, it is likely that maintenance similar to what is recommended in the Structural Repair Analysis Report will be needed approximately every 50 years. Additionally, over the next 50 years, the dam will necessitate annual maintenance, operations, periodic inspections, and insurance, incurring additional costs within the evaluated timeframe. Table 2 highlights and compares estimated long term costs of the dam, outlining initial repairs, 50-year life cycle cost represented in 2023 dollars, and an estimation of the 50-year life cycle cost in future spending based on a 5% annual inflation rate. Once this 50-year life cycle is complete, the dam will continue to require maintenance and repair as long as it stands. After the completion of this 50-year life cycle, the dam will necessitate ongoing maintenance and repairs for the duration of its existence.

Table 2: Cost Comparison for Maintaining Dam

Cost Comparison	
Initial Repairs	$1.6 Million
Life Cycle Cost of Dam through 50 years in 2023 dollars (including initial repairs)	$4.7 Million
Life Cycle Cost of Dam through 50 years in future spending (based on 5% inflation rate) (including initial repairs)	$22.4 Million

3. Maintain the Dam Under Potential Regulatory Changes

This proposed alternative (Alternative 2) evaluates maintaining the Ford Manchester Dam by implementing essential repairs based on GEI's structural analysis recommendations identified in Section 2.1.1 with the additional consideration for dam improvements that might be required based on proposed amendments to EGLE Dam Safety regulations (Part 315 of NREPA).

3.1 Considerations

In 2021, the EGLE Dam Safety Task Force released a document outlining recommended more stringent regulatory requirements to enhance dam safety in Michigan, which align with national standards. These proposals suggest amendments to Part 315, Dam Safety (Part 315) of the Natural Resources and Environmental Protection Act, 1994 PA 451, as amended. At the time of this report, it is uncertain when and if these recommendations will be included in the Dam Safety Act. However, given the life span of a dam, it is in the interest of the Village to evaluate potential long-term added costs if legislation approves more stringent measures. Table 3 highlights the major potential regulatory changes that would most significantly impact long-term maintenance of the Ford Manchester Dam and Village obligations. These recommended changes are based on the dam's classification as a 'High Hazard' dam by the state of Michigan.

Table 3: Summary of Potential Regulatory Changes for High Hazard Dams

Regulatory Change	Current	Proposed
Engineering Inspections	3 years	1 year (visual), 10 years (in-depth evaluation)
Spillway Capacity	200-year (1/2 PMF if over 40 feet high) or flood of record	PMF or IDF
Licensing Requirements	None	15-year Registration
Financial Assurance	None	Required
Insurance	None	Required
Emergency Action Plan	Update Annually – No Exercise Requirements	Update Annually – 5-year Exercise Requirement

3.1.1 Dam Inspection Frequency

If dam regulations change, the Village may be required to contract and fund yearly high-level visual dam inspections much like what was done in 2022, if not provided by the State as currently done. In addition to annual inspections, the Village will also be required to perform periodic (no more than every 10 years) independent comprehensive reviews of the original design, construction, maintenance, repair, and probable failure modes conducted by a qualified and licensed team of engineers. This comprehensive assessment will likely include exploratory investigations and detailed engineering analyses.

3.1.2 Spillway Capacity

The spillway capacity at the Ford Manchester dam currently meets dam safety requirements for a 200-year flood discharge. However, updated regulations will necessitate spillway capacity considerations for the Probable Maximum Flow (PMF) or Inflow Design Flood (IDF) events. The PMF is considered the flood that would be expected from the most severe combination of critical meteorological and hydrologic conditions. The IDF is the flood flow above which the incremental increase in downstream water surface elevation due to failure of a dam is no longer considered to present an unacceptable additional downstream threat. Both PMF and IDF events can exceed the magnitude of a 200-year flood, potentially requiring an increased spillway capacity. Analysis based on data from a comparable site suggests that accommodating a PMF storm event could potentially necessitate doubling the spillway capacity or require significant dam modifications.

Determining the maximum IDF utilizes a risk-based approach for sizing the spillway, versus the prescriptive approach of the PMF. Determining the IDF event through hydraulic modeling specific to this site may result in a spillway capacity lower than the PMF. Consequently, the existing spillway may require only nominal improvements to meet the necessary standards. The establishment of site-specific PMF and IDF values will necessitate completing Hydrologic and Hydraulic (H&H) Modeling of the dam and dam breach inundation analyses.

3.1.3 Licensing Requirements

Under current regulations, a dam owner only seeks a permit through the State of Michigan at the time of construction. The proposed regulations necessitate the Village of Manchester to apply for a license renewal every 15 years. During the renewal process the Village will report on maintenance, operation, and engineering investigations, including annual inspection reports and independent comprehensive reviews. Failure to secure a license renewal could require the removal of the dam at the Village’s expense.

The recommended licensing requirements dictate that the dam owner must maintain adequate insurance to cover all liabilities resulting from a dam failure. As part of the licensing renewal, the Village is also required to provide evidence of fiscal responsibility or security to ensure the continued safe operation and maintenance of the dam.

3.1.4 Other Benefits and Drawbacks

In addition to action items needed to maintain the dam discussed above, Table 4 outlines other benefits and drawbacks of for this alternative.

Table 4: Benefits and Drawbacks of Maintaining Dam Under Potential Regulatory Changes

Maintain Dam Alternative	
Benefit	Drawback
- Current recreational use maintained. - Upgraded structure would meet proposed regulatory amendments.	- Water quality issues and ecosystem disruption. - Disrupt fish passage. - Continued expense for the life of the dam. - Maintenance costs and aging infrastructure. - Continued sediment buildup. - Regulatory upgrades likely more expensive than alternative 1.

3.2 Initial Cost Estimate

Initial repair cost is projected at $1,600,000, aligning with Alternative 1. For any dam-in scenario, these repairs are required in the near-term.

3.3 50-year Life Cycle Cost Estimate

As outlined in section 2.3, the next 50 years the dam will require annual maintenance, operations, periodic inspections, and insurance for the dam, resulting in additional costs within the assessed timeframe. The inspection frequencies are increased to annually as outlined in section 3.1.1 and comprehensive assessments are included every ten years. The cost of spillway capacity increase is anticipated to be incurred within 10 years, assuming a grandfather period to meet new regulations. Table 2 provides a comparative analysis of the dam's estimated long-term costs, accounting for potential legislative changes. Table 5, similar to the discussion in section 2.3, compares initial repairs, the 50-year life cycle cost in 2023 dollars, and an estimate of the 50-year life cycle cost adjusted for a 5% annual inflation rate. The increased costs are attributed to expanding spillway capacity, increasing insurance coverage, and intensifying inspection requirements, necessitating more thorough examinations.

Table 5: Cost Comparison for Maintaining Dam Under Potential Legislative Changes

Cost Comparison	
Initial Repairs	$1.6 Million
Life Cycle Cost of Dam through 50 years in 2023 dollars (including initial repairs)	$7.8 Million
Life Cycle Cost of Dam through 50 years in future spending (Based on 5% inflation rate) (including initial repairs)	$36.7 Million

4. Dam Removal

The proposed alternative (Alternative 3) is the removal of the Ford Manchester Dam which includes full removal of the spillway, and all related components to restore the River Raisin to a more natural condition. Structures required for supporting the old powerhouse/village office and state highway M-52 will remain in place. The steps needed for dam removal vary significantly based on site-specific factors. These factors include: the quantity of sediment built up behind the impoundment, onsite sediment contaminants, river restoration measures, and the effects dewatering the impoundment will have on the remaining structures and infrastructure.

4.1 Considerations

For this evaluation, no site-specific data collection was conducted, and the considerations below are based on a typical dam removal and desktop analyses. For a clear understanding of the Ford Manchester Dam impoundment, dam, and surrounding infrastructure not related to the dam, such as the M-52 bridge running over the spillway, onsite investigation and data collection will be required. These additional investigations could include depth of refusal, data collection to understand sediment thicknesses within the impoundment, sediment sampling to test for contaminates, geotechnical borings and review all drawings and structural analyses from M-52 and dam appurtenances.

4.1.1 Sediment Management and Characterization

One of the leading cost factors for dam removal depends on the quantity and quality of sediment within an impoundment. In a typical impoundment, sediment accumulation gradually builds up behind the dam, forming a wedge-shaped deposit. As water and the sediment it carries flows into the impoundment it slows down and the sediment settles out due to the reduced velocity of the water. Over time, this sediment accumulates, creating a wedge-like configuration that extends from upstream extent of the impoundment towards the dam much like what is found in Figure 1. The exact method of sediment management and probable removal will depend on site specific sediment volume and characterization.

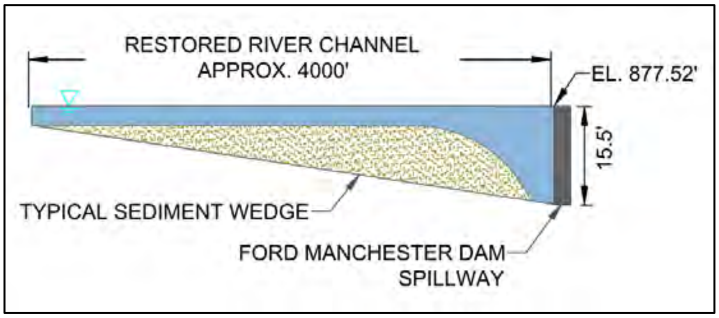

Figure 1: Probable Sediment Wedge in the impoundment

If the quality and volume of sediment meet certain EGLE guidelines, it is conceivable that through controlled dewatering, accumulated sediment could be allowed to travel downstream or could be excavated and placed onsite in upland areas. Conversely, if the exposed sediment within the impoundment holds contaminants that exceed levels greater than the residential direct contract criteria also set by EGLE, the sediment may have to be transported to an appropriate disposal site/facility. Site investigations need to be conducted to determine appropriate site-specific sediment management. The regulatory requirements and fate of accumulated sediment often has one of the largest impacts on dam removal costs and therefore is a factor that should be understood as early as possible in the process of a dam removal project.

4.1.2 Removal of Dam and Management of Water

Control of water during dam removal is a critical aspect that should be considered throughout the design and construction phases of the project to limit dam safety concerns related to an uncontrolled release of water and to adequately address sediment management and transport downstream. There are several ways to dewater an impoundment and control the flow of water during a dam removal project. Often, to remove the spillway and associated structures, a temporary cofferdam is installed and flow from the impoundment diverted around the dam. Once flow is diverted, the dam would then be deconstructed in a controlled manner. Other methods for dewatering can be considered and include bypass pumping or siphon system, or incremental demolition within active flow. Every dam removal is unique in the site characteristics and layout of the existing infrastructure. Based on initial structural evaluations, it is anticipated that incremental demolition could be a viable option at this site. However, further hydrologic, hydraulic and structural investigations would be required during the design phase.

4.1.3 River Restoration Measures

After the dam is removed, the restored river channel is returned to a more natural form by creating a channel to resemble the stream bank width, depth, and meanderings of the pre-dam river channel. This restores the natural hydraulics of the river and reintroduces sediment transport to the river reach. Along with the river channel, it is important to establish a sufficient floodplain to provide relief for larger flood flows and encourage a stable river channel. Figure 2 illustrates a possible stream restoration overview for the Ford Manchester Dam Impoundment.

Figure 2: Possible Stream Restoration Overview

The site-specific design would depend on river geometries gathered at an appropriate reference reach of the River Raisin that is in a stable condition and verified through hydrologic and hydraulic analyses. Specific attributes measured within this reach include bankfull width and depth, or the width and depth of the channel just before the water enters the floodplain, the width of the floodplain bench, and the sinuosity of the river. For this water body, the sinuosity of the river is crucial given the distinct meandering of the River Raisin. Figure 3 from Wildland Hydrology visually depicts the design characteristics factored into a river restoration design.

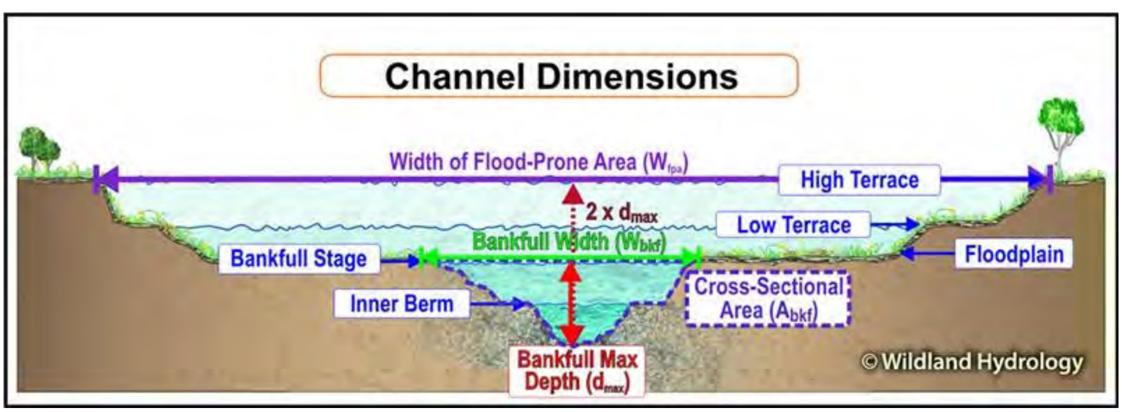

Figure 3: Cross Section of a Typical Restored Channel

In addition to stream channel restoration and establishing a sufficient floodplain, bank stabilization measures and habitat structures may be installed to promote restoration of the channel and floodplain. This work could include design and construction of large wood structures and seeding or planting plans for the exposed bottomlands.

4.1.4 Impoundment Property Ownership

Based on our desktop analysis, there appear to be at least thirty-two acres of land that will be exposed from dam removal. Most or perhaps all this land would be floodplain or wetlands and perhaps some fringe of it would become upland. Based on information obtained from the publicly available online Washtenaw County Plat map and illustrated in Figure 4.

Figure 4: Parcel Map from Washtenaw County Plat Map

The Village stands to gain more than five acres of land from the dewatered impoundment. This property is located adjacent to the Ford Manchester Dam and south of Furnace St. Further, a single individual privately owns a sizable portion of the impoundment. Title work will be necessary to understand the property rights associated with the bottomlands. Additionally, coordination with the adjacent property owner will be necessary.

In addition to the dam removal action items discussed above, Table 6 outlines other benefits and drawbacks of the dam removal alternative.

Table 6: Other Benefits and Drawbacks of Dam Removal

Dam Removal Alternative	
Benefit	Drawback
- Improved condition of river ecosystem and surrounding natural resources. - Possible parkland development opportunities for the Village. - Removing all future expenses associated with the dam. - Mitigating risk from the dam structure or a dam failure. - Greater potential for outside funding opportunities to complete work.	- Immediate upfront costs to rehabilitate the dam may cost less than removal of the dam. - Change in recreational use of impoundment.

4.2 Initial Cost Estimate

Because the scope of this feasibility study did not include any onsite data collection the cost estimate is based on previous work at other locations. Site investigations are necessary a more accurate estimate as some components can vary greatly in cost from location to location. For example, the cost of managing sediment can vary from $15 - $75 per cubic yard and is based on the quantity of sediment in the impoundment and any contaminants found within the sediment. The cost estimate for removing the dam is $5.3 to 7.5 million if sediments are clean. If sediments are contaminated, this could expand to $6.2 – 8.4 million. Depending on the sediment contamination concerns, costs could extend beyond the material that is being excavated. For a more accurate estimation, additional studies have been recommended in Section 6.

4.3 50-year Life Cycle Cost Estimate

If the dam is removed, long-term maintenance and upkeep costs become negligible. Table 7, same as the discussion in Section 2.3 and 3.3, contrasts the removal cost, the 50-year life cycle cost in 2023 dollars, and an estimate of the 50-year life cycle cost adjusted for a 5% annual inflation rate. In this alternative, the dam would be removed, and a natural river channel would replace it, eliminating the need for any future maintenance or repair.

Table 7: Cost Comparison for Removing the Dam

Cost Comparison	
Initial Repairs	$5.3 – $7.5 Million
Life Cycle Cost of Dam through 50 years in 2023 dollars (including removal costs)	$5.3 – $7.5 Million
Life Cycle Cost of Dam through 50 years in future spending (Based on 5% inflation rate) (including removal costs)	$5.3 – $7.5 Million

5. Potential Funding Sources

There are potentially several funding sources available to aid the Village with design and construction costs for both maintaining the dam and removing the dam. Because many of the funding sources focus on the reestablishment of fish passage, ecological restoration, and increased river connectivity, most of the funding available is associated with dam removal. These funding sources, which include investments from local, state, and federal agencies, often require matching contributions from the project applicant. The rules for matching contribution percentages vary based on the funding source.

5.1 Funding to Maintain Dam

Limited funding opportunities are available for qualified recipients seeking dam rehabilitation. These funding sources would consider the Ford Manchester dam's overall risk, the extent of necessary repairs of the proposed projects, and the resulting risk reduction from the proposed project. While the dam is classified as high hazard, the Structural Analysis Report completed by GEI indicates it is in good condition, which may negatively impact the Village's eligibility for grant funding.

5.2 Funding for Dam Removal

Many of the available funding sources are based on a competitive pool of applicants where dam removal projects or other aquatic restoration projects are evaluated based on the amount of upstream habitat that is opened because of the removal. In the case of Ford Manchester Dam, the next upstream barrier is the privately owned Manchester Mill Dam less than a mile away within the Village of Manchester. Because of this short stretch of river, other projects reconnecting a much greater length are likely to receive a higher ranking than the removal of the Ford Manchester Dam. As such, there may be incentives for the Village in partnering with the owner of the Mill Dam to develop a more comprehensive dam removal and river restoration project. If the Mill Dam were also to be removed, this would open approximately 15 miles of River Raisin.

Regardless, the River Raisin Watershed council is interested in restoring this area of the watershed basin and would potentially be interested in partnering with the Village if the Village were to move forward with dam removal. Additionally, with the increased focus on dam safety risk reduction, there are currently State and Federal funding sources for dam removal projects. Being a high hazard dam, dam removal at this site would score well with these grant programs.

Appendix B includes a spreadsheet of known potential funding sources that could aid the Village in funding the rehab and removal activities.

6. Additional Studies Needed

Based on our experience completing dam removal and dam rehabilitation projects as well as our review of available information, the following additional data investigations and analyses area recommended for alternatives 2 and 3:

- Structural analysis of the M-52 overpass substructure and superstructure systems and the concrete wall adjacent to the Village offices.
- Geotechnical investigations.
- Hydrologic and Hydraulic (H&H) Modeling of River Raisin and impoundment within the project study area, including dam breach inundation mapping (if maintaining dam).
- Sediment quantification and classification through sediment testing and sampling.
- River reference reach investigations to inform river restoration design.

7. Summary

Based on the estimates and information presented in this report, The Village and its community must assess both engineering and non-engineering factors when selecting the most suitable alternative. Some of these factors include:

- Initial cost of repairs and removal,
- Life cycle cost of maintenance and upkeep,
- Potential funding opportunities for each alternative,
- Risk liability of the dam,
- Community and local organization interest in maintaining or removing the dam and
- Future use of the impoundment or floodplain after dam rehabilitation or river restoration.

Assessing these factors, along with others identified in this report and by The Village, will aid in determining the most appropriate alternative for the Village of Manchester. Table 8 summarizes the cost comparison of the three alternatives, as cost typically plays a significant role in the decision-making process.

Table 8: Cost Comparison for Removing the Dam

Cost Comparison			
	Alternative 1	Alternative 2*	Alternative 3*
Initial Repairs	$1.6 Million	$1.6 Million	$5.3 – $7.5 Million
Life Cycle Cost of Dam through 50 years in 2023 dollars	$4.7 Million	$7.8 Million	$5.3 – $7.5 Million
Life Cycle Cost of Dam through 50 years in future spending (Based on 5% inflation rate)	$22.4 Million	$36.7 Million	$5.3 – $7.5 Million
*These estimates are based on desktop analysis and similar projects, not on site-specific data.			

Appendix A

Cost Estimate

Appendix A - Ford Manchester Dam
Opinion of Probable Cost - Conceptual Design

OPINION OF PROBABLE COST - CONCEPTUAL DESIGN

Project: Manchester Disposition Study
Client: The Village of Manchester
Dam/Scenario: Dam Repair

Project No.: 2204052
Date: 12/14/2023
Estimated by: LH/JM
Checked by: DD

5% Assumed Annual Interest Rate

Item	Description	Estimated Cost	Years to Expenditure	Today's Dollars	Future dollars	Notes
0.00	**Maintain Dam Scenario**					
0.01	Initial Repair Cost	$ 1,600,000	0	$ 1,600,000	$ 1,600,000	Based on GEI Structural Analysis
			Subtotal	**$ 1,600,000**	**$ 1,600,000**	
1.00	**50-Year Life Cycle Regulatory Requirements - No Legistlation Change**					
1.01	Inspections (3 year cycle)	$ -	0			EGLE currently provides inspections every 3 years.
1.02	Maintenance and Operations	$ 10,000	0	$ 500,000.00	$ 2,090,000	Total cost of standard operation and maintenance
1.03	Inspections In Depth (every 10yrs)		10		$ -	
1.04	Licensing and Insurance Requirements (annual)	$ 10,000	0	$ 500,000.00	$ 2,090,000	Estimated Cost - Obtain current insurance coverage for more accurate value.
1.05	Increased Spillway Capacity (10yrs)		10	$ -		
1.06	Major rehabilitation/repairs	$ 1,000,000	50	$ 1,000,000.00	$ 11,470,000	Assume substantial repairs every 50 years. End of 50-year life cycle.
			Subtotal	**$ 2,000,000**	**$ 15,650,000**	
			Estimated 50-year Life Cycle Cost	$ 3,600,000	$ 17,250,000	
			Contingency (30%)	$ 1,080,000	$ 5,180,000	
			Total 50-year Life Cycle Cost	**$ 4,680,000**	**$ 22,430,000**	
2.00	**50-Year Life Cycle Regulatory Requirements - Legislation change**					
2.01	Inspections (annual)	$ 10,000	0	$ 500,000	$ 2,090,000	Assuming EGLE will no longer provide inspections
1.02	Maintenance and Operations	$ 10,000	0	$ 500,000	$ 2,090,000	Total cost of standard operation and maintenance
1.03	Inspections In Depth (every 10yrs)	$ 100,000	10	$ 500,000	$ 2,710,000	In depth inspecton - Year 10, 20, 30, 40 & 50
1.04	Licensing and Insurance Requirements (annual)	$ 20,000	0	$ 1,000,000	$ 4,190,000	Estimated Cost - Obtain current insurance coverage for more accurate value and adjust for additional coverage.
1.05	Increased Spillway Capacity (10yrs)	$ 2,500,000	10	$ 2,500,000	$ 4,070,000	In 10 years, modify spillway to meet PMF/IDF flow rates.
1.06	Major rehabilitation/repairs	$ 1,000,000	50	$ 1,000,000	$ 11,470,000	Assume substantial repairs every 50 years. End of 50-year life cycle.
			Subtotal	**$ 6,000,000**	**$ 26,620,000**	
			Initial Construction Cost	$ 7,600,000	$ 28,220,000	
			Contingency (30%)	$ 2,280,000	$ 8,470,000	
			Total 50yr Life Cycle Cost	**$ 9,880,000**	**$ 36,690,000**	

Information presented on this sheet represents our opinion of probable costs in 2023 dollars. Unit and lump-sum prices are based on costs for similar projects, engineering judgment, and/or published cost data. Client administrative/engineering costs and regulatory fees not included. Actual bids and total project costs may vary based on contractor's perceived risk, site access, season, market conditions, etc. No warranties concerning the accuracy of costs presented herein are expressed or implied. Future dollars is calculated using an inflation rate of 5% per year over 50 years where applicable.

Appendix A - Ford Manchester Dam
Opinion of Probable Cost - Conceptual Design

OPINION OF PROBABLE COST - CONCEPTUAL DESIGN

Project: Manchester Disposition Study
Client: The Village of Manchester
Dam Removal

Project No.: 2204052
Date: 12/14/2023
Estimated by: LH/JM
Checked by: DD

	Item	Description	Quantity	Units	Unit Price	Total Cost	Notes
	1.00	**Water Management**					
	1.01	Erosion and Sediment Control	1	LS	$ 50,000	$ 50,000	
	1.02	Temporary Access Roads, Facilities and Laydown Areas	1	LS	$ 150,000	$ 150,000	
	1.03	Incremental Demolition and Construction Dewatering	1	LS	$ 600,000	$ 600,000	$15,000/day for 30 days + misc dewatering for restoration
					Subtotal	**$ 800,000**	
Clean Sediment	**2.00**	**Dam Removal - Clean Sediment**					
	2.01	Concrete Demolition	5,000	CYD	$ 200	$ 1,000,000	
	2.02	Excavation	10,000	CY	$ 15	$ 150,000	
	2.03	Constructed Engineered Riffle	2,593	CYD	$ 150	$ 390,000	
	2.04	Powerhouse and Bridge Structural Modifications	1	LS	$ 1,000,000	$ 1,000,000	
					Subtotal	**$ 2,540,000**	
	3.00	**Stream Restoration - Clean Sediment**					
	3.01	Stream Restoration (passive)	4,100	LFT	$ 50	$ 210,000	
	3.02	Stream Restoration (full restoration)	4,100	LFT	$ 400	$ 1,640,000	
					Construction Subtotal Passive Restoration	**$ 3,540,000**	
					Construction Subtotal Full Restoration	**$ 4,980,000**	
		Passive Clean Sediment					
	4.00	Unknown Scope Items			30%	**$ 1,060,000**	Unknown Scope Items
	5.00	Engineering Design and Permitting			10%	**$ 350,000**	Engineering Design and Permitting
	6.00	Engineering and Construction Observation			10%	**$ 350,000**	Engineering and Construction Observation
		Full Restoration Clean Sediment					
	4.00	Unknown Scope Items			30%	**$ 1,490,000**	Unknown Scope Items
	5.00	Engineering Design and Permitting			10%	**$ 500,000**	Engineering Design and Permitting
	6.00	Engineering and Construction Observation			10%	**$ 500,000**	Engineering and Construction Observation
		Clean Sediment Passive Restoration Total Estimated Cost				**$ 5,320,000**	
		Clean Sediment Full Restoration Total Estimated Cost				**$ 7,470,000**	
Contaminated Sediment	**2.00**	**Dam Removal - Contaminated Sediment**					
	2.01	Concrete Demolition	5,000	CYD	$ 200	$ 1,000,000	
	2.02	Excavation	10,000	CY	$ 75	$ 750,000	
	2.03	Constructed Engineered Riffle	2,593	CYD	$ 150	$ 390,000	
	2.04	Powerhouse and Bridge Structural Modifications	1	LS	$ 1,000,000	$ 1,000,000	
					Subtotal	**$ 3,140,000**	
	3.00	**Stream Restoration - Contaminated Sediment**					
	3.01	Stream Restoration (passive)	4,100	LFT	$ 50	$ 210,000	
	3.02	Stream Restoration (full restoration)	4,100	LFT	$ 400	$ 1,640,000	
					Construction Subtotal Passive Restoration	**$ 4,140,000**	
					Construction Subtotal Full Restoration	**$ 5,580,000**	
		Passive Contaminated Sediment					
	4.00	Unknown Scope Items			30%	**$ 1,240,000**	Unknown Scope Items
	5.00	Engineering Design and Permitting			10%	**$ 410,000**	Engineering Design and Permitting
	6.00	Engineering and Construction Observation			10%	**$ 410,000**	Engineering and Construction Observation
		Full Restoration Clean Sediment					
	4.00	Unknown Scope Items			30%	**$ 1,670,000**	Unknown Scope Items
	5.00	Engineering Design and Permitting			10%	**$ 560,000**	Engineering Design and Permitting
	6.00	Engineering and Construction Observation			10%	**$ 560,000**	Engineering and Construction Observation
		Clean Sediment Passive Restoration Total Estimated Cost				**$ 6,220,000**	
		Clean Sediment Full Restoration Total Estimated Cost				**$ 8,370,000**	

Information presented on this sheet represents our opinion of probable costs in 2023 dollars. Unit and lump-sum prices are based on costs for similar projects, engineering judgment, and/or published cost data. Client administrative/engineering costs and regulatory fees not included. Actual bids and total project costs may vary based on contractor's perceived risk, site access, season, market conditions, etc. No warranties concerning the accuracy of costs presented herein are expressed or implied. Future dollars is calculated using an inflation rate of 5% per year over 50 years where applicable.

Appendix B

Potential Grant Funding Sources

GEI Consultants, Inc.
1801130 - City of Ann Arbor, Barton Dam and Superior Dam
Superior Dam Removal - Potential Funding Sources

Fiscal Year	Due Date	Organization	Program Topic/Name	Project Types Funded (Key Words)	Eligible Grantees	$/Grant	Match	Web Links	Geographical Boundaries/Limits	Contact	Phone	Email
2023	2/24/23	GLFT - Great Lakes Fishery Trust	Ecosystem Health and Sustainable Fish Populations: Habitat Protection and Restoration	preserve essential habitat; protect, restore, and stabilize important fish habitats; increase habitat availability	non-profit orgs, educational institutions, state, tribal and local governments	$500,000 for disbursement		https://portal.glft.org/documents/2641-2023_glft_habitat_protection_application_guidance-pdf	Great Lakes Basin	Kathryn Frens	(517) 371-7468	kfrens@glft.org
2023	5/31/2023	NFWF - National Fish and Wildlife Foundation	Sustain Our Great Lakes	improve and enhance: Stream and riparian habitat, coastal wetlands, and Great Lakes and tributaries water quality	non-profit orgs, educational institutions, state, tribal and local governments	$200,000 to $1,000,000.	1:1 preferred	http://www.nfwf.org/greatlakes/Pages/2018rfp.aspx	Great Lakes basin	Aislinn Gauchay	612-564-7284	aislinn.gauchay@nfwf.org
2024	1/24/24	MDNR - Michigan Department of Natural Resources	Fisheries Habitat Grant Program	rehabilitate inland lakes, Great Lakes, rivers and streams habitat whose key physical processes that control aquatic habitat and fish production are impaired, including key processes : hydrology; connectivity; material recruitment and movement; geomorphology; and water quality.	non-profit orgs; local, state, federal and tribal government agencies	$25,000+	minimum 10%	https://www.nfwf.org/programs/sustain-our-great-lakes-program/sustain-our-great-lakes-2023-request-proposals	State of Michigan	Chip Kosloski	517-284-5965	kosloskic3@michigan.gov
2023	6/15/23	Trout Unlimited	Embrace a Stream Program	coldwater fisheries conservation, on-the-ground restoration, protection, conservation that benefit trout and salmon fisheries	TU councils and chapters	$10,000	1:1	http://www.tu.org/conservation/watershed-restoration-home-rivers-initiative/embrace-a-stream	Nationwide	Mike Kuhr	(414) 588-4281	mikek.trout@yahoo.com
2024	1/21/24	EGLE - Michigan Department of Environment, Great Lakes, and Energy	Dam Risk Reduction Program	dam removal, critical maintenance	Entities that own or operate a dam in the state of Michigan	$350,000 for all projects	10%	https://www.michigan.gov/dnr/buy-and-apply/grants/aq-wl/dams	Michigan	Mason Manuszak	989-370-1528	ManuszakM@Michigan.gov
2023		USFWS - U.S. Fish and Wildlife Service	Midwest Region Fish Passage Program	dam and barrier removal	government, watershed groups, tribes, others			https://www.fws.gov/fisheries/fish-passage.html	Wisconsin, Ohio, Missouri, Minnesota, Michigan, Indiana, Illinois, and Iowa			