MODEL INVESTIGATION OF SEDIMENT AND HYDRAULIC CONCERNS FOR MIDAMERICAN ENERGY COMPANY’S NEW GEORGE NEAL NORTH WATER INTAKE STRUCTURES

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ABSTRACT

Hydraulic model investigations were conducted for MidAmerican Energy Company’s George Neal North Station to improve newly designed circulating-water pump-intake structures. Both sediment and hydraulic models were built at undistorted geometrical scales of 1:30 and 1:12, respectively. The sediment model indicated that a series of flow-turning vanes would be needed to reduce the sediment intrusion into the intake bays. A variety of pump-ump modifications were also developed using the intake model, including floor and backwall splitters, sidewall floor-corner fillets, perforated plates to improve pump-approach flow distributions, and skimmer walls for free-surface vortex suppression.

ACKNOWLEDGMENTS

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I. INTRODUCTION

Electrical power generating stations produce a large quantity of thermal energy after generating electrical power. In the majority of cases, this residual thermal energy cannot, or at least not permanently, be used in a practical sense. Since this heat still must be dissipated, cooling systems are required. A cooling system for a power plant needs a large quantity of coolant, which is withdrawn from a river, a lake, or the sea with large capacity pumps. The coolant pumps are integrated in an intake building, providing the optimal water transfer from the water source to the cooling cycle. However, a proper matching of the intake facility to the characteristics of the water source as well as to the characteristics of the pump intakes can be a challenging task for the successful operation of the power plant.

For example, undesirable sedimentation in front of the intake building may obstruct the water inflow. Limitation on the electrical output of the power plant, as well as expensive dredging at regular intervals, could be a result of the intake blockage by sediment. Inside the intake chambers, vortices in the vicinity of the pump bell can damage the pump or reduce its effectiveness.

Due to the many parameters involved, the water flow surrounding the intake is so complex that, in spite of ample engineering and empirical knowledge, it is impossible to preclude all the named problems by exclusive use of theoretical considerations during the design phase of a new intake. Therefore, it is common to conduct experimental model tests to achieve satisfactory pump-operating conditions.

Even though it is possible to adapt a new intake design to an already built intake facility, it is preferable to carry out practical model tests during the design phase before construction.
II. STUDY OBJECTIVES

A new fossil power plant circulating-water pump intake is planned on the left bank of the Missouri River near Sioux City, Iowa for “George Neal North” station by MidAmerican Energy Company. The water intake structure was designed by Burns & McDonnell in Kansas City, Missouri to supply condenser cooling water. This intake structure is planned to be located on the Iowa side of the Missouri River and designed for accommodating six vertical pumps in six separate pump bays with water discharge capacities ranging from 37,000 gpm (82 cfs) up to 145,000 gpm (323 cfs) for a total combined discharge of 495,000 gpm (1,102 cfs).

Due to past siltation problems experienced at other facilities along the Missouri River, sediment-related problems are anticipated at the George Neal North (GNN) station as well. Potential problems related to vortices and non-uniform flow distribution inside the intake structure also exist at large capacity pump intakes. However, most of the newly designed intake structures, which are investigated to solve the aforementioned hydraulic problems, can be optimized without major changes. For example, scour-promoting flow turning vanes in front of intake buildings are able to prevent sedimentation. Integration of splitters, baffle columns and perforated plates into intake bays can produce a more uniform flow and suppress formation of free-surface and sub-surface vortices.

The current research project consisted of two parts. The first one was the experimental investigation of the planned intake structure under as-designed conditions with regard to sedimentation, and the second was the investigation of hydraulic performance of the designed intake structure related to vortices and uneven velocity distributions within the intake bays. Two undistorted scaled models were built and run for these purposes. One was a sediment model built at an undistorted geometrical model scale of 1 to 30 with a moveable sand-bed and the other was an intake model built at an undistorted geometrical model scale of 1 to 12.

III. SIMILARILTY LAWS

The two models were built at undistorted geometric scales. This means that the ratio of all the lengths in every model to their corresponding lengths in the prototype was equal:
where

\[ L_r = \frac{L_m}{L_p} \]  \hspace{1cm} (3-1)

\( L_r \): Length ratio

\( L_m \): Model length

\( L_p \): Corresponding prototype length

Due to the free-surface flow in both models, it was necessary to include the gravitational influence into the calculations of the model discharges, velocities, and times. Therefore, the Froude number similarity between model and prototype discharges was used.

\[ F_r = \frac{F_m}{F_p} = 1 \]  \hspace{1cm} (3-2)

\( F_r \): Froude number ratio

\( F_m \): Froude number of the model

\( F_p \): Froude number in the prototype

where

\[ F = \frac{V}{(gL)^{1/2}} \]  \hspace{1cm} (3-3)

\( V \): Characteristic flow velocity

\( g \): Gravitational constant

\( L \): Characteristic length

With equations (3-2) and (3-3) the velocity and the time ratios are given by

\[ V_r = \frac{V_m}{V_p} = L_r^{1/2} \]  \hspace{1cm} (3-4)

\[ T_r = L_r^{1/2} \]  \hspace{1cm} (3-5)

The discharge ratio follows from (3-4) as

\[ Q_r = V_r A_r = L_r^{1/2} L_r^2 = L_r^{3/2} \]  \hspace{1cm} (3-6)
IV. SEDIMENT MODEL

The sediment model was first built and run to determine whether or not the sedimentation problems previously experienced at other intake facilities along the Missouri River are likely to occur at the newly designed intake structure. If siltation problems were detected, the model was then to be used to study the effectiveness of installing scour promoting, submerged flow-turning vanes in front of the intake structure to avoid such problems. Therefore, the sediment model included a portion of the river surrounding the intake structure and the intake structure itself, but no internal devices, such as pump bells or flow straightening devices were included. Because the objective was to determine the long-term bed-form development, the model was run for weeks during each test phase and was stopped temporarily only for bed profile measurements.

4.1 Description of Sediment Model

The movable-bed, hydraulic sediment model was built at an undistorted geometrical scale of 1 to 30. It included an 8-ft wide and 27.5-ft long rectangular basin, six as-designed pump intake bays, and a scaled portion of the Missouri River. The river part was about 240-ft wide and 825-ft long in prototype dimensions.

The river flow was generated by a pump connected to a 7.5 hp pump withdrawing water through a 10-in. diameter pipe out of the tail box and re-introducing it through a diffuser pipe into the head box. The 10-in. pipe was contracted to form a venturi at one part to make discharge measurements with a two-tube manometer. The river stage was adjusted with a simple tailgate at the downstream end of the basin and by changing the water discharge circulating in the model. Figure 4.1 shows the main parts of the sediment model.

The river section was filled with mason sand whose median particle size was 0.34 mm. The size frequency distribution curve shows the exact percentage of particular grain sizes (see figure 4.2). No specific similitude laws were used to select this sand, because it was mainly chosen to get an active bed-load movement to study the sand deposition patterns and bed-elevation changes while transport rates were less important. The river shore was modeled with crushed quarry rocks whose median diameter was about 1 in.
The intake structure shown in figure 4.3 was built geometrically similar to the prototype and attached to a rectangular model basin. The water flow through the intake structure was withdrawn separately through each pump bay using pipes (1 to 2 in. in diameter) attached to the back walls. Each pipe was fitted with a valve and a calibrated orifice meter connected to a two-tube manometer for measuring the discharge. The four pipes from units 1 and 2 led into a bigger pipe connected to a pump powered by a 1 hp motor. The other two pipes from unit 3 led into a pipe connected to a second pump powered by another 1 hp motor. Water withdrawn was fed back to the model basin at the tail box (see photo 4.1).

4.2 Scaled Values for Sediment Model

The undistorted geometric scale ratio $L_r$ of the sediment model was 1 to 30. The water discharges from the intake bays were determined according to the Froude number similarity between model and prototype. With equation (3-6) it follows:
With the known prototype discharges, the model discharges in the different intake units can be calculated as follows:

$$Q_r = L_r^{5/2} = \frac{1}{4929.50}$$  \hspace{1cm} (4-1)
Figure 4.3: Plan and sections of model intake structures
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<th>Discharge Ratio $Q_r$</th>
<th>Model Discharge (cfs)</th>
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<tr>
<td>1</td>
<td>83.5</td>
<td>$\frac{1}{4929.5}$</td>
<td>0.0169</td>
</tr>
<tr>
<td>2</td>
<td>144.8</td>
<td>$\frac{1}{4929.5}$</td>
<td>0.0294</td>
</tr>
<tr>
<td>3</td>
<td>322.9</td>
<td>$\frac{1}{4929.5}$</td>
<td>0.0655</td>
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Table 4.1: Prototype and model discharges

With equation (3-4) the velocity ratio $V_r$ is:

$$V_r = \frac{V_m}{V_p} = L_r^{\frac{1}{2}} = \frac{1}{5.48}$$

(4-2)

and with equation (3-5) the time ratio $T_r$ is:

$$T_r = L_r^{\frac{1}{2}} = \frac{1}{5.48}$$

(4-3)

Different from the intake discharge ratios, the river flow ratio was increased slightly from the value determined by the Froude number similarity. This was needed to get a realistic bed-load movement. First, it was set to $1.25*Q_r$ during preliminary tests, where $Q_r$ is the river flow ratio at Froude number equivalence between model and prototype. It was then found that this discharge was too high and the ratio was subsequently reduced to $1.1*Q_r$.

The flow rate of the water in the model river section in prototype dimensions was

$$Q_p = W_p H_p V_p = 210 \text{ (ft)} \times 15 \text{ (ft)} \times 4 \text{ (ft/s)} = 12,600 \text{ cfs}$$

(4-4)

where

- $W_p$: Width of the river section
- $H_p$: Mean flow depth in the river section
- $V_p$: Mean flow velocity in the river section

The prototype mean river flow velocity $V_p$ in the model river section of the Missouri River was assumed to be $4 \frac{ft}{s}$, and the model flow cross-section area $A_m$ was $3\frac{1}{2}$ square ft. Therefore, the flow rate in model dimensions follows with equation (3-6) and Froude number equivalence to
\[ Q_m = V_m A_m = V_p L_r \frac{1}{2} A_m = 2.56 \text{cfs} \] (4-5)

The initial model river discharge was increased by a factor of 25\% as
\[ Q_{m,Froude=1.25} = 1.25 \times Q_m = 3.20 \text{cfs} \] (4-6)

The finally adopted river discharge was increased by a factor of 10\% as
\[ Q_{m,Froude=1.1} = 1.1 \times Q_m = 2.81 \text{cfs} \] (4-7)

### 4.3 Experimental Procedure

The experimental procedure consisted of

- Initial setting up the model (section 4.3.2.)
- Preliminary test (section 4.3.3.)
- Initial re-setting up the model (section 4.3.2.)
- Test case 1: As-designed conditions (section 4.3.4.)
- Test case 2: Flow turning vanes installed (1\textsuperscript{st} Layout) (section 4.3.5.)
- Test case 3: Flow turning vanes installed (2\textsuperscript{nd} Layout) (section 4.3.6.)
- Measurements of the riverbed elevation during the test (section 4.3.1.)

The first step in preparation to run the sediment model was filling the model basin with water approximately to the desired river water level. This had to be done very slowly in order not to destroy the existing bed-form pattern. Thereafter, air was purged off the river-circulation line by a vacuum attached to an air vent on top of the pipe. Then, air was drawn out of two tubes connected to the manometer. The main pump was then activated to start the river flow. Initially, the pump had to be run at a low speed, and it was then increased gradually to reach the desired discharge. Once the river discharge was established, air in the tubes connecting the orifice meters of the intake pipes to the manometer was purged off by a vacuum pump. The two pumps for the intake chamber discharges were then started. The calculated discharges (see table 4.1) were set using the valve integrated in each pipe. Finally, the river surface elevation was properly adjusted by adding water to or withdrawing it from the river basin and using the tailgate at the downstream end of the model.

The model was observed on a daily basis. Changes were looked for in the riverbed surface pattern and elevation, and checks were made for sand accumulation inside the intake bays. Furthermore, changes were also made for the water level and the
flow rates of the pipes. Bed elevation measurements were made (see section 4.3.1), and pictures were taken when there were major bed changes that were observed in the surrounding area of the intake structure. If a lot of sand was washed away near the river basin inlet at the head box, new mason sand was added to reestablish the mean riverbed elevation of 1,040 ft in full scale. The constantly decreasing water level (due to evaporation) was equalized from time to time by introducing fresh water into the model basin.

The procedure for stopping model operation and draining water from the model basin was to repeat the same steps for the starting of the model in their reverse order. First, the two intake pumps were stopped, and then the speed of the main pump was decreased slowly before it was shut off. Afterwards, the air vent of the main pipe was opened, letting the water flow out of the pipe. Finally, a sump pump was used to drain water out of the model basin.

4.3.1 Measurement Procedure

Bed elevation measurements were taken once or twice a week regularly and more often when significant changes in the sediment pattern near the intake structure were found. Before each measurement, the model operation was stopped and the water drained slowly as described in section 4.3. A point gauge was then used to measure the sand level at certain points in the surrounding area of the intake structure. To get comparable measurement data at all points, the point gauge was moved along a measurement rail placed perpendicular to the river-flow direction on the sidewalls of the model basin (see photo 4.2). The rigid aluminum rail was moved along the sidewalls. A reference point for all the measurements of the sand and water surface elevations was set at the top of the intake structure at 1,078 ft. Measurement data of the sand elevation were usually taken at 104 points in the vicinity of the intake bays. The points were distributed to eight sections (A through H) located perpendicular to the river flow direction at a distance of 60 ft in prototype dimensions between each row. Section C was located at the upstream end of the intake structure, sections A and B were upstream of C, and sections D to H downstream of C. Point 1 in every section was located 2½ ft in full scale away from the intake structure. Each section contained thirteen measurement points with intervals of 5 ft in prototype dimensions between them. The pattern of the measurement points is shown in figure 4.4.
4.3.2 Initial Setup

The mean riverbed elevation in the model river section was about 1040 ft and the low water level was about 1055 ft above the mean sea level. The initial model setup is shown in figure 4.5.

While the water elevation during the model operation was kept at the initial level, the riverbed was leveled to the initial setting only twice, once before the preliminary test and again before the tests under as-designed conditions.
4.3.3 Preliminary Tests

The preliminary test was mainly run to discover the proper river velocity which would move the model bed material as bed load and produce proper model-scale dunes. The river flow was initially increased by 25% from the Froude number equivalence. After 110 hours it was found that the river discharge was too high. Therefore, it was then reduced to $1.1^*Q_m$ and the model operation was started again. This river discharge produced desirable two-dimensional dunes, and it was kept for the remaining test period.

The preliminary tests also showed a large head loss between the head box and the rest of the model basin. Perforated plates initially installed in the head box were found to produce too much head loss and therefore, were replaced by plates with larger perforations. Furthermore, a lateral flow in the left bank (looking downstream) was recognized near the head box (see photo 4.3). This was caused by shore rocks of the model basin that blocked the flow, resulting in non-uniform inlet flow distributions in the upstream half of the model river flow. The left quarter of the perforated plates at the head box was then closed with a solid plate to avoid this problem.

4.3.4 Test 1: As-Designed Conditions

In test 1 the model was run for 476 model hours under as-designed conditions, which is about 109 days in prototype time. No flow turning devices were installed during this test.
Starting from a flat riverbed at EL 1040 ft, the bed elevation soon dropped at both the upstream and the downstream ends of the intake structure to about EL 1036 ft. These changes were due to the end piers that extended further into the river than other piers of the intake bays. Turbulent flows induced by flow separation produced these local scour holes. Photos 4.4 and 4.5 show those deep scour holes that were taken after 133 hrs of model operation.

In contrast to that, the bed elevation at the other part in front of the intake structure rose during the same operation period. Occasionally, crests of sand dunes reached EL 1044 ft which is the top of the stop log, resulting in sand entering the intake bays and depositing there. For example, the bed-elevation measurements after 314 operation hrs indicated that all the measured elevations near the intake structure were above EL 1042 ft and many dune crests reached higher than EL1044 ft (see photo 4.6 and figure 4.6).

It should be noted that the chart in figure 4.6 is distorted. The distance between each measurement row was 60 ft while the distance between each measurement point was only 5 ft.
Photo 4.4: Scour hole at the upstream end of the intake structure after 133 hrs of operation (flow from left to right)

Photo 4.5: Scour hole at the downstream end of the intake structure after 133 hrs of operation (flow from left to right)

Photo 4.6: Dune crests reaching the stop log crest (EL 1044 ft) during test 1 (flow from left to right)
4.3.5 Test 2: Flow Turning Vanes Installed (1st Layout)

After test 1, 15 flow turning vanes were installed in front of the intake structure. Each vane was 10 ft wide (in prototype dimensions) and installed at an angle of 20° to the river flow. The vanes were installed in two rows, the first one being 10 ft from the intake face and the second 15 ft. The distance between two vanes in each row was 36 ft and the top elevation of the vanes was at about EL 1046 ft. The exact vane layout is depicted in figure 4.7.

Figure 4.6: Bed contour observed near the intake structure after 314 hrs operation

Figure 4.7: First fifteen-vanes layout in test 2
The model was operated for 345 hrs with this vane layout, which is about 79 days in prototype time. During the first half of this test, the scour trench at the upstream end of the intake bays that already developed during test 1 was found to extend downstream along the whole intake structure. Measurements taken after 163 hrs showed that all bed elevation values of the grid points near the stop log were lower than EL 1040 ft, as can be seen in figure 4.8.

![Figure 4.8: Bed contour observed after 163 hrs operation in test 2](image-url)

However, this fully developed scour trench did not remain as a stable trench. The bed level near the stop log in front of the downstream bays (units 1 and 2 bays) increased gradually again during the second half of this test. Measurements after 345 hrs operation showed that bed elevations near the downstream intake bays were between EL 1041 ft and EL 1044 ft, as shown figure 4.9 and photo 4.7. Again, substantial sand was found in the downstream intake bays. In contrast, in the upstream sections of the intake structure the bed elevation was maintained below EL 1040 ft and no sediment was found in the intake bays.

Due to the satisfying scour trench formed in the upstream half and the undesirably high riverbed elevation in the downstream half of the intake structure, the pattern of the downstream vanes was altered slightly for the next test to achieve stronger scouring actions.
4.3.6 Test 3: Flow Turning Vanes Installed (2nd Layout)

For the final test run the distance between the seven downstream vanes was shortened from 36 ft to 30 ft, and two additional vanes were installed at the downstream end of the two rows (see figure 4.10).
The model was operated for 427 hrs in this layout, which is about 97 days in prototype time. With the new vane arrangement, a satisfactory scour trench developed over the downstream section. After 224 hrs of operation in test 3, all the measured elevations in front of the intake bays were below EL 1042 ft (see figure 4.11). After 427 hrs, all of these points were below EL 1040 ft (see figure 4.12). Figure 4.13 shows the measured bed elevations along the intake structure.

The scour trench developed and prevented the bed-load sediment from entering into the pump bays. Photo 4.8 shows the bed contour in front of the intake structure. It can be seen that the dune crests in the model basin were perpendicular to the stop log but between the flow turning vanes perpendicular to the vanes. That shows how the vanes turned the bed-load sediment away from the intake structure.
Figure 4.12: Bed contour observed near the intake structure after 427 hrs operation in test 3

Figure 4.13: Riverbed elevation along the stop log after 427 hrs operation in test 3
4.4 Conclusions and Recommendations

The following conclusions and recommendations were derived from the present 1:30-scaled undistorted moveable bed model.

It is recommended that two rows of scour-promoting, flow-turning vanes along the stop log of the intake structure be installed. One row consisting of nine vanes at a distance of 10 ft from the intake face, the other eight vanes 15 ft (see figure 4.10). The vanes should be 10 ft wide and the top elevation at EL 1046 ft.

These vanes should promote a scour-trench along the stop log of the intake structure while the stop log itself should act as a sediment barrier wall, thereby avoiding bed-load sediment entraining the pump bays. Suspended sediment entering the pump bays can never be stopped completely. However, the lower bed elevation at the stop log due to the scour trench, should reduce the suspended-sediment concentration because this concentration is usually highest at the riverbed and lower towards the free surface.

V. PUMP-INTAKE MODEL

The pump-intake model was first built and run to observe general flow behaviors inside the pump-intake bays with respect to uneven pump-approach velocity distributions, vortices, and prerotation in the pump column. If these problems were detected, the model was then used to investigate the effectiveness of installing corrective
measures, such as perforated plates, floor and backwall splitters, and corner fillets.

### 5.1 Classification of Intake Vortices

Since it was important to distinguish the strength of vortices appearing at pump intakes, the ARL (Alden Research Laboratory) classification system was used for this study. This classification is based on the identification of distinct flow features, which indicate the strength of vortices. The ARL classification system defines six types of free-surface vortices (shown in figure 5.1) and three types of subsurface vortices (shown in figure 5.2).

![Figure 5.1: ARL classification of free-surface vortices](image1)

![Figure 5.2: ARL classification of boundary-attached subsurface vortices](image2)
While free-surface vortices are usually easy to detect, dye injection was needed to observe subsurface vortices. Subsurface vortices are typically boundary-attached, such as those on the floor, backwall, and sidewalls, and extend towards the pump bell.

5.2 Criteria for Satisfactory Pump Operation

In numerous experimental studies at the IIHR, guidelines to obtain proper pump operating conditions have been developed. They include the following model criteria for satisfactory operation of prototype pump installations:

- No detectable boundary-attached vortices extending into the pump bell
- No free-surface vortices stronger than ARL-type 2
- No velocity along a concentric circle in the pump column that varies by more than 5% from the average velocity on the same circle
- No swirl angles inside the pump column exceeding 5°
- No persistent, large scale, “unsteadiness”, “waviness”, or turbulence in the intake-bay approach flow

5.3 Description of Intake Model

The hydraulic intake model was built at an undistorted geometrical scale of 1:12. It included a 12 ft wide and 45.4 ft long rectangular river basin (144 ft to 545 ft in full scale) and six as-designed pump intake bays. In contrast to the sediment model, the bed of the intake model was fixed and flat.

The model river flow was generated by a pump connected to a 75 hp motor which withdrew water out of a large sump through a 12 in. diameter pipe and circulated it through a diffuser pipe into the head box. The flow rate was measured by a electromagnetic flow meter attached to the main pipe. The water was fed back to the sump through the tail box at the end of the basin. The river stage was altered with a height adjustable tail gate built of scantlings at the downstream end of the basin. Figure 5.3 and photos 5.1 through 5.5 show the main parts of the intake model.
Figure 5.3: Pump-intake model basin and intake structures

The intake structure in the model was built geometrically similar to the prototype. The internal components of the intake chambers and the pump bells were reproduced in detail (see figure 5.4). The water flow through the six pump intakes was separately adjustable. Therefore, each of the pipes (from 3 to 5 in. in diameter) was fitted with two valves and a calibrated orifice meter connected to a two-tube manometer for measuring the discharge. The pipes of all pumps led into a manifold connected to a pump powered by a 10 hp motor. The withdrawn water was led through the tail box into the sump.

5.4 Scaled Values for Intake Model

The water discharges of the model river flow and the intake bays in the intake model were set to Froude number similarity between model and prototype. With equation 3-6 follows:

\[ Q_r = L_r^{5/2} = \frac{1}{498.83} \]  \hspace{1cm} (5-1)

\[ L_r = \frac{1}{12} \]

With the known prototype discharges, the model discharges in the different intake units can be calculated as shown in table 5.1.
5.3: Pump intake structure

Photo 5.4: Intake discharge pipes and pump

Photo 5.5a: Downstream view of model intake structure
Photo 5.5b: Units 2 and 3 pump intake bays

Photo 5.5c: Close-up view of unit-3 pump-intake bay
Figure 5.4: Plan and elevations of 1:12-scale pump-intake model
<table>
<thead>
<tr>
<th>Pump Intake Unit</th>
<th>Prototype Discharge (cfs)</th>
<th>Discharge Ratio ( Q_r )</th>
<th>Model Discharge (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>83.5</td>
<td>( \frac{1}{498.83} )</td>
<td>0.167</td>
</tr>
<tr>
<td>2</td>
<td>144.8</td>
<td>( \frac{1}{498.83} )</td>
<td>0.290</td>
</tr>
<tr>
<td>3</td>
<td>322.9</td>
<td>( \frac{1}{498.83} )</td>
<td>0.647</td>
</tr>
</tbody>
</table>

Table 5.1: Prototype and model discharges

The mean river flow velocity in prototype dimensions is about \( 4 \, \text{ft/s} \). With equation (3-4) the model river flow velocity was determined as follows:

\[
V_m = L_{\text{1/2}} \times 4 \frac{\text{ft}}{s} = 1.16 \frac{\text{ft}}{s}
\]  

The model river flow cross-section area was 10 ft wide and 0.833 ft wide:

\[
A_m = d_m \times w_m = 0.833 \, \text{ft} \times 10 \, \text{ft} = 8.33 \, \text{ft}^2
\]  

The model river flow discharge followed as:

\[
Q_m = V_m A_m = 9.62 \, \text{cfs}
\]  

5.5 Experimental Procedure and Test Results

In this experiment, six model pumps were observed separately. To make the description easier, the pumps were enumerated from #1 (upstream) to #6 (downstream). Four test cases were chosen. Table 5.2 shows the conditions of these cases and the type of data taken.

The first step to run the intake model was installing pitot tubes in those pump bells where velocity measurements were planned to be taken. The model basin and the intake structure were then flooded very slowly by opening the main valve at the head box and starting the operation of the main pump at low speed. Then the pump speed was increased gradually to the calculated model river discharge. Once the river flow was established, air trapped in the pump intake pipes as well as in tubing connecting the orifice meters of the intake pipes to the manometer was purged off by a vacuum pump. Individual pump intake flows were then started by actuating the 10-hp pump and the calculated discharges were set by adjusting the valves integrated in each pipe. Finally,
Test Case 1: As Designed Pump Sump Layout

<table>
<thead>
<tr>
<th>Pump</th>
<th>#1</th>
<th>#2</th>
<th>#3</th>
<th>#4</th>
<th>#5</th>
<th>#6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Operation</td>
<td>full speed</td>
<td>full speed</td>
<td>full speed</td>
<td>full speed</td>
<td>full speed</td>
<td>full speed</td>
</tr>
</tbody>
</table>

**Measured Data in Test Case 1:**

| Velocity | measured | measured | measured | measured | - | - |
| Swirl Angle | measured | measured | measured | measured | measured | measured |

Test Case 2: As Designed Pump Sump Layout

<table>
<thead>
<tr>
<th>Pump</th>
<th>#1</th>
<th>#2</th>
<th>#3</th>
<th>#4</th>
<th>#5</th>
<th>#6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Operation</td>
<td>shut off</td>
<td>full speed</td>
<td>shut off</td>
<td>full speed</td>
<td>full speed</td>
<td>full speed</td>
</tr>
</tbody>
</table>

**Measured Data in Test Case 2:**

| Velocity | - | measured | - | measured | - | - |
| Swirl Angle | - | measured | - | measured | measured | measured |

Test Case 3: Perforated Plates Installed

<table>
<thead>
<tr>
<th>Pump</th>
<th>#1</th>
<th>#2</th>
<th>#3</th>
<th>#4</th>
<th>#5</th>
<th>#6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Operation</td>
<td>full speed</td>
<td>full speed</td>
<td>full speed</td>
<td>full speed</td>
<td>full speed</td>
<td>full speed</td>
</tr>
</tbody>
</table>

**Measured Data in Test Case 3:**

| Velocity | - | measured | - | - | - | - |
| Swirl Angle | measured | measured | measured | measured | measured | measured |

Test Case 4: Perforated Plates, Splitters, and Fillets Installed

<table>
<thead>
<tr>
<th>Pump</th>
<th>#1</th>
<th>#2</th>
<th>#3</th>
<th>#4</th>
<th>#5</th>
<th>#6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Operation</td>
<td>full speed</td>
<td>full speed</td>
<td>full speed</td>
<td>full speed</td>
<td>full speed</td>
<td>full speed</td>
</tr>
</tbody>
</table>

**Measured Data in Test Case 4:**

| Velocity | measured | - | measured | - | - | - |
| Swirl Angle | measured | - | measured | - | measured | - |

Table 5.2: Summary table of test cases and measured data

The river level was properly adjusted to EL 1050 ft (low water level). Measurement as described in section 5.5.1. were then taken, and dye was injected to observe and classify vortex types.

The procedure for stopping model operation and draining water from the model basin was to repeat the same steps for the starting of the model in the reverse order. First, the intake pump was stopped and the air-vent valves were opened. Then the main river-basin water-supply pump was shut off. Finally, drainage valves in the model basin were opened.

**5.5.1 Measurement Procedure**

Measurements on model testing included local velocities at pump throats and prerotations in pump columns. Pitot tubes connected to a two-tube manometer were used to measure the local velocities. Velocity data were taken at 25 points in a cross-sectional area at the pump throat (the narrowest part of the pump bell) (see figure 5.5 for the three model pump bells) – 24 points were located along eight different angular positions on
three concentric circles and one in the center of the pump throat. The pitot tube positions (shown in figure 5.6) were calculated by using the equal area method. With this method the cross-section area was divided into three equal area circle rings and one circle in the middle. If the radius of the surrounding pump throat is $R$, the confining radii $c_i$ of the circle areas are:

\[
c_1 = \frac{R}{2} \quad (5.5)
\]

\[
c_2 = \frac{R}{\sqrt{2}} \quad (5.6)
\]

\[
c_3 = \frac{\sqrt{3}R}{2} \quad (5.7)
\]

The measurements were taken on concentric circles, as shown in figure 5.6, in the middle of the calculated circle areas:

\[
p_1 = 0R \quad \text{(measurement in the center)} \quad (5.8)
\]

\[
p_2 = \frac{c_1 + c_2}{2} = 0.604R \quad (5.9)
\]

\[
p_3 = \frac{c_2 + c_3}{2} = 0.787R \quad (5.10)
\]

\[
p_4 = \frac{c_3 + c_4}{2} = 0.933R \quad (5.11)
\]

Pitot tube measurements were taken in pump intake units 2 and 3. Therefore, each pump bell in these units had a hole to insert the probe. The 25 different positions were reached by altering the arm length of the tube that was inserted into the column and by rotating the entire pump column. The differential head between the static pressure and the stagnation pressure at each probe position was measured with the manometer and then converted to velocity data using a reduced form of the Bernoulli equation:

\[
v_p = \sqrt{\frac{2(p_0 - p_s)}{\rho}} = 8.023\sqrt{\Delta h_{0,s}} \quad (5.12)
\]

$v_p$: Velocity at the pitot tube

$p_0$: Static pressure

$p_s$: Stagnation pressure

$\rho$: Specific gravity of water

$\Delta h_{0,s}$: Difference head between static and stagnation pressure
Figure 5.5a: Unit 1 model pump bellmouth (ITT 54x42 WCA pump)

Figure 5.5b: Unit 2 model pump bellmouth (ITT 66x48 WCA pump)
Figure 5.5c: Unit 3 model pump bellmouth (ITT 96x66 WCA pump)

Figure 5.6: Velocity measurement positions (Nakato 2000)
Vortimeters installed in all the six pump columns were used to measure the prerotation of the flow inside the pump column. The vortimeters consisted of four zero-pitched blades that were supported by low-friction pivoted shafts. The blades extended over about 95% of the pump column diameter.

Prerotation data were taken by counting vortimeter revolutions for 15 minutes. Revolutions per minute were set in relation with the average flow velocity inside the pump column using formulas 5.13 to 5.15 to calculate the swirl angles. The swirl angle is the deviation between the pump-column flow direction from the line parallel to the pump-column wall in degrees. Photo 5.6 shows a pump column in intake unit 3 with pitot tube and vortimeter installed.

The swirl angle \( \theta \) is:

\[
\theta = \arctan \left( \frac{v_p}{v_a} \right)
\]  

(5.13)

The peripheral velocity is:

\[
v_p = \frac{\pi dn}{60}
\]

(5.14)

\( d \): outer pump-column diameter

\( n \): propeller revolutions per minute

The axial velocity is:

\[
v_a = \frac{Q_s}{A_c}
\]

(5.15)

\( Q_s \): model pump discharge

\( A_c \): cross-section area in column

5.5.2 Test Case 1: As-Designed Conditions

Velocity measurements in test run 1 showed very uniform velocity distributions inside of all four pump bells in intake units 2 and 3. The maximum deviation of a single measured velocity on a concentric circle from the average measured velocity on that circle was only 2% (see figures 5.7 through 5.10).
Prerotation measurements showed the maximum swirl angle of 3.1° in pump intake units 1 and 2, but the maximum swirl angle of 6.7° was observed in pump intake unit 3 which exceeded the criteria for satisfactory pump operation. The direction of the prerotation was clockwise in intake unit 1 and counterclockwise in units 2 and 3.

Dye injection into the model flow field showed free-surface and subsurface vortices of ARL - type 2 (see photos 5.7 to 5.8 for boundary-attached vortices). Occasionally, free-surface vortices of type 3 occurred in the left half of the intake bays in units 2 and 3. Flow stagnation areas were observed in the corners between floor, backwall, and the sidewalls. Large-scale flow rotations were observed around the pump columns.
Figure 5.7: Normalized velocity distribution in pump #1 (percentage deviation from the average velocity on the same circle is shown in parenthesis)

Figure 5.8: Velocity distribution in pump #2
Figure 5.9: Velocity distribution in pump #3

Figure 5.10: Velocity distribution in pump #4
5.5.3 Test Case 2: As-Designed Conditions with Two Adjacent Pumps Shut Off

Test 2 was run to investigate the flow behavior under different pump operating conditions in which pumps #1 and #3 were shut off.

The velocities measured in the active pumps were found to be more uniformly distributed as compared with the previous test runs. The maximum velocity deviation from the average of a circle was only 1% (see figures 5.11 and 5.12).

The swirl angles in intake units 2 and 3 also decreased considerably. In intake unit 3, it was 3.9°, and in unit 2, it was 0.7°. Swirl angles of the two small pumps in unit 1 were slightly increased to 3.8° and 1.1°, respectively. The orientation of the rotation was clockwise in intake unit 1 and counterclockwise in intake units 2 and 3.

Dye injection showed free-surface vortices and subsurface vortices of type 2 at all pumps. Occasionally free-surface vortices of type 3 were observed in the left half of the intake bays of unit 3. Flow stagnation areas were observed in the corners between floor, backwall, and the sidewalls. Large-scale flow rotations were observed around the pump columns.
Figure 5.11: Velocity distribution in pump #2

Figure 5.12: Velocity distribution in pump #4
5.5.4 Test Case 3: Perforated Plates Installed

Perforated plates are often used to reduce the prerotation in pump intakes if clogging problems are not anticipated. Since high prerotation was observed under as-designed conditions, perforated plates were installed in the stoplog slots downstream from the traveling screens in test case 3. Figure 5.14 shows the arrangement of holes on the perforated plates. The open area percentage was 32.6%.

Since the swirl angle measurement was most important in this test case and it was assumed that the perforated plates would not considerably change the flow velocity pattern inside the pump columns, velocity data were taken only at one pump to validate this presumption. Figure 5.14 shows the velocities in pump column #2. The maximum deviation of a velocity from the average velocity on the same concentric circle was 2%.

The measured swirl angles in intake units 2 and 3 were considerably lower than in the tests without the perforated plates. The maximum swirl angle of 3.9° was observed at pump #2. At pump #1 in the same intake unit, a swirl angle of 1.8° was measured. In unit 2, the angles were 2.3 and 1.6°, respectively. The orientation of the prerotation in these units was counterclockwise. The swirl angle at the upstream pump in intake unit 1 was decreased to 2.6°, but the angle at the downstream pump was slightly increased to 4°. The direction of the prerotation in this unit was counterclockwise.

Free-surface vortices of type 3 occurred more often than observed in the previous tests, mostly in intake unit 3 and sometimes in unit 2. As in the previous tests, subsurface vortices of type 2 occurred in all pump intake units, while free-surface vortices occurred in intake units 1 and 2 (photos 5.9 to 5.11). The perforated plates helped to produce a more even approach flow to the pump columns and reduced the swirl angles in all except in pump column #6. Therefore, the perforated plates were in place in the intakes for the next tests. Again, flow stagnation areas were observed in the corners between the floor, the backwall, and the sidewalls. Large-scale rotations were observed around the pump columns.
Area of pie section ADE = \( \frac{1}{6} \times \left(\frac{\pi}{4}\right) \times 3^2 = 1.1781 \text{ sq in} \)

Area of equilateral triangle ABC = \( \frac{1}{2} \times 5 \times 5 \times \sin(60 \degree) = 10.825 \text{ sq in} \)

Therefore, the open area percentage is
\[ \frac{3 \times 1.1781}{10.825} = 0.326 \text{ (32.6\%)} \]

Figure 5.13: Perforation pattern for perforated plate

Figure 5.14: Velocity Distribution in pump #2
5.5.5 Test Case 4: Perforated Plates, Splitters, and Corner Fillets Installed

Splitters are generally used to suppress boundary-attached vortices whereas fillets are used to suppress flow separation and stagnation areas. Since those flow phenomena were observed in the previous tests, floor- and backwall splitters as well as floor-sidewall and floor-backwall fillets were installed in test 4. Figures 5.15 to 5.17 show drawings of the designed splitters and fillets for each pump intake unit. Photo 5.12 shows the installed splitters and fillets at pump column #3.
Splitters and fillets were installed only at one of the two pumps in each intake unit to study the effectiveness of these devices. In the previous three tests, always a similar vortex-behavior was observed at two pumps in the same intake unit.

The velocity pattern inside the pump columns was not significantly changed compared to the measurements in the previous tests. The maximum deviation of a single velocity from the medium velocity on a concentric circle was 1\% in intake unit 3 and 2\% in unit 2 (figures 5.18 and 5.19).

The swirl angle was reduced in this test to very low values of 1.3° and 0.4° in intake units 3 and 1, while it was slightly increased, but still satisfying with 2.9° in unit 2. The flow-rotation direction was counterclockwise in all the pump columns.
The floor splitter considerably helped create a uniform approach flow to the pump bell. The large-scale rotation around the pump column was eliminated. No persistent flow stagnation or separation areas were found in the corners between the floor, the backwall, and the sidewalls (see photo 5.13). Free-surface vortices were still likely to occur in the left half of the pump intake bays (see photo 5.14).

5.6 Conclusions and Recommendations for ITT 54x42 WCA, ITT 66x48 WCA, and ITT 90x66 WCA Pumps

The experimental results obtained from tests 1 and 2 under as-designed conditions showed swirl angles up to 6.7°, subsurface vortices of type 2, and free-surface vortices of type 3 while the velocity inside the pump column was very uniform. Since the prerotation and vortex values exceeded the criteria for satisfactory pump operation, there was a strong need to install vortex and prerotation suppressing devices into the intake chambers.
Figure 5.18: Velocity distribution in pump #1

Figure 5.19: Velocity distribution in pump #3
Therefore, perforated plates, splitters, and fillets were designed based on experience gained from previous intake investigations conducted at the IIHR. These devices straightened the flow in the intakes, reduced the prerotation to values below 3° and eliminated subsurface vortices.

Free-surface vortices observed in all test cases were able to be successfully suppressed by a surface beam that was installed at the downstream end of each internal pump-sump bay. Each surface beam should be such that the bottom elevation is located at EL 1045 ft above MSL and 5 ft tall. The thickness of each surface beam should be determined assuming the maximum differential head of 5 ft. This free-surface vortex suppressor was used in additional model tests that are reported in the following chapter.

VI. ADDITIONAL PUMP-INTAKE MODEL TESTS

It was requested by MidAmerican Energy Company that additional tests be conducted with different pumps manufactured by FlowServe and ITT because final bids were not in during the present model investigations.
6.1 FlowServe Pumps

Three different FlowServe pumps that could be used for units 1, 2, and 3 were tested, including 48 VX, 39 APM, and 57 APM pumps. Their physical parameters are shown in table 6.1 and prototype pump bellmouth profiles are shown in figures 6.1a through 6.1c, respectively.

<table>
<thead>
<tr>
<th></th>
<th>Unit 1 CW</th>
<th>Unit 2 CW</th>
<th>Unit 3 CW</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pump Model</td>
<td>48 VX</td>
<td>39 APM</td>
<td>57 APM</td>
</tr>
<tr>
<td>Speed (rpm)</td>
<td>585</td>
<td>509</td>
<td>354</td>
</tr>
<tr>
<td>Floor Clearance (in.)</td>
<td>27</td>
<td>19</td>
<td>28</td>
</tr>
<tr>
<td>Bell Diameter (in.)</td>
<td>54</td>
<td>64</td>
<td>98.5</td>
</tr>
<tr>
<td>Column Diameter (in.)</td>
<td>48</td>
<td>54</td>
<td>80</td>
</tr>
</tbody>
</table>

Table 6.1: Characteristics of FlowServe pumps

Figure 6.1a: Full-scale unit-1 model pump bellmouth (FlowServe 48 VX pump)

Figure 6.1b: Full-scale unit-2 model pump bellmouth (FlowServe 39 APM pump)
The final design layout for the unit-1 pump bays with the FlowServe 48 VX pumps is shown in figure 6.2. Under the modified bay conditions, no vortex appeared within the pump bays and desirable pump-approach flow conditions were achieved, as illustrated in photo 6.1. The maximum swirl angle observed under the final bay configurations was 1.1° and no vortex was observed.

Pump-bay modifications designed for unit 2 pump bays using the FlowServe 39 APM pumps are shown in figure 6.3. Excellent pump-approach flow patterns observed are shown in photo 6.2. The maximum swirl angle in pump #3 was 1.9°.
Photo 6.1: Smooth pump-approach flow pattern in pump #5 (FlowServe 48 VX pump) under final bay modifications

Figure 6.3: Splitters and fillets designed for intake unit 2 (FlowServe 39 APM pump)
The final bay modifications designed for unit-3 intake bays using the FlowServe 57 APM pumps are shown in figure 6.4 and desired pump-approach flow patterns observed in pump bay #1 are depicted in photo 6.3. Pump-throat velocity distributions obtained under the as-designed (no bay modification) and the modified pump-bay configurations are shown in figures 6.5 and 6.6, respectively. The maximum swirl angle was 3.8° under the as-designed conditions, but it was 1.9° when the pump-bay modifications were implemented. No vortex formed under the final pump-bay modifications.
Figure 6.4: Splitters and fillets designed for intake unit 3 (FlowServe 57 APM pump)

Photo 6.3: Smooth pump-approach flow pattern in pump #1 (FlowServe 57 APM pump) under final bay modifications
Figure 6.5: Velocity distribution obtained in FlowServe 57 APM pump #1 under as-designed conditions

Figure 6.6: Velocity distribution obtained in FlowServe 57 APM pump #1 under final bay modifications
6.2 ITT Pumps

One ITT pump, 102x72 WCA, that could be used for unit 3 was tested in this phase of the model investigations. ITT pumps to be used for units 1 and 2 are 54x42 WCA and 66x48 WCA whose models were investigated previously and the test results are presented in chapter 5. It should be noted that ITT 90x66 pumps that were initially tested for unit 3 intake (test results shown in chapter 5) will be replaced by 102x72 WCA pumps. Table 6 summarizes pump characteristics of the ITT pumps. The prototype pump bellmouth profile for the ITT 102x72 WCA pump is shown in figure 6.7.

<table>
<thead>
<tr>
<th>Pump Model</th>
<th>Unit 1 CW</th>
<th>Unit 2 CW</th>
<th>Unit 3 CW</th>
</tr>
</thead>
<tbody>
<tr>
<td>Speed (rpm)</td>
<td>54x42 WCA</td>
<td>66x48 WCA</td>
<td>102x72 WCA</td>
</tr>
<tr>
<td>Floor Clearance (in.)</td>
<td>600</td>
<td>600</td>
<td>400</td>
</tr>
<tr>
<td>Bell Diameter (in.)</td>
<td>27</td>
<td>33</td>
<td>51</td>
</tr>
<tr>
<td>Column Diameter (in.)</td>
<td>54</td>
<td>66</td>
<td>102</td>
</tr>
<tr>
<td>Column Diameter (in.)</td>
<td>42.5</td>
<td>48.75</td>
<td>72</td>
</tr>
</tbody>
</table>

Table 6.2: Characteristics of ITT pumps

![Figure 6.7: Unit 3 model pump bellmouth (ITT 102x72 WCA)](image)

Pump bay modifications tested for the ITT 102x72 WCA pump are sketched in figure 6.8. Under this bay layout neither floor nor free-surface vortices formed in the pump bay. Photo 6.4 shows laminar flow patterns observed on the pump-bay floor. Pump-throat velocity distributions measured under the as-designed (no bay modification involved) and the final bay modifications are shown in figures 6.9 and 6.10 respectively.
As can be seen in figure 6.10, the maximum velocity deviation of 0.04 was observed in Angle 5. The swirl angle was only 1.3° which is significantly below the critical angle of 5°.

![UNIT-3 INTAKE BAY WITH ITT 102X72WCA PUMP (FULL SCALE)](image)

Figure 6.8: Splitters and fillets designed for intake unit 3 (ITT 102x72 WCA Pump)

VII. RECOMMENDATIONS ON INTAKE STRUCTURES

Through detailed physical model investigations using 1:12-scale geometrically undistorted pump-intake models, desirable pump-operating conditions that satisfy the modeling criteria set forth in section 5.2 were achieved for three pump units. The recommended parameters for the pump-bay modifications can be found in figures that are summarized in table 6.3.

REFERENCE

Photo 6.4: Smooth pump-approach flow pattern on sump floor in pump #1 (ITT 102x72 WCA Pump) under final bay modifications

Figure 6.9: Velocity distribution obtained with ITT 102x72 WCA pump #1 under as-designed conditions (all pumps #1 through #6 in operation)
Figure 6.10: Velocity distribution obtained in ITT 102x72 WCA pump #1 under final bay modifications (pumps #1 through #6 in operation)

<table>
<thead>
<tr>
<th>Pump Model</th>
<th>Modification Layout Shown in</th>
<th>Perforated Plates*</th>
<th>Surface Beams**</th>
<th>Maximum Prototype Swirl Angles Expected</th>
</tr>
</thead>
<tbody>
<tr>
<td>ITT 54x42 WCA (Unit 1)</td>
<td>Figure 5.15</td>
<td>Yes</td>
<td>Yes</td>
<td>0.4°</td>
</tr>
<tr>
<td>ITT 66x48 WCA (Unit 2)</td>
<td>Figure 5.16</td>
<td>Yes</td>
<td>Yes</td>
<td>2.9°</td>
</tr>
<tr>
<td>ITT 96x66 WCA (Unit 3)</td>
<td>Figure 5.17</td>
<td>Yes</td>
<td>Yes</td>
<td>1.3°</td>
</tr>
<tr>
<td>ITT 102x72 WCA (Unit 3)</td>
<td>Figure 6.7</td>
<td>Yes</td>
<td>Yes</td>
<td>1.3°</td>
</tr>
<tr>
<td>FlowServe 48 VX (Unit 1)</td>
<td>Figure 6.2</td>
<td>Yes</td>
<td>Yes</td>
<td>1.1°</td>
</tr>
<tr>
<td>FlowServe 39 APM (Unit 2)</td>
<td>Figure 6.3</td>
<td>Yes</td>
<td>Yes</td>
<td>1.9°</td>
</tr>
<tr>
<td>FlowServe 57 APM (Unit 3)</td>
<td>Figure 6.4</td>
<td>Yes</td>
<td>Yes</td>
<td>1.9°</td>
</tr>
</tbody>
</table>

* Perforation pattern – see figure 5.14 (extend from sump floor to MWL at EL 1060 ft)

* The surface beam should be installed over the entire bay width, extending vertically from EL 1045 ft to EL 1052 ft

Table 6.3: Recommended pump-bay modifications for each pump