# Design Documentation Report

## Discharge Capacity Upgrades

**Indianford Dam**

**Dam Key Sequence No. 608**

**Field File No. 53.04**

Report prepared for

Rock Koshkonong Lake District

Report prepared by

Mead & Hunt

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# 1. Introduction

## A. Project description

The Indianford Dam (Project) is located on the Rock River in Indianford, Wisconsin and is the hydraulic control for Lake Koshkonong located upstream. The Project is owned and operated by the Rock-Koshkonong Lake District (RKLD). The Project structures consist of a gated spillway, an uncontrolled overflow spillway, a non-overflow structure that serves as the right spillway abutment, and a decommissioned powerhouse with functional wicket gates.[1]

## B. Background

The RKLD has found it increasingly difficult to maintain the level of Lake Koshkonong in recent years due to high flows in the Rock River and limited discharge capacity at the Project. These conditions have hindered the RKLD's ability to comply with the water level restrictions imposed by the Wisconsin Department of Natural Resources (WDNR) and have greatly increased the number of days that slow/no-wake conditions have been declared on the lake.

The RKLD seeks to increase the discharge capacity of the Project to improve their ability to control upstream lake levels. The RKLD hired Mead & Hunt, Inc. (Mead & Hunt) in December 2019 to act as their design engineer. Mead & Hunt's initial design concept involved construction of a new crest gate spillway between the uncontrolled overflow spillway and the powerhouse, where the non-overflow section currently exists. However, we identified an alternative solution during the conceptual design phase – modification of the decommissioned powerhouse to significantly increase its discharge capacity, which is currently limited by how much flow the wicket gates can pass. This alternative has several advantages over the more conventional gated spillway approach, including:

- Approach and discharge channels already exist, significantly limiting the extent of work in the waterway.
- Due primarily to the width of the intake structure, the discharge capacity of the modified powerhouse will be approximately three to four times greater than that of the gated spillway option.
- There are fewer unknowns associated with the powerhouse retrofit alternative in comparison to the gated spillway option.

Based on these considerations, the RKLD Board of Commissioners voted unanimously to have Mead & Hunt move forward with design of modifications to both powerhouse bays to increase the Project's discharge capacity.

[1] Project structures are listed left to right (east to west) looking downstream.

## C. Proposed work

The proposed Project modifications generally consist of the following:

- Reinforcement of the east/west walls and divider wall in the powerhouse to allow for removal of portions of the turbine pit floors
- Removal of the wicket gates, draft tube cones, and associated mechanicals
- Selective demolition and removal of a portion of the turbine pit floors
- Placement of new concrete within turbine pits and draft tubes to improve discharge hydraulics
- Removal of the existing upstream bulkheads and bulkhead support frames
- Fabrication of six, new slide gates and installation of these gates in the existing bulkhead slots
- Procurement of dedicated rising-stem operators for all six slide gates
- Upgrades to the existing electrical service to provide sufficient power for the gate operators
- Construction of a new operator support frame and integral operating platform, including interior stairway access and appropriate safety measures
- Reconfiguration of the existing debris boom to improve handling of large debris
- Removal of the existing trash rack to allow leaves and small debris to be sluiced through the powerhouse
- Construction of a permanent gravel equipment pad to allow for removal of large, waterlogged debris that passes under the debris boom
- Underwater concrete surface repairs at intake piers and abutments
- Installation of steel framing and grating over existing holes in powerhouse floor

Drawings showing the work to be performed are provided in Appendix A.

## D. Schedule

The RKLD plans to bid this project within one month of receiving the WDNR's approval and intends to select a contractor in mid to late March 2021. We anticipate that construction activities will commence in April 2021, as river conditions allow.

We anticipate that the contractor will want to use the new slide gates as a means of dewatering the powerhouse to allow for the proposed structural modifications. As such, the anticipated sequence of major work items is as follows.

- Initiate fabrication and procurement of slide gates and slide gate operators (6 to 8-month lead time anticipated)
- Remove existing bulkhead support frames
- Fabricate and install operating platform and stairway access
- Complete electrical service upgrades
- Install slide gates and slide gate operators
- Dewater powerhouse and complete structural modifications (one bay at a time)
- Reconfigure debris boom and remove trash rack

Due to the long lead times that are anticipated for the slide gates and slide gate operators, the proposed work will likely extend into 2022. The RKLD intends to give contractors the option of a phased construction approach, as we anticipate that this may result in more favorable bids. However, the construction contract will state that all work shall be completed by October 31, 2022.

# 2. Structural

## A. Powerhouse removals

### (1) Description

The proposed work includes removal of portions of the existing powerhouse structure. These removals include:

- Steel bulkhead support frame at powerhouse intake deck
- Both reinforced concrete generator bases
- Both steel head plates that currently cover the holes in the generator floor
- Wicket gates and associated machinery
- Portions of the reinforced concrete turbine pit floors, including both turbine bases
- Steel draft tube cones
- Portions of the reinforced concrete intake header beams that serve as the sealing surface along the top of the existing upstream bulkheads

### (2) Design considerations

The reinforced concrete turbine pit floors provide lateral support to the powerhouse abutment walls and the divider wall between bays. As a result, we have designed structural reinforcements to maintain sufficient lateral support for these walls. These components, which include the edge beams and closure walls shown in the drawings, must be constructed and allowed to reach design strength prior to initiating removal of portions of the turbine pit floors. The design of these components is discussed in Section 2.B.

## B. Powerhouse reinforcements

### (1) Description

The proposed work includes construction of cast-in-place, reinforced concrete structural reinforcing members within both turbine pit bays. Components include:

- Two edge beams in each turbine pit bay along the abutment wall and divider wall
- One closure wall toward the back of each turbine pit bay, spanning east to west between the abutment and divider wall

We will also be placing a sloped concrete “spillway” in both draft tube areas. This component will be composed of mass concrete with limited reinforcement at the face to control temperature and shrinkage cracking. The purpose of this component is to improve hydraulics in the powerhouse sluiceways created through the proposed modifications and to reduce the likelihood of negative pressures and vibrations during flow releases. As this is a non-structural element, design calculations have not been provided.

### (2) Design criteria

A portion of the existing powerhouse turbine pit floor will be removed to increase the discharge capacity of the powerhouse. It was assumed that this floor acts as a lateral brace between the abutment and pier walls. As a result, new reinforced concrete edge beams were designed along the abutment and pier walls to replace the lateral restraint lost due to partial removal of the turbine pit floor. Additionally, a new reinforced concrete closure wall was designed along the downstream edge of the turbine pit floor opening. The new closure wall will act as a column and provide structural support to the downstream ends of the new edge beams, as well as providing better hydraulics through the powerhouse by isolating areas where water may have a tendency to stagnate. The design assumes that removal of the turbine pit floor will not be initiated until construction of the edge beams and closure wall have been completed. The design was completed in accordance with the following codes and references:

American Concrete Institute, *Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary*, 2014.

American Society of Civil Engineers, *Minimum Design Loads for Buildings and Other Structures (ASCE 7-10)*, 2010.

American Institute of Steel Construction, *Steel Construction Manual*, Fifteenth Edition, 2017.

The following historical documentation was also referenced:

Mead & Hunt, Inc., Indianford Dam and Powerhouse Repair Drawings, 2004.

Wisconsin Power and Light Company, Original Construction Drawings, 1928.

### (3) Design methodology

The following is a summary of the methodology that was utilized in the design of the new reinforced concrete edge beams and closure walls:

#### Edge beams

The edge beams were designed to provide lateral support to the existing pier and abutment walls to account for removal of a portion of the turbine pit floor. The edge beams were designed as simply-supported members with horizontal restraint provided at both ends. At the upstream end, the edge beam will be supported by a group of post-installed concrete anchors embedded into the remaining turbine pit floor. At the downstream end, the edge beam will be supported by the new closure wall, which acts as a column between the east and west edge beams. The design assumes one bay is dewatered, with lateral earth pressure and outrigger loads acting on the abutment wall.

**Closure walls**

The closure wall was designed to act as a column that resists end reaction loads from the downstream ends of the edge beams. The closure was also designed to simultaneously resist horizontal hydrostatic pressure due to the water inside the turbine pit.

**(4) Load cases**

The relevant load cases that were evaluated as part of this design are presented in Table 2-1.

**Table 2-1. Load Cases – Edge Beam and Closure Wall**

| Load Case | Description[2] | Edge Beam | Closure Wall |
|---|---|---|---|
| I | Basic Combination 1 | N/A | 1.4D |
| II | Basic Combination 2 | 1.2D + 1.6L | 1.2D + 1.6L |

**(5) Assumptions/Methodology**

The following is a summary of the assumptions made as a part of this design:

- The compressive strength ($f_c$') of the existing concrete was assumed to be 2,500 psi.
- The yield strength ($f_y$) of the existing steel reinforcement was assumed to be 33 ksi.
- The compressive strength ($f_c$') of the new concrete was assumed to be 4,500 psi.
- The yield strength ($f_y$) of the new steel reinforcement was assumed to be 60 ksi.
- One powerhouse bay will be dewatered at a time during construction; therefore, completion of the new edge beams and downstream closure wall are required before turbine pit floor removals can be initiated.
- Outrigger loads from a 75-ton crane were assumed to be present adjacent to the abutment wall while the powerhouse bay is dewatered.

**(6) Results**

A summary of the demand-to-capacity ratios (unity factors) for the edge beam and closure wall designs are provided in Table 2-2.

[2] Loading combinations referenced from Section 2.3.1 of ASCE 7.

**Table 2-2. Edge Beam and Closure Wall Design Results**

| Structural Component | Unity Factor |
|---|---|
| **Edge Beams** | |
| Flexure | 0.05 |
| Shear | 0.53 |
| Anchorage | 0.62 |
| **Closure Walls** | |
| Combined Axial & Flexure | 0.52 |
| Anchorage | 0.25 |

The unity factors summarized in Table 2-2 are based on the maximum force results for the load combinations considered. All unity factors are less than 1.0. As a result, the edge beam and closure wall designs are considered to be adequate based on the design criteria used.

Supporting documentation for the edge beam and closure wall designs are provided in Appendices B1 and B2, respectively.

## C. Slide gates

The slide gates will be designed by the contractor's fabricator. The Technical Specifications identify workmanship, fabrication, and performance requirements for the slide gates (see Appendix D). Relevant sections include:

- 05 50 14 (Structural Metal Fabrications)
- 09 97 02 (Painting: Hydraulic Structures)
- 35 20 16 (Slide Gates)

Major design requirements for the slide gates include, but are not limited to:

- Height = 12 feet
- Width = 9 feet 11-1/4 inches
- Depth = 9-1/2 inches
- Design hydraulic head = 12 feet
- Design impact load = 10,000 pounds
- Maximum dry weight = 8,000 pounds
- Maximum mid-span deflection under design hydraulic head = ¼ inch

## D. Slide gate operator platform

### (1) Description

The slide gate operator platform will support the gate actuators and provide access to the actuators for gate operation.

**(2) Design criteria**

The following is a summary of the design criteria that serves as a basis for this design:

American Concrete Institute, *Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary*, 2014.

American Institute of Steel Construction, *Steel Construction Manual*, Fifteenth Edition, 2017.

American Society of Civil Engineers, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE 7-16)*, 2017.

Wisconsin Department of Transportation, *WisDOT Bridge Manual*, July 2019.

**(3) Design methodology**

The following is a summary of the methodology that was utilized in this design:

- The structural framing for the gate operator platform was designed using the RISA-3D, which is a structural analysis software program.

- Hand calculations were performed in the design of the structural connections and anchorages.

**(4) Loads**

The following loads were considered in the design of the gate operator platform:

- The maximum anticipated ultimate load due to gate operation was determined to be 29,500 pounds. This load is the aggregate of the anticipated gate weight, a nominal gravity load due to silt/debris build-up, and the estimated frictional sliding resistance between the gate's bearing strips and the steel-lined gate slots. Supporting calculations for the gate operating load are provided in Appendix B3.

- Live load = 40 psf

- Snow load = 18.2 psf (based on $p_g$ = 30 psf, $C_e$ = 0.9, $C_t$ = 1.2, and $I_s$ = 0.8)

**(5) Assumptions**

The following is a summary of the assumptions made as a part of this design:

- The compressive strength ($f_c$') of the existing concrete was assumed to be 2,500 psi.

- All new plate steel, structural channels, and angles will conform to ASTM A36 ($F_y$ = 36 ksi, $F_u$ = 58 ksi).

- All new wide-flange and structural tee sections will conform to ASTM A992 ($F_y$ = 50 ksi, $F_u$ = 65 ksi).

- All high-strength bolts will conform to ASTM F3125, Grade A325.

- All welds will be made using E70XX electrodes (70 ksi).

- The dry weight of each slide gate was assumed to be 8,000 pounds.

- Maximum hydraulic head on the slide gate during operation was assumed to be 12 feet.

- The gravity load due to silt/debris build-up on the horizontal surfaces of the gate was assumed to weigh 600 pounds.

- The static coefficient of friction between the steel-lined gate slots and the ultra-high molecular weight polyethylene (UHMW-PE) bearing strips was assumed to be 0.30.

**(6) Results**

All members and connections have acceptable factors of safety in accordance with design criteria referenced in this section. The following supporting design documentation is provided in the appendices:

- The RISA-3D model summary and results for the gate operator structure frame design are provided in Appendix B4.

- Column base plate and anchorage design computations are provided in Appendix B5.

- Column (top) bearing plate and bolted connection design computations are provided in Appendix B6.

- Design computations for the connection of the beams to the brick masonry powerhouse wall are provided in Appendix B7.

- Design computations for the connection of the beams supporting the slide gate operators are provided in Appendix B8.

- Platform grating design computations are provided in Appendix B9.

## E. Generator floor hole covers

### (1) Description

The existing steel head plates will be removed because they may extend down into the flow with water sluicing through the powerhouse. New steel-framed hole covers with grating were designed to eliminate the potential for someone falling into the hole in the generator floor.

### (2) Design criteria

The following is a summary of the design criteria that serves as a basis for this design:

American Concrete Institute, *Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary*, 2014.

American Society of Civil Engineers, *Minimum Design Loads for Buildings and Other Structures (ASCE 7-10)*, 2010.

American Institute of Steel Construction, *Steel Construction Manual*, Fifteenth Edition, 2017.

### (3) Design methodology

The steel beams were designed for the tributary loads on the beam with the largest unsupported span using the ENERCALC structural analysis software. The number and spacing of the beams were determined based on geometry and the span that the grating used on the gate operating platform could withstand. This was to allow for the use of the same type of grating throughout.

### (4) Load cases

The controlling load case was 1.2D + 1.6L.

### (5) Assumptions/Methodology

The following is a summary of the assumptions made as a part of this design:

- All new plate steel will conform to ASTM A36 ($F_y$ = 36 ksi, $F_u$ = 58 ksi).
- All new wide-flange sections will conform to ASTM A992 ($F_y$ = 50 ksi, $F_u$ = 65 ksi).
- The compressive strength ($f_c$') of the existing concrete was assumed to be 2,500 psi.
- The beams were designed for a 9.4 psf grating dead load applied over a 2.75-foot tributary width and a 15 pounds per linear foot (plf) beam self-weight.
- The beams were designed for a 125 psf live load and a 900-pound concentrated live load acting at the midpoint. Both live loads were conservatively assumed to act concurrently.
- The beams were assumed to be simply supported.

#### (6) Results

The steel beams that support the grating have unity factors of 0.39 and 0.11 for flexure and shear, respectively. The grating is sufficient for the loading and unsupported span length. Supporting documentation for the generator floor hole cover design is provided in Appendix B10.

## F. Concrete in-fill areas at exterior steps

#### (1) Description

The columns at the far east and west ends of the slide gate operator platform are located such that a portion of the base plate and two anchors fall where there are currently concrete steps. Both the east and west steps will be filled in with reinforced concrete to provide sufficient support for the columns and anchorages.

#### (2) Design criteria

The reinforced concrete modifications at the east and west concrete steps will allow structural support for the exterior platform columns. The following is a summary of the design criteria that serves as a basis for this design:

American Concrete Institute, *Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary*, 2014.

Wisconsin Power and Light Company, Original Construction Drawings, 1928.

#### (3) Design methodology

After review of the original construction drawings, it was concluded that the existing east and west concrete stairs are an integral part of the powerhouse abutments. As a result, it was concluded that reinforced concrete structural modifications to the stairs could be made to allow support from the new platform structure to the existing abutment walls and foundation. The reinforced concrete design for the stair modification was completed as a column to resist both axial and lateral loads.

#### (4) Load cases

The relevant load cases that were evaluated as part of this design were determined based on the operator platform design, which include the following:

1. 1.4D
2. 1. 2D+1.6L+0.5S
3. 1.2D+1.6S+1.0L
4. 1.2D+1.6L+0.5S+1.2GDL+1.6G+1.4F
5. 1.2D1.6S+1.0L+1.2GDL+1.6G+1.4F
6. GDL+G+F

Where,

D = Dead Load (member weight)
L = Live Load (pedestrian load)
S = Snow Load
GDL = Gate Dead Load
G = Gate Gravity Load
F = Gate Friction Load

**(5) Assumptions/Methodology**

The following is a summary of the assumptions made as a part of this design:

- The compressive strength ($f_c$') of the existing concrete was assumed to be 2,500 psi.
- The compressive strength ($f_c$') of the new concrete was assumed to be 4,500 psi.
- The yield strength ($f_y$) of the new steel reinforcement was assumed to be 60 ksi.
- The factored loads on the columns were conservatively orientated to produce the maximum force effect on the anchors for post-installed anchor design.
- The new platform columns were assumed to bear only on the new concrete. The existing 1-foot wide concrete between the stairs and powerhouse intake was conservatively neglected.

**(6) Results**

A summary of the unity factors for the reinforced concrete structural modifications to the stairs are provided below.

| | **Demand, $R_u$** | **Resistance, $\phi R_n$** | **Ratio** |
|---|---|---|---|
| Axial & Flexure | -- | -- | 0.14 |
| Side Anchors | 2.27 | 7.19 | 0.32 |
| Anchorage | 13.13 | 15.78 | 0.84 |

The results shown above are the maximum force results for controlling Load Combination 4. All unity factors are less than 1.0. As a result, the design for the reinforced concrete structural stair modification is considered to be structurally adequate based on the design criteria used. Supporting documentation for the concrete step in-fill design is provided in Appendix B11.

## G. Powerhouse stability

### (1) Description

The proposed modifications to the powerhouse include removal of existing concrete and placement of new concrete. We have revisited the stability analysis for the powerhouse to assess the effect of these changes on the global stability of the structure.

### (2) Analysis criteria

The following is a summary of the criteria that served as a basis for this analysis:

- The stability analysis was performed in accordance with the requirements of the Wisconsin Department of Natural Resources (WDNR) Administrative Code NR 333.

- During periods of high flow, tailwater levels rise much faster than headwater levels and the head differential at the Project approaches zero. Using engineering judgement, the flood loading condition will not be the controlling load case. As a result, this load case was not considered.

- The Project is located in a region of low seismicity (Zone 1). As a result, earthquake loading was not considered.

Federal Energy Regulatory Commission, *Engineering Guidelines for the Evaluation of Hydropower Projects, Chapter 3*, *Gravity Dams*, October 2002.

### (3) Analysis methodology

The powerhouse stability analysis performed by Mead & Hunt in 2003 was used as a basis for this analysis. In the 2003 analysis, the powerhouse was assessed in its entirety by segmenting the structure into shapes to allow for simple volume and center of gravity calculations. Headwater, tailwater, uplift, and hydrostatic forces within the powerhouse were also calculated.

To analyze the stability of the structure for the proposed modifications, the existing analysis was revised to reflect proposed conditions. A 3-dimensional model of the powerhouse was developed using the SolidWorks modeling program and locations of concrete removal and addition were identified. The model was used to compute volumes, centers of gravity, and appropriate distances to the assumed overturning point. Volumes of water within the powerhouse that would be displaced by concrete or otherwise removed due to the proposed modifications were also quantified. Headwater, tailwater, and uplift loads were not changed from the 2003 analysis.

The stability of the powerhouse was analyzed for sliding and overturning using the gravity method. The summation of loads and moments from the 2003 report were combined with loads associated with the proposed modifications to yield new results.

**(4) Load cases**

The relevant load cases that were evaluated as part of this analysis are presented in Table 2-3.

**Table 2-3. Load Cases – Powerhouse**

| Load Case | Description | Headwater Level | Tailwater Level |
|---|---|---|---|
| I | Normal Operating Conditions | 775.3 | 770.6 |
| IIA | Normal Conditions with Ice | 775.3 | 770.6 |
| IV | Normal Conditions with One Unit Dewatered | 775.3 | 770.6 |

**(5) Assumptions**

The following is a summary of the assumptions made as part of this analysis:

- Loads calculated in the stability analysis performed by Mead & Hunt in 2003 were adopted into this analysis.
- Loads were added or removed from the previous analysis to represent planned modifications of the powerhouse.
- The assumed foundation friction angle was referenced from the 2003 stability analysis. This value was based on 2002 borings and Hunt, R., "Geotechnical Engineering Investigations Manual", McGraw-Hill, 1984.
- Uplift pressures were calculated using a weighted creep distribution and were not changed from the 2003 stability analysis.
- The turbine base concrete was not considered in the 2003 stability analysis. However, removal of the turbine base concrete weight was included in our analysis. This is a conservative assumption.

**(6) Results**

Results of the analysis show that the proposed modifications to the powerhouse, which add more concrete than it removes, increase the stability of the structure with respect to sliding and have a negligible effect on overturning stability. The revised stability analysis results are summarized in Table 2-4. Supporting documentation is provided in Appendix B12.

**Table 2-4. Stability Analysis Results – Powerhouse**

| | Case I (Normal) | Case IIA (Ice) | Case IV (Dewatered) |
|---|---|---|---|
| Headwater Level | 775.3 | 775.3 | 775.3 |
| Tailwater Level | 770.6 | 770.6 | 770.6 |

**Table 2-4. Stability Analysis Results – Powerhouse**

| | Case I (Normal) | Case IIA (Ice) | Case IV (Dewatered) |
|---|---|---|---|
| **Stability in Overturning (Modified Powerhouse)** | | | |
| Resultant as % Base (Without Uplift) | 42% | 41% | 42% |
| Acceptable Range = 33.3% to 66.7% | | | |
| Resultant as % Base (With Uplift) | 39% | 37% | 39% |
| Acceptable Range = 25% to 75% | | | |
| Maximum Net Base Pressure (ksf) | 1.90 | 2.04 | 1.66 |
| **Stability in Overturning (Existing Powerhouse)** | | | |
| Resultant as % Base (Without Uplift) | 42% | 41% | 42% |
| Acceptable Range = 33.3% to 66.7% | | | |
| Resultant as % Base (With Uplift) | 38% | 36% | 38% |
| Acceptable Range = 25% to 75% | | | |
| Maximum Net Base Pressure (ksf) | 1.9 | 2.04 | 1.58 |
| **Stability in Sliding (Modified Powerhouse)** | | | |
| Shear Friction Factor (SFF) | 5.16 | 3.30 | 4.54 |
| Minimum FERC-Recommended SFF | 1.5 | 1.5 | 1.5 |
| **Stability in Sliding (Existing Powerhouse)** | | | |
| Shear Friction Factor (SFF) | 5.0 | 3.2 | 4.17 |
| Minimum FERC-Recommended SFF | 1.5 | 1.5 | 1.5 |

# 3. Hydraulics

## A. Powerhouse sluiceway geometry

Measures were taken to enhance the hydraulic performance of the modifications made to the powerhouse. It is worth noting that computational fluid dynamics (CFD) modeling was not conducted to analyze the modified powerhouse configuration. Because CFD modeling was not used, a conservative design approach was taken to minimize the potential for adverse hydraulic effects such as the formation of eddy currents, vortices, negative pressures and scour/erosion of the concrete. Curvilinear features were implemented where possible to minimize these affects. Also, a portion of the draft tube bays will be filled with concrete to form a sloped spillway to guide flows through the opening and minimize the chances of negative pressures and vibrations occurring at the cutout through the turbine pit floor and prevent eddies from forming that could cause scour of the concrete draft tube floor.

## B. Estimated discharge capacity and approach velocity

Hydraulic calculations were performed to estimate the discharge capacity of the powerhouse and approach velocities upstream of the powerhouse intake for the normal pool condition and for flood conditions. Basic hydraulic principles were used in place of CFD modeling to estimate discharges and velocities for a range of conditions. The calculations are provided in Appendix C.

### (1) Normal pool condition

For the normal pool condition, the calculations assumed a headwater elevation of 775.3 feet and a tailwater elevation of 770.6 feet. The maximum discharge will occur during normal pool conditions due to the lesser degree of submergence when compared to the flood conditions. The hydraulic calculations for this condition were based on the following assumptions about discharge characteristics through the structure:

- Orifice flow through the intake opening to the powerhouse
- Submerged weir flow through the cutout/opening in the turbine pit floor
- Submerged orifice flow through the draft tube opening

The calculations show that the cutout/opening in the turbine pit floor and the draft tube act in tandem as the hydraulic control through the structure for normal pool conditions. Assuming all slide gates at the intake are fully open, the resulting total discharge through the powerhouse during normal pool conditions is approximately 4,200 cubic feet per second (cfs). Based on this discharge, a velocity of 6.3 feet/second was computed for just upstream of the powerhouse intake.

### (2) Flood conditions

Discharges and approach velocities upstream of the powerhouse were computed for both the 10-year flood event, and the 100-year flood event. The assumed headwater and tailwater elevations for both flood events were obtained from the 2015 *Flood Insurance Study* for Rock County and are provided in Table 3-1 below.

**Table 3-1. Headwater and Tailwater Elevations for Flood Conditions**

| Flood Condition | Headwater El. (ft, NGVD 29) | Tailwater El. (ft, NGVD 29) |
|---|---|---|
| 10-year Flood | 779.1 | 778.1 |
| 100-year Flood | 782.2 | 781.9 |

For both flood conditions analyzed, the passageway through the powerhouse would be completely submerged. Therefore, it was assumed that the intake opening would act as the hydraulic control since it creates the smallest opening through the structure and that it would be acting as a submerged orifice. The resulting discharges and approach velocities upstream of the powerhouse intake are provided in Table 3-2 below assuming all slide gates at the intake are fully open:

**Table 3-2. Computed Powerhouse Discharges and Approach Velocities for Flood Conditions**

| Flood Condition | Discharge (cfs) | Approach Velocity (ft/s) |
|---|---|---|
| 10-year Flood | 2,560 | 2.8 |
| 100-year Flood | 1,400 | 1.2 |

# 4. Mechanical

Each of the six slide gates will be raised and lowered by dedicated, electrically-powered hoists. Due to the hydraulic forces and the anticipated coefficient of friction between the gate leaf bearing strips and the steel-lined slots, the slide gate operators will need to be capable of pushing the gates downward in addition to lifting the gates upward. This ruled out a cable hoist system.

The slide gate operators will consist of dual ACME jack-screw units attached to each gate. The jack-screw units will be driven by a common shaft that is rotated by a single actuator gear motor drive. The ACME screws will be attached to the top of the gate near each side of the gate leaf. The jack-screw units will be mounted on stands that are bolted to the gate operator platform framework. If properly aligned, the dual action of the ACME screws will ensure that the gate travels straight upward and downward.

The single gear motor actuator drive unit will be located midway between the ACME jack-screw units on a stand that is bolted to the gate operator platform framework. Shafts will run from the actuator unit out to each jack-screw unit. The actuator gear motor drive will be a self-contained unit that houses the motor, required gearing, limit switches, position sensing, over-torque limiters, and all control and power wiring.

The slide gate operators will be procured by the contractor. The Technical Specifications identify performance requirements for the slide gate operators (see Appendix D). Relevant sections include:

- 09 97 13 [Exterior Coating of Steel Structures (Unsubmerged)]
- 35 01 41 (Slide Gate Operators)
- 35 20 20 (Electrical Equipment for Gate Hoist)

# 5. Electrical

## A. Existing conditions

The existing overhead service at the Project site is sized for 100A at 240/120V. A utility riser resides approximately 25 feet from the powerhouse. Power is routed from the riser underground to a hot sequence meter mounted outside the powerhouse. Load side conductors off the meter travel in a conduit on the exterior of the building and enter the structure into a 100A recessed panel. Existing loads include general purpose receptacles, lighting, welding receptacles, and two out-of-service motors that were once used to operate components in the powerhouse that have been removed.

## B. Service upgrades

Six electrically-powered slide gate operators with integral controls will be installed as part of this project. Exterior light fixtures and maintenance receptacles will also be installed on the gate operator structure to provide for safe working conditions and ease of operations and maintenance. The existing service will need to be upgraded to a 250A 480V service.

### (1) Service replacement considerations

It is anticipated that the slide gate operator motors will be sized at 5 horsepower. Using National Electrical Code (NEC) Tables 430-148 and 430-150, we assumed the motors will draw 28A at full operation. Keeping in mind starting in-rush and an assumed minimum circuit amperage (MCA) of 35A for each motor, an additional 128A of new motor load is required to be added to the existing service. The motors will be provided with full-voltage or across-the-line starters. There is not an electronic means to prevent all motors from being operated at once. Thus, we have concluded that the existing service is too small.

### (2) Service replacement components

The existing riser will be reused and a new step-down transformer will be provided by the utility company at the time of construction. A 150 KVA 3P-4W with a 480Y/277V secondary transformer was selected from a list of standard sizes that the utility company offers to ensure that the new system will provide sufficient energy for gate operation. Spare capacity has been provided to accommodate future projects, growth, or additions to the system. The utility company will provide transformer, medium voltage work, secondary conductors, CT's, and a meter for the service upgrade. The electrical contractor (EC) will provide a new CT cabinet, meter socket, transformer secondary raceway, load-side raceway out of the CT cabinet routed to a new 480V lighting and appliance (LA) panel with a 250A bus, and a 200A main circuit breaker (MCB). The EC will also provide a new 15 KVA 480:240/120V 1P-2W step-down transformer to re-serve the existing 240V panel. The existing panel will be removed and replaced with a new 240V panel with a 100A bus. All raceway disconnects and wiring for both line voltage and low voltage will be provided by the EC for full operation of the slide gate operators.

All equipment installed outdoors will be rated as NEMA 4X and NEMA 3R for CT cabinet and meter socket. Equipment installed indoors will be rated as NEMA 12 to prevent dirt and dust buildup in the equipment.

# 6. Estimate of Probable Cost

Our estimate of probable cost for the work associated with this project is $2,440,000. This includes engineering design and construction support services, preparation of the grant application, and anticipated construction costs. A detailed cost estimate is provided in Appendix E.

# Appendix A. Drawings