LABORATORY TESTS ON HYDRAULIC MODELS OF THE HASTINGS DAM

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CONTENTS

I. INTRODUCTION

ARTICLE PAGE

1 Location _________________________________________________________  7
2 The problem____________________________________________________ 7
3 Purpose of the tests____________________________________________  8
4 Location and personnel___________________________________________10
5 Acknowledgments ________________________________________________ 10
6 General results obtained __________________________________________10

II. THE HASTINGS PROJECT

7 Earth dam_______________________________________________________ 12
8 Lock____________________________________________________________ 12
9 Boule dam______________________________________________________ 12
10 Tainter gate section _____________________________________________14
11 Stilling basin___________________________________________________14
12 Toe protection__________________________________________________14

III. HYDROLOGY OF THE WATERSHED

13 Drainage area ___________________________________________________ 16
14 Precipitation ____________________________________________________16
15 Runoff__________________________________________________________16
16 Temperature____________________________________________________16
17 Regulation_______________________________________________________16

IV. THEORETICAL PRINCIPLES GOVERNING THE USE OF HYDRAULIC MODELS

18 Hydraulic similitude in models____________________________________18
19 Simulation of roughness in hydraulic models_________________________19

V. DESCRIPTION OF GENERAL MODEL

20 The general model _______________________________________________22
21 Lock and spillway gates___________________________________________22
22 Model of river channel____________________________________________23
23 Toe protection___________________________________________________25
24 Setting the model lock and dam____________________________________25
25 River baffle______________________________________________________25
26 Tailwater rating curves____________________________________________26
27 Tailwater control__________________________________________________26
28 Channel gages____________________________________________________28
29 Laboratory water supply____________________________________________29
30 Method of measuring water supply_________________________________29
31 Calibration of 90-degree V-notch weirs_______________________________29

VI. ENLARGED MODEL IN GLASS FLUME

32 Purpose of glass flume tests_______________________________________30
33 Construction of the glass flume____________________________________30
## CONTENTS

<table>
<thead>
<tr>
<th>ARTICLE</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weir</td>
<td>33</td>
</tr>
<tr>
<td>Calibration of weir</td>
<td>34</td>
</tr>
<tr>
<td>Model discharge</td>
<td>34</td>
</tr>
<tr>
<td>Construction of model</td>
<td>35</td>
</tr>
</tbody>
</table>

### VII. POND REGULATION REQUIREMENTS

| 38 Flowage problems                                  | 36   |
| 39 Scope of the model tests                          | 36   |

### VIII. EROSION BELOW ORIGINAL STILLING BASIN FOR CONSTANT POND LEVEL

| 40 Tests on general model                            | 37   |
| 41 Scour conditions below apron                       | 38   |
| 42 Back roller below apron sill                       | 40   |
| 43 Tests in glass flume                               | 40   |
| 44 Stilling basin inadequate                          | 40   |

### IX. DEVELOPMENT OF NEW STILLING BASIN FOR CONSTANT POND LEVEL

| 45 Theoretical considerations                         | 42   |
| 46 Elevation of apron floor determined                | 44   |
| 47 Length of apron determined                         | 45   |
| 48 Tests on baffle piers                              | 49   |
| 49 Other stilling basin features—sloping apron, sills, etc | 49   |
| 50 New stilling basin tested in the general model     | 51   |
| 51 Stilling basin side walls                          | 51   |
| 52 Final design for new stilling basin                | 51   |

### X. EROSION BELOW ORIGINAL STILLING BASIN UNDER VARYING POND LEVEL

| 53 Operation requirements                            | 53   |
| 54 Original stilling basin inadequate                 | 54   |
| 55 Formation of sand bar below boule' dam             | 54   |

### XI. DEVELOPMENT OF NEW STILLING BASIN FOR VARYING POND LEVEL

| 56 Length of apron                                    | 57   |
| 57 Elevation of apron and final design                | 57   |
| 58 Location of new stilling basin                     | 59   |
| 59 Test on stilling basin in general model            | 61   |
| 60 Attempted spreading of jet from stilling basin     | 61   |
| 61 Design of stepped sill for tainter gates           | 61   |

### XII. MISCELLANEOUS STUDIES

| 62 Dredging at east end of spillway section           | 64   |
| 63 Study of currents near the structure               | 64   |
| 64 Calibration of Hastings dam                        | 67   |
| 65 Operation schedule                                 | 69   |
| 66 Photographs                                        | 71   |
# LIST OF ILLUSTRATIONS

<table>
<thead>
<tr>
<th>FIGURE</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Map of watershed above Hastings dam</td>
</tr>
<tr>
<td>2</td>
<td>Map of the site of Hastings lock and dam</td>
</tr>
<tr>
<td>3</td>
<td>Typical cross section of earth dam</td>
</tr>
<tr>
<td>4</td>
<td>General plan and sections of Hastings lock</td>
</tr>
<tr>
<td>5</td>
<td>General plan and section of boule pass and spillway of Hastings dam</td>
</tr>
<tr>
<td>6</td>
<td>Model of Hastings lock and dam and of river channel completed</td>
</tr>
<tr>
<td>7</td>
<td>Constructing model of Mississippi river channel at the side of the Hastings project</td>
</tr>
<tr>
<td>8</td>
<td>Rating curves of the Mississippi river at Hastings dam</td>
</tr>
<tr>
<td>9</td>
<td>Model of Hastings project with testing apparatus ready for calibrations</td>
</tr>
<tr>
<td>10</td>
<td>View of glass flume</td>
</tr>
<tr>
<td>11</td>
<td>Near view of large-scale model of original stilling basin in glass flume</td>
</tr>
<tr>
<td>12</td>
<td>Looking upstream on the large-scale model in the glass flume</td>
</tr>
<tr>
<td>13</td>
<td>Condition below original stilling basin after discharging 15,000 cubic feet per second for a period of 30 minutes</td>
</tr>
<tr>
<td>14</td>
<td>Erosion below original stilling basin resulting from variable discharge of 15,000 to 35,000 cubic feet per second through four tainter gates</td>
</tr>
<tr>
<td>15</td>
<td>Erosion below Hastings dam resulting from variable discharge of 10,000 to 65,000 cubic feet per second through eight tainter gates</td>
</tr>
<tr>
<td>16</td>
<td>Erosion test on the original stilling basin in the glass flume</td>
</tr>
<tr>
<td>17</td>
<td>Result of the test shown in Figure 16</td>
</tr>
<tr>
<td>18</td>
<td>Curves showing the characteristics of the hydraulic jump</td>
</tr>
<tr>
<td>19</td>
<td>Test in the glass flume on stilling basin with length increased to 120 feet</td>
</tr>
<tr>
<td>20</td>
<td>Stilling basin 120 feet long being tested without baffle piers</td>
</tr>
<tr>
<td>21</td>
<td>Stilling basin shown in Figure 19 with floor lowered 5 feet</td>
</tr>
<tr>
<td>22</td>
<td>Test on stilling basin shown in Figure 21 without baffle piers</td>
</tr>
<tr>
<td>23</td>
<td>New stilling basin for constant pond regulation</td>
</tr>
<tr>
<td>24</td>
<td>New stilling basin shown in Figure 23 in operation</td>
</tr>
<tr>
<td>25</td>
<td>Effect of baffle piers in stilling basin on velocity conditions below the dam</td>
</tr>
<tr>
<td>26</td>
<td>Designs of new stilling basins for constant and for varying pond levels at Hastings dam</td>
</tr>
<tr>
<td>27</td>
<td>Stages required to be maintained at the Hastings dam for 9-foot navigation</td>
</tr>
</tbody>
</table>
## LIST OF ILLUSTRATIONS

<table>
<thead>
<tr>
<th>FIGURE</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>28</td>
<td>Original stilling basin operating under varying pond level, discharge 6,000 cubic feet per second</td>
</tr>
<tr>
<td>29</td>
<td>Stilling basin shown in Figure 28 discharging 15,000 cubic feet per second</td>
</tr>
<tr>
<td>30</td>
<td>New stilling basin designed for varying pond level, discharge 10,000 cubic feet per second</td>
</tr>
<tr>
<td>31</td>
<td>Stilling basin as in Figure 30 with discharge increased to 20,000 cubic feet per second</td>
</tr>
<tr>
<td>32</td>
<td>Stilling basin and discharge as in Figure 30 except flow divided between two stilling basins</td>
</tr>
<tr>
<td>33</td>
<td>Conditions are the same as in Figure 32 except discharge increased to 20,000 cubic feet per second</td>
</tr>
<tr>
<td>34</td>
<td>Condition below new stilling basin designed for varying pond level after test in the small-scale model of the Hastings dam</td>
</tr>
<tr>
<td>35</td>
<td>Effect on downstream velocity conditions of stepped sill below those tainter gates outside of stilling basin</td>
</tr>
<tr>
<td>36</td>
<td>Recommended dredging at Hastings dam to improve flow conditions through east end of spillway</td>
</tr>
<tr>
<td>37</td>
<td>Currents below Hastings dam</td>
</tr>
<tr>
<td>38</td>
<td>Calibration curves of 4 and 8 tainter gates</td>
</tr>
<tr>
<td>39</td>
<td>Calibration curves of 12, 16, and 20 tainter gates</td>
</tr>
<tr>
<td>40</td>
<td>Operation schedule recommended for the Hastings dam</td>
</tr>
</tbody>
</table>
LABORATORY TESTS ON HYDRAULIC MODELS OF THE HASTINGS DAM

1. INTRODUCTION

1. Location—An essential part of the development by the national government of a deeper navigable waterway on the Upper Mississippi river was recently accomplished when a lock and dam designed by and constructed under the supervision of the U.S. Army Engineers was completed across the Mississippi river at Hastings, Minnesota, about 26 miles south of St. Paul. This project constitutes the initial and a very important unit in the proposed plan for canalization of the Upper Mississippi river. The plan contemplates the provision of a series of slack-water pools formed by permanent, low-head dams each with a lock or locks, combined with minor channel improvements. In the design of the Hastings dam the engineers were faced with the task of coordinating it with subsequent projects to create a unified system and also of establishing, so far as possible, a reliable precedent whereby the dams designed later for similar sites along the Upper Mississippi river might be built. Figure 1 shows the location of the Hastings dam with respect to the drainage area under the supervision of the St. Paul District Office, and Figure 2 orients the project with local geographical features.

2. The problem—In general, the dams will be constructed upon unstable, alluvial foundations and must be supported on wood piles and protected by steel sheet piling. Under conditions of this nature it is imperative that the structures be safeguarded against disastrous erosion and under-cutting below the dam. Few exact criteria have yet been evolved whereby such a problem can be solved successfully. Heretofore it has been left to the judgment and experience of the engineers to provide such protection in the design as was considered to be necessary and adequate. Recently engineers have conceived a new and very inexpensive method of direct attack on problems of this kind. Models are now built of hydraulic...
structures and are studied under laboratory control, in which any manner of field condition can be simulated. Pertinent problems that defy mathematical analysis present little difficulty when subjected to this method of solution. Without question great economies will be effected in the future construction of hydraulic works by thus enabling the engineer to assure himself of correctness in design before undertaking actual construction. The design of the Hastings lock and dam was based on available information and was consistent with sound theory and good practice. However, in order to leave no point in the safety of this important structure open to question, it was decided to study its behavior in hydraulic models under simulated operating conditions.

3. Purpose of the tests—The primary aims in undertaking these model investigations were as follows:

Fig. 1. Map of the territory under supervision of the U. S. Engineer Office, St. Paul District. The district includes the watershed of the Upper Mississippi river above and including the Wisconsin river. The outline of the drainage area above Hastings dam is shown by a heavy broken line.
Fig. 2. Map of the site of Hastings lock and dam.
A. To determine whether proper and adequate provisions had been made in the design for the prevention of erosion below the dam.

B. To develop corrective features if and as found necessary.

C. To study current conditions which would affect navigation, particularly in the lock approaches.

D. To study hydraulic characteristics of the structure and determine coefficients for the spillway section.

E. To determine the scheme of operation which will prove most advantageous to navigation and which will at the same time minimize the possibility of erosion along the toe of the dam.

F. To make such general studies as will be of value in connection with the operation of the dam.

G. To make general studies applicable to the design of other similar dams.

4. Location and personnel—The investigations were conducted in the hydraulic laboratory of the University of Iowa, at Iowa City, during the period November 1929 to April 1930, under the direction of the District Engineer at St. Paul, Minnesota. H. M. Hill, Engineer at St. Paul, laid out the broad lines of the experiments, supervised the testing, and made frequent visits to the laboratory to study the models. H. M. Anderly, Engineer in charge of construction of the Hastings dam, also kept closely in touch with the experiments. The tests were made by Martin E. Nelson, Assistant Engineer, and C. Donald Rieck, Surveyman, of the St. Paul District Office.

5. Acknowledgments—Much indebtedness is acknowledged to Professor Floyd A. Nagler, director of the laboratory, for generous advice and assistance of great value which he rendered during the construction and testing of the models as well as in the interpretation of the results of the experiments; likewise, to Mr. D. L. Yarnell, Senior Drainage Engineer of the U. S. Bureau of Public Roads, who greatly assisted in the work by the sharing of his experience in model testing.

6. General results obtained—Much to the gratification of the engineers who were interested, these experiments made on models of the Hastings lock and dam proved to be very successful. In fact, more valuable results than had been hoped for at the outset were achieved.
Details of the various tests conducted and accomplishments attained will be recounted in the body of this paper, but briefly stated, the following observations and conclusions may be noted here.

A. The adequacy of the original design, particularly the proposed stilling basin for protection of the dam against erosion, proved to be doubtful under the most favorable operating conditions.

B. If subjected to other than normal service, the stilling basin was found to give unsatisfactory protection against scour.

C. Operation of the stilling basin located adjacent to the boule dam caused a bar to be deposited so as to obstruct entrance to the navigable pass.

D. For the above causes it was deemed advisable to supplement the stilling basin in the design with an additional stilling basin, location and dimensions of which were developed by means of model experiments.

E. Calibrations of the dam with several combinations of gate openings were made and discharge coefficients were computed for the dam under various conditions of operation.

F. A scheme of operation was developed which will minimize the possibility of scour and which will maintain a flow condition in the lock approaches satisfactory for navigation.

G. It was discovered that three gates at the east end of the dam, which are located in a sand bar, were far less effective than other sections of the dam in carrying flood discharges. For increasing their efficiency, dredging up and downstream was found necessary.

H. Much general knowledge was gained regarding the use of models for hydraulic investigations. The practicability of such studies was found to warrant their continuance in connection with other navigation projects in the Upper Mississippi Valley Division.

I. A great deal was learned regarding the action of stilling basins and valuable data was secured which may be applied toward the development of general principles of design.

J. A type of stepped sill was developed to be used below the tainter gates not located in either stilling basin, to protect the stream bed immediately downstream from these gates against erosion.
II. THE HASTINGS PROJECT

7. Earth dam—An earth fill dam about 3250 feet long with an approximate average height above ground surface of 16 feet was constructed across Buck Island and Lake Rebecca, low areas west of the main channel of the Mississippi river. A cutoff wall of steel sheet piling 25 to 40 feet long was driven into the foundation along the center line of the earth dam. A typical section is shown in Figure 3.

8. Lock—A lock, 110 feet wide and 500 feet long in the clear, with a lift of about 20 feet at low water, was built along the west side of the main channel of the river. The lock gates are of the vertically framed type. A plan and sections of the lock are shown in Figure 4.

9. Boule dam—A navigable pass, or boule dam, 100 feet wide, was constructed in the river channel adjacent to the lock. When the pass is closed, eight steel A-trestles, hinged to the sill, are raised into an upright position parallel to the direction of flow and support steel panels provided with rollers. Should some emergency require operation of the lock to be suspended, the panels can be lifted by means of a small derrick and the supporting frames folded into a recess in the boule floor, leaving an unobstructed navigable passage, 100 feet wide and about 6.5 feet deep at extreme low water stage. During high floods this pass may also be used to enlarge the spillway capacity of the dam. The location and general features of the boule dam are shown on Figure 5.
Fig. 4. General plan and sections of Hastings lock.
10. **Tainter gate section**—A spillway section of 20 steel tainter gates, each 30 feet wide and 20 feet high, spans the remainder of the channel. The top of the tainter gate sill is at elevation 676.4, about 0.2 foot lower than minimum low water. The piers between the gates are 5 feet thick, their upstream faces are vertical semi-cylinders, and stop log grooves are provided in either side to permit closing of individual openings in case it becomes necessary to inspect or repair the gates. On Figure 5 are shown the spillway section and details of the tainter gates.

11. **Stilling basin**—In accordance with the original design of the Hastings dam a stilling basin was constructed adjacent to the boule pass. This basin receives the discharge from four tainter gates. It is 80 feet long and is enclosed by a sill 5 feet high. On the floor of the basin, which is 5.2 feet below low water, are located two lateral rows of baffle piers 26 and 41 feet, respectively, downstream from the tainter gate sill. The piers of one row are staggered with respect to those of the other row. The general features of the stilling basin are shown on Figure 5.

12. **Toe protection**—For further protection against erosion below the dam, cribs of 10-inch by 12-inch timber are built along the downstream side of all the units in the movable section and are filled with stone varying in weight from 15 pounds to 1000 pounds. A section of riprap, 12 to 55 feet wide, of 0.5 to 3-ton derrick rock, is placed on the river bottom downstream from the cribs.
Fig. 5. General plan and section of boule pass and spillway of Hastings dam.
III. HYDROLOGY OF THE WATERSHED

13. Drainage area—Above the Hastings dam the Mississippi river has a drainage area of about 37,040 square miles, practically all within the state of Minnesota (See Fig. 1). The headwaters of the main streams in the watershed rise in north central and western Minnesota in a marshy terrain.

14. Precipitation—The mean annual precipitation over the watershed increases quite uniformly from about 23 inches on the west to about 29 inches along the eastern border. Approximately 65 per cent of the precipitation falls during the summer in the months of May to September, inclusive, and only 10 to 20 per cent of this annual precipitation falls as snow, the higher percentage occurring along the northern end of the watershed.

15. Runoff—The average flow in the Mississippi river at Hastings, as observed over a period of 26 years, has been 10,010 cubic feet per second, and the maximum recorded discharge was 117,000 cubic feet per second, which occurred on July 22, 1867. This value was used in designing the spillway capacity of the dam. The lowest average discharge recorded during a period of 15 consecutive days was 2,176 cubic feet per second and occurred August 1 to 15, 1926.

16. Temperature—The mean annual temperature ranges from 38 to 44 degrees Fahrenheit, increasing from north to south. Mean monthly temperatures below 45 degrees occur in the northern part from October to April, inclusive. The streams in the watershed are, as a rule, frozen over from the latter part of November to about the middle of March, and snow covers the ground during November or December to March, inclusive. The Mississippi is generally open for navigation from April 11 to November 10, inclusive, the exact date of opening and closing depending upon the time of spring thaw and fall freeze-up.

17. Regulation—As a part of an original scheme to aid in maintaining a navigable depth in the Upper Mississippi river
below the Twin City lock and dam, the government constructed at the headwaters a series of six reservoirs, begun in 1884 and completed in 1912, for the purpose of storing water during the periods of excess flow and discharging the same when the channel depth becomes too shallow for navigation. These reservoirs ordinarily increase the low summer flow by as much as 5,000 cubic feet per second, but have no material effect on flood flows.
IV. THEORETICAL PRINCIPLES GOVERNING THE USE OF HYDRAULIC MODELS

18. Hydraulic similitude in models—When small-scale models are to be used for the purpose of analyzing hydraulic problems, it is very important that the basic principles are fully understood. Furthermore, in order that all hydraulic phenomena in the model shall be exactly homologous to corresponding conditions in the original, and in order that the results from model experiments may be applied to the prototype with assurance of accuracy, it is necessary that the model be constructed and operated in strict accordance with the laws relating to hydraulic similitude. This requires that the form of the stream bed, the surface texture, linear and transverse sections, and superimposed structures be reproduced in conformity to geometric similarity. It also demands that the paths described by certain water particles in the smaller system shall be geometrically similar and have the same proportionality factor as have the paths of corresponding particles in the original.

The fundamental relations between similar systems are comparatively simple and can be easily derived by using the following relations for the various quantities involved:

<table>
<thead>
<tr>
<th>Name of Quantity</th>
<th>Full Size</th>
<th>Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width, in feet</td>
<td>B</td>
<td>Bm</td>
</tr>
<tr>
<td>Depth, in feet</td>
<td>D</td>
<td>Dm</td>
</tr>
<tr>
<td>Head, in feet</td>
<td>H</td>
<td>Hm</td>
</tr>
<tr>
<td>Area, in square feet</td>
<td>A</td>
<td>Am</td>
</tr>
<tr>
<td>Velocity, in feet per second</td>
<td>V</td>
<td>Vm</td>
</tr>
<tr>
<td>Discharge, in cubic feet per second</td>
<td>Q</td>
<td>Qm</td>
</tr>
</tbody>
</table>

Let \( R \) denote scale ratio or the ratio of linear dimensions in the original to linear dimensions in the model; i.e., \( R = \frac{B}{B_m} \)

Then,

\[
\begin{align*}
B &= RB_m \\
D &= RD_m \\
H &= RH_m \\
A &= BD = R^2B_mD_m = R^2A_m \\
V_m &= C(2gH_m)^{0.5} \\
V &= C(2gH)^{0.5} = C(2gRH_m)^{0.5} = R^{0.5}V_m \\
Q &= AV = R^2A_mR^{0.5}V_m = R^{2.5}Q_m
\end{align*}
\]

18
In model testing it sometimes becomes necessary for practical reasons, and it is entirely conformable with the theory, to construct the model on a distorted scale; that is, the vertical scale is made different from the horizontal scale; in which case the following relations will exist between model and prototype.

If now $R_h$ and $R_v$ represent the horizontal and vertical scale ratios, respectively,

$$
B = R_h B_m
$$
$$
D = R_v D_m
$$
$$
H = R_v H_m
$$
$$
A = B D = R_h B_m R_v D_m = R_h R_v A_m
$$
$$
V_m = C(2gH_m)^{0.5}
$$
$$
V = C(2gH)^{0.5} = C(2gR_v H_m)^{0.5} = R_v^{0.5} V_m
$$
$$
Q = AV = R_h R_v A_m R_v^{0.5} V_m = R_h R_v^{1.5} A_m V_m = R_h R_v^{1.5} Q_m
$$

If for the purpose of illustrating these relationships, it be assumed that a model is built with horizontal and vertical scale ratios of 1 to 100 and 1 to 50, respectively, which were the proportions used in the general model of the Hastings dam, then,

$$
V_m = \frac{V}{R_v^{0.5}} = \frac{V}{50^{0.5}} = 7.07
$$

and

$$
Q_m = \frac{Q}{R_h R_v^{1.5}} = \frac{Q}{100(50^{1.5})} = 85,855
$$

19. Simulation of roughness in hydraulic models—It may be well here to mention briefly in what manner the roughness of a stream is simulated in a model. It is evident from the definition of geometric similarity that the coefficient of roughness in a model must be considerably less than that in the original, and the following mathematical procedure will prove that a definite theoretical relationship does exist between the friction factors of the two systems.

The Manning formula for velocity in open channels is:

$$
V = kr^{0.67} s^{0.5}/n
$$

in which, $V =$ mean velocity in feet per second,

$n =$ coefficient of roughness—the same as used in Kutter's formula,

$r =$ mean hydraulic radius, in feet,

$s =$ slope of the water surface,

$k =$ 1.486.
Geometric similarity requires that in an undistorted model the bottom slopes be identical in both systems and it can easily be proved that water surface slopes for steady flow must also be equal.

Therefore,

\[ S = \frac{V^2n^2}{k^2r^{1.33}} = \frac{(V_m^2n_m^2)}{(k^2r_m^{1.33})} \]

\[ n_m = \frac{(nVr_m^{0.67})}{(Vm_r^{0.67})} \]

\[ V/V_m = R^{0.5} \text{ and } r/r_m = R \]

Therefore,

\[ n_m = \frac{(R^{0.5}n)}{R^{0.67}} = n/R^{0.17} \]

An average value of \( n \) for a natural stream may be assumed to be 0.025. If again a scale of 1 to 100 be assumed, the coefficient of roughness in a model of such a stream should be

\[ n_m = 0.025/100^{0.17} = 0.011, \]

which is a very close approximation to the roughness obtained in model channels built of sand-cement mortar.

When the scale of the model is distorted,

\[ S = \frac{V^2n^2}{k^2r^{1.33}} = \frac{S_m(R_v/R_h)}{(V_m^2n_m^2R_v)/(k^2r_m^{1.33}R_h)} \]

\[ V/V_m = R_v^{0.5} \]

Where the river channel is very wide in comparison to its depth, and the ratio of distortion in the model is small, it may be assumed that the hydraulic radius is the same as the depth. Then

\[ r/r_m = R_v \]

and,

\[ n_m = \frac{(nVr_m^{0.67}R_h^{0.5})}{(Vm_r^{0.67}R_v^{0.5})} = nR_h^{0.5}/R_v^{0.67} \]

If, as in the Hastings model,

\[ R_v = 50 \]

\[ R_h = 100 \]

and assuming that \( n \) again equals 0.025

\[ n_m = 0.025(100)^{0.5}/50^{0.67} = 0.018 \]

This indicates that a model constructed to a distorted scale should be formed with a considerably coarser surface texture than one constructed to a uniform scale, because the slopes in the channel are exaggerated by an amount equal to the distortion ratio. Two methods may ordinarily be employed to supply the necessary resistance to flow to compensate for this increased slope. (1) The surface texture of the channel may be suitably roughened; or (2) the slope of the entire
model may be adjusted until the correct velocities are obtained. The latter method may be applied very easily if the flume containing the model channel is so constructed as to permit its being tilted until the proper water surface slope is obtained for the desired condition of flow.

Thus it is seen that friction conditions in models of river channels can be simulated with at least as great a degree of accuracy as that with which the value of $n$ in the natural stream is known. In model experiments, such as those made on the Hastings dam, which are concerned with relatively short sections of river channel in the vicinity of the dam site, any error made in simulating friction in the model will be unimportant in regard to its effect on the specific problems involved. In projects, however, which involve river control, channel erosion, sedimentation, and determination of backwater and surface gradient, roughness becomes a vital factor and a careful representation of it in the model is extremely important.
V. DESCRIPTION OF GENERAL MODEL

20. The general model—Since problems varying greatly in scope were involved in these studies, it was necessary to construct and test two models in order to cover properly the field of investigation. A small-scale model of the whole structure was made for studying current conditions affecting navigation and for observing general hydraulic characteristics of the dam. Certain sections of the spillway were represented in a model on a somewhat larger scale in order to make more detailed studies of erosion, its causes and prevention, which model is described in a later section of this report.

The general model included the entire Hastings structure with the exception of the earth dam. The latter has no spillway area and consequently would not greatly affect the problems under investigation. The total length of the movable dam including the lock is about 1000 feet, a model of which on a horizontal scale of 1 to 100 could be conveniently accommodated in a steel testing flume, 10 feet wide and 42 feet long, made available for this purpose. In order to avoid streamline flow in the model, and to make possible more accurate determinations of water surface elevations, the vertical scale ratio was made equal to 1 to 50. The fixed parts of the model were constructed of seasoned white-pine lumber. All joints were set in white lead, and all exposed surfaces given two coats of linseed oil and about five coats of shellac, as a protective measure against the distortion that would tend to take place when the model was submerged. This preparation did not eliminate warping entirely, but during four months of alternate exposure to water and drying, the elevations of the sills changed only a few thousandths of a foot.

21. Lock and spillway gates—The gates in the lock were made of one-sixteenth inch sheet iron, bent to fit the angle of the mitre sills. The boule dam was closed by a single piece of one-sixteenth inch sheet iron.
THE GENERAL MODEL

A full-size tainter gate in the Hastings spillway has a curved upstream face with a radius of 28 feet. A distorted model of such a tainter gate takes the shape of an elliptical instead of a circular segment. In order that the discharge coefficient for partially opened gates should not be materially affected by an arbitrary distortion in shape or in relative movement, the model gates were made in the form of the best circular segment which practically coincided with their theoretical elliptical curve, and were swung about an axis which caused them to travel along the same relative paths as are described by the full-size gates. This caused the axis of the gates in the model, as shown on Figure 6 and on subsequent photographs of the general model, to pass through a point downstream from their actual center on the full-size structure.

![Fig. 6. Model of Hastings lock and dam and of river channel completed. Scale: 1 to 100 horizontal, 1 to 50 vertical.](http://ir.uiowa.edu/uisie/2)

22. Model of the river channel—To direct the water to and away from the model dam in a manner duplicating nature, a section of the river channel at the site, about 4000 feet long, was reproduced to scale in the testing flume. From soundings made in the river in July 1929, at intervals of 200 feet, model cross sections were cut in 28-gage galvanized sheet iron. These strips, clamped loosely between two 1-inch by 3-inch boards and placed in position in the bottom of the flume, were adjusted up or down until their tops were at the correct elevations as determined by means of a surveyor’s level. Then the boards were nailed together, holding the cross section of the model river channel securely between them. In this man-
ner cross sections of the river at intervals of 2 feet, corresponding to 200 feet in nature, were placed in the flume and the space between them paved with a sand-cement mortar. (See Fig. 7.) The surface of the concrete was brought to the correct contours by troweling until uniform curves and lines were obtained between the cross sections.

Immediately downstream from the model dam a section of the channel about 6 feet long, corresponding to 600 feet in nature, was not paved as described above, but was formed in sand for the purpose of studying the stability of the stream bed, at and below the toe of the dam as shown on Figure 6. The sand used in the scour area was not reduced in size by the same proportion as were all other parts of the model since it would have been practically impossible to obtain sand of the required fineness. Silt would have quickly washed out of the model and in a short time created a nuisance in the laboratory water supply system. Although quantitative determinations of erosion could not accurately be made on a stream bed of sand coarser than model proportions, it was possible to make comparative studies of the effect of various toe protection features, and any condition indicated to be unsatisfactory would unquestionably be so, provided all other pertinent factors existed in their proper scale ratios.

Along the east side of the river at the site of the dam, a bar of sand and gravel existed the face of which intersected the structure about 175 feet from the abutment. The eleva-

Fig. 7. Constructing model of Mississippi river channel at the site of Hastings lock and dam. Three templets are shown in position with the bottom course of concrete poured between them.

http://ir.uiowa.edu/uisie/2
tion of the top of the bar varied from about 680 to 695, and it was not overflowed except at flood stages. In order to make the four tainter gates located on the bar effective in discharging water, it was necessary in the model to cut away a part of the bank, upstream and downstream, to the elevation of the tainter gate sill, and back as far as the east abutment.

23. **Toe protection**—The timber cribbing, which is a part of the toe protection along the downstream side of the lock and dam, was represented in this model by strips of clear white pine, one-eighth inch by three-sixteenths inch, tied together in the correct form by means of copper wire. Protection stone placed in the cribs, and below them, has been specified in two classifications: (1) Riprap to be rough, unhewn quarry run, varying in size from 15 to 200 pounds each; and (2) derrick stone to range in weight from 0.5 to 5 tons. It would have been impractical to attempt to proportion protection stone for the general model to correct geometric dimension because of its distorted scale. Instead, the riprap and derrick stone were proportioned to provide the same relative volume for the model, that is, 1/500,000 of the volume of the size specified for the actual structure, and were selected by screening gravel through the nearest standard size sieves. The following table gives the sieve sizes which were used:

<table>
<thead>
<tr>
<th>Stone in full size structure</th>
<th>Sieve used for model stone</th>
</tr>
</thead>
<tbody>
<tr>
<td>15-pound</td>
<td>No. 10</td>
</tr>
<tr>
<td>200-pound</td>
<td>¼-inch</td>
</tr>
<tr>
<td>½-ton</td>
<td>⅜-inch</td>
</tr>
<tr>
<td>5-ton</td>
<td>¾-inch</td>
</tr>
</tbody>
</table>

24. **Setting the model lock and dam**—The model of the lock and dam was set about midway of the testing flume. Its upstream side was bonded into the concrete river channel by means of several large spikes driven part way into the wood. All sills were set accurately to their correct relative elevations by means of a surveyor’s level, and in checking after the concrete around the model was set, the elevations were found to be in error by an average of only one-thousandth of a foot, corresponding to 0.05 foot in the full-size structure.

25. **River baffle**—The weirs discharged into the upper end of the steel testing flume through an inverted hopper, constructed to spread the water uniformly across the channel.
A crushed-rock baffle was placed about a foot downstream from the beginning of the model river channel, to comb the water and to reduce the turbulence which would otherwise accompany its entrance into the river. By means of canvas shields placed behind the baffle, the regimen of the stream approaching the model was controlled, to duplicate as nearly as possible the natural conditions.

26. **Tailwater rating curves**—An open-river rating curve for the Mississippi river at Hastings by which to duplicate tailwater stages in the model was not available. The Chicago, Burlington, and Quincy R. R. Co., however, has kept a 50-year record of daily stages at the railroad bridge at Hastings. Copies of readings taken during the period 1916 to 1928, inclusive, were obtained, from which, in conjunction with the St. Paul record of discharge—the flow at Hastings being substantially the same as that at St. Paul—a fairly reliable rating curve was developed for the dam site. The stage of the Mississippi river at Hastings is somewhat affected by backwater from the St. Croix river, which enters about 3 miles downstream, the effect varying with the flow in both streams. In the model studies it was necessary to know the maximum and minimum stages which might occur for any flow at the dam, the one affecting the hydraulic capacity of the structure and backwater conditions upstream, and the other affecting erosion problems at the toe of the dam. A series of curves was therefore drawn up, using the method described above and at the same time segregating the plotted points according to the quantity of flow discharged by the St. Croix river on the corresponding dates. Rating curves were thus obtained for the Mississippi River at Hastings for the following amounts of discharge from the St. Croix river: 3000 cubic feet per second; 10,000 cubic feet per second; 20,000 cubic feet per second; 30,000 cubic feet per second; and 40,000 cubic feet per second. These curves are shown on Figure 8.

27. **Tailwater control**—The tailwater elevations in the model were controlled by means of a flap gate hinged to the bottom of the flume at the lower end of the model river channel, which could be raised or lowered with a hoist constructed for that purpose. The gate then acted as an overflow weir whose crest
Fig. 8. Rating curves of Mississippi river at Hastings dam with backwater effects from various discharges of St. Croix river.
was raised or lowered as required to give the desired tailwater elevation as indicated by the point gage a short distance upstream from the control. On Figure 9 is shown the general model and apparatus.

28. **Channel gages**—Staff gages, reading in feet of stage above low water, were set on the river lock wall, above and below the dam. For more accurate determination of water surface elevations one Gurley point gage was set over the pond, about 6 feet, corresponding to 600 feet in nature, upstream from the dam; and another was set over the tailwater channel about 16 feet, corresponding to 1600 feet in nature, downstream from the dam. The gage over the pond was fastened to a rider which could be moved on a horizontal transverse bar for measuring the pond elevation at any point across the channel. The tailwater gage was set stationary over the middle of the main course of the stream. The gages were set to a common zero by taking simultaneous readings on a horizontal water surface with no flow passing through the flume. The gages were re-checked as the tests progressed and were found to have retained a common zero reading until
the very close of the experimental work, when the pond gage
read 0.003 foot, corresponding to 0.15 foot in nature, higher
than the tailwater gage.

29. Laboratory water supply—A circulating water supply
is used in all hydraulic experiments in the laboratory. The
water is pumped from two storage pits, located beneath the
first floor of the building, to the top floor into a constant-
level supply tank, from which testing apparatus in any part
of the laboratory can be supplied. In the supply tank is an
overflow crest, 734 feet long, which spills all surplus water
not needed for tests in progress and assures a constant head
on all apparatus supply valves. The waste from the constant-
level tank and the water discharged from all testing apparatus
returns again to the storage pits. The pumping capacity is
about 4000 gallons per minute, obtained by means of two
Gould centrifugal pumps, a 10-inch, 3000 G.P.M. and a 6-inch,
1000 G.P.M. A 2-inch city main can be opened to deliver an
additional 250 gallons per minute, if necessary, and, in emer­
gency, the supply can be increased for short periods by draw­
ing from the storage in the constant-level tank.

30. Method of measuring water supply—Two 90-degree V-
notch weirs were used to measure the quantity of water sup­
plied to the general model of the Hastings dam. The weirs,
one 15 inches and the other 11 inches deep, were cut as ac­
curately as possible by means of hand tools, in the ends of
sheet iron tanks, 3 feet by 3 feet by 4 feet and 2 feet by 2 feet
by 4 feet, respectively. The edges of the weirs were rigidly
braced by small steel angles to prevent such bulging as would
otherwise occur under varying heads. Vertical and horizontal
baffles of 1-inch by 3-inch boards spaced about 1/8 inch apart
were placed in each tank around the supply pipe to obtain
uniform flow approaching the weirs. Gurley hook gages
were used for measuring the head on each weir.

31. Calibration of 90-degree V-notch weirs—The weirs were
calibrated and rating curves developed to give the relation of
discharge to gage height. For the sake of brevity the cali­
bration data is here omitted. It may be mentioned, however,
that the results obtained were in remarkably close agreement
with those given by the Cone formula for discharge over a
90-degree V-notch weir, $Q = 2.487 H^{2.4800}$. 
VI. ENLARGED MODEL IN GLASS FLUME

32. Purpose of glass flume tests—The general model described above was not suitably adapted to quantitative studies of erosion nor for observing secondary currents and their actions on the stream bed. Furthermore, results of experiments performed on large-scale models can be applied with more confidence to the full-size structure than can those from tests on models of smaller scale. For these reasons, and to substantiate the results of capacity calibrations made on the tainter gates in the general model, it was decided to construct also a model of a single tainter gate to an undistorted scale of approximately 1 to 14, which model was to be installed in a glass-sided flume.

33. Construction of the glass flume—For this purpose a temporary wooden flume, 39 feet long, 2.5 feet wide, and about 3 feet deep, was erected on the main floor of the laboratory. For a length of 13 feet the side walls were constructed of heavy glass plates through which the phenomena in and around the model during its operation could be observed. Horizontal and vertical coördinate lines of black thread stretched against the glass facilitated location of points in the model and these became a permanent part of all photographic records made of tests in this flume. The horizontal lines were spaced two feet apart, and the vertical lines 5 feet apart, referred to the full-size structure.

A cylindrical gate installed over a circular orifice in the bottom of the downstream end of the flume was used to regulate and control tailwater elevations. By rotating the cylinder it was raised and lowered on a threaded bar which also guided its direction of movement, thereby increasing or decreasing the area of the discharge opening. Piezometers were connected to the side walls to indicate pond and tailwater elevations. Figures 10, 11, and 12 illustrate the arrangement of this flume, model, and testing apparatus.
Fig. 10. View of glass flume. A large scale model of a tainter gate and section of original stilling basin installed in this flume ready for erosion test.
34. **Weir**—For measuring the quantity of water admitted to the flume, a weir tank, 2.5 feet, by 3 feet, by 8 feet, was constructed of one-sixteenth inch sheet iron, all joints being welded. Into one end was welded a 0.5-inch steel plate, 18 inches high, machined along the top to a thin edge, to form the crest of a rectangular suppressed weir, 2.5 feet long. A Gurley hook gage was used to measure the head above the weir crest. The water supply connection was made to the...
discharge line from the 10-inch pump. From the constant-level tank could then be drawn, through the same pipe line, the supply from the 6-inch pump, and incidentally the supply tank tended to function as an equalizer for variation in the flow from the larger pump.

35. Calibration of weir—The rectangular weir was calibrated by taking volumetric measurements in the east storage pit during time intervals when the head on the weir was maintained practically constant. For heads below 0.7 foot, this calibration was found to produce results almost identical with the values given by the formula,

\[ Q = 3.34H^{1.47}(1 + 0.56 H^2/d^2) \]

From 0.7 to 0.9-foot head, higher discharges were indicated in the calibration than the values obtained from the above equation, the difference varying to as much as 2.5 per cent. Because of the limited capacity of the storage pit it was difficult to obtain an accurate calibration at high heads, only a few seconds being required for the pit to fill, and time did not permit making the number of repeated determinations required to obtain reasonably accurate average results. Therefore, and in view of the close agreement between the theoretical values and the experimental determinations that were made, the values of discharge given by the above equation were used where the calibration was considered doubtful, and where discharges higher than the range of the calibration were required.

36. Model discharge—The relation between the discharge in the model and that in the full size structure is given by the formula,

\[ Q/Q_m = R^{2.5} \]

(See article 18)

As explained below, the scale ratio for the glass flume model was,

\[ R = 13.94 \]

Then,

\[ Q_m = Q/R^{2.5} = Q/13.94^{2.5} = Q/725.8 = 0.00138Q \]

*Handbook of Hydraulics by H. W. King.*
37. **Construction of model**—In this model it was proposed to study sections of the spillway and of the various types of toe protection provided in the design. To make this study as detailed as possible, the model was constructed of a section of the spillway included between the center lines of two succeeding piers only, that is, one tainter gate and a half-pier at either end, to a scale limited only by the inside width of the flume. The glass flume as constructed measured 30.12 inches between the inside walls. A tainter gate section between the center lines of succeeding piers is 35 feet wide in the full-size structure, a model of which was built to fit into the glass flume on a scale of 1 to 13.94. The scale of this model was the same in all dimensions. The sill and piers, which in the full-size structure are of concrete, were built in the model of fir flooring and white-pine lumber, with the grain laid in such directions that the least amount of distortion due to swelling of the wood should take place. The gate was made of steel plate one-eighth inch thick, rolled to proper curvature. It was operated about the center of curvature of its face. The model was placed in the glass flume with its downstream edge about a foot upstream from the glass section. Below the gate were then installed stilling basins and stone protection in model dimensions. For studies on that portion of the spillway not discharging into the stilling basin, the same model gate was used, but the stone protection was then placed immediately below the tainter gate sill. All important details of the model and stream bed under investigation as well as currents affecting them could be observed through the glass walls.

The timber cribbing which forms a part of the toe protection for the dam was represented in this model by rippings, 0.75 inch by 0.88 inch, nailed together in the proper shape.

Stone protection for this model varied in size as follows:

<table>
<thead>
<tr>
<th>Full Size</th>
<th>Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weight</td>
<td>Weight in pounds</td>
</tr>
<tr>
<td>15 lbs.</td>
<td>0.0055</td>
</tr>
<tr>
<td>200 lbs.</td>
<td>0.074</td>
</tr>
<tr>
<td>0.5 ton</td>
<td>0.369</td>
</tr>
<tr>
<td>5 ton</td>
<td>3.69</td>
</tr>
</tbody>
</table>
VII. POND REGULATION REQUIREMENTS

38. Flowage problems—In general, an impounding dam in a stream is operated to maintain a practically constant pond elevation, particularly where power development at the dam is a feature. In the Hastings project, however, power development is of negligible importance. As stated before in this paper, the essential requirement is that pond elevations must be maintained which will provide a channel 6 feet deep (ultimately 9 feet deep) between Hastings and Twin City lock and dam, and yet not incur unnecessary flowage damages above the structure. Hence, as the natural stage of the river rises, a full head at the dam is not necessary for maintaining the project depth, nor is it permissible because of the infringement on privately owned flowage rights. Due to the extensive industrial development which lies in this area, the value of real estate subject to overflow with regulation for a constant pond elevation is prohibitive, particularly in the vicinity of St. Paul and South St. Paul. Consequently, flowage has been purchased only up to the 700 contour in the pool, assuming that, by careful regulation of pond levels, flowage damages can be avoided and yet navigable depths be maintained throughout the project channel.

39. Scope of the model tests—To gain information useful in the design of other similar structures, experiments on the models of the Hastings dam were made to develop adequate toe protection, not only for normal situations, but also for operating conditions varying in degree of severity far beyond that expected to occur at Hastings. In a first series of tests the pond elevations were not varied in the way it is proposed to operate the Hastings dam, but instead, the pond at the dam was maintained constant at a maximum stage of 18.5 feet above low water for all discharges. Another series of tests followed in which operating conditions as they are expected to occur at Hastings were duplicated as consistently as possible.
VIII. EROSION BELOW ORIGINAL STILLING BASIN FOR CONSTANT POND LEVEL

40. Tests on general model—The first step taken in the scour studies on models of the Hastings lock and dam was to determine whether or not the toe protection as designed was adequate and satisfactory, first, under operation maintaining full pond elevation at all discharges, and second, under a system of operation wherein the pond elevations were lowered with increased discharge in accordance with a definite schedule. These studies were concentrated principally upon the stilling basin, since its function is to dissipate scouring velocities under ordinary conditions of flow, at least to the extent that erosion on the stream bed below the dam shall in no wise endanger the stability of the structure. If under either of these conditions of operation, the design of the toe protection was found to be unsatisfactory, corrective features were to be developed.

In the first series of experiments the stilling basin in the general model was tested under various rates of flow below 32,000 cubic feet per second, the maximum capacity of four
tainter gates. The pond was maintained constant at an elevation of 696.4, Memphis datum, which corresponds to an 18.5-foot stage above low water at the dam. Tailwater elevations were kept normal for that condition under which the St. Croix river would be discharging less than 5,000 cubic feet per second.

41. Scour conditions below apron—The stilling basin under this scheme of operation disclosed a very undesirable feature in its behavior in that a pronounced standing wave formed immediately over the cribbing and stone protection below the apron. Other smaller waves followed downstream on the rip-rap and stream bed. These waves had scouring qualities; that is, the velocities along the bottom were relatively higher in proportion to the average velocities in a vertical section than would be the case under normal turbulent flow in the natural channel. This condition existed in a greater or less degree.
at all discharges, but for flows less than 15,000 cubic feet per second the scouring tendency was not damaging to the stone protection nor to the stream bed. As the discharge increased above 15,000 cubic feet per second, however, the wave action became increasingly suggestive of a condition very detrimental to the safety of the structure. The incessant whipping and

pounding of the major wave on the downstream panel of the cribbing and on the riprap, rapidly scoured away all movable material in individual pieces weighing less than about 200 pounds in the full size. Within a short while, the lower half of the cribbing was laid bare to a depth of about 3 feet, and

Fig. 15. Erosion below Hastings dam resulting from variable discharge of 10,000 to 65,000 cubic feet per second through eight tainter gates. The gates were wide open during the test for about 4.5 hours. The tailwater elevation was normal for every discharge. The bottom of the depression is about 25 feet below the tainter gate sills. Note again the sand bar obstructing the approach to the boule pass.
the small derrick stone was washed away. Downstream from the stone protection, a hole in the sand bed was scoured to a depth of 15 feet and this depression kept working back toward the stilling basin. As erosion on the stream bed progressed in the direction of the stilling basin, the stone protection was gradually undermined, and even the heaviest derrick stones were rolled downstream into the pit that had been washed out. Figures 13, 14, and 15 illustrate the results of tests on the original stilling basin in the general model.

42. Back roller below apron sill—The sill at the lower end of the apron had an elevation about 3 feet higher than that of the top of the cribbing. In this leeward area was formed a bottom back roller which tended to provide a natural toe protection by drawing small stones back against the sill, forming a fillet of loose rock in the corner. This protection would be of little value, however, if erosion were to proceed into the cribbing.

43. Tests in glass flume—In tests on the model in the glass flume results were obtained, which were practically identical with those just described and which brought out in even more detail the seriousness of the situation. Here the progress of erosion could be clearly observed—how the stones were transported downstream away from the crib, and how the sand bed eroded and washed away along the lower end of the stone protection. On Figure 16 is shown a test in the original stilling basin in progress in the glass flume and the result is shown in Figure 17.

44. Stilling basin inadequate—During these tests it was clearly indicated that, under the scheme of operation adopted for the experiments, the stability of the structure was in imminent danger, even under normal circumstances. Most practical engineering projects require a factor of safety in design as a provision against failure due to unusual requirements of operation. The factor of safety in the case of the Hastings stilling basin did not appear to be adequate. In view of this situation it was thought advisable to develop by model tests an additional stilling basin, thus providing adequate protection against toe erosion under the most trying operating conditions that reasonably might be expected to occur.
Fig. 16. Erosion test on the original stilling basin in the glass flume. Discharge 29,000 cubic feet per second, pond at a stage of 18.5 feet, tailwater elevation normal. Note the standing waves downstream from the stilling basin.

Fig. 17. Result of the test shown in Figure 16. Duration of the test was about 5 hours. Note the resemblance to the condition shown in Figure 15.
IX. DEVELOPMENT OF NEW STILLING BASIN FOR CONSTANT POND LEVEL

45. Theoretical considerations—The function of a stilling basin is primarily to create and control a hydraulic jump within a restricted area, thus providing a means of dissipating the surplus kinetic energy accumulated by the water released under pressure of the pond through partially open spillway gates. The energy transformation must be complete to such a degree that the residual scouring tendencies can be resisted by the material of which the tailwater stream bed is composed, and the structure safeguarded against dangerous toe erosion.

The two most important factors to be considered in the design of a stilling basin, because they largely govern its performance, are the depth of the tailwater over the floor of the apron and the length of the apron. The first of these is perhaps the more vital of the two, since sufficient depth of tailwater is a prerequisite to the development of a complete hydraulic jump. The apron should be of sufficient length to insure a thorough energy change and to insure a uniform distribution of velocities across the section at the end of the apron as the water is about to be released onto the bed of the stream. Height and shape of the sill around the apron are also of importance in determining the character of flow which passes onto the stream bed. Other devices installed in the basin, such as baffle piers and toothed or plain sills, have their relative merits and are discussed to some extent later in this paper.

It was proposed to design the new stilling basin to be operated under a full head at all discharges below 32,000 cubic feet per second, the approximate maximum capacity of 4 tainter gates. The tailwater was to be maintained normal at all discharges, with a backwater effect due to the St. Croix river discharging less than 5,000 cubic feet per second.

The correct theoretical depth of a stilling basin floor below tailwater is that which will completely develop a hydraulic
jump within the basin. With the aid of the accompanying diagram, Figure 18, the theoretical elevation of the apron for the above mentioned operating conditions will be determined. The curve, "Depth above Jump, $D_1$" is a graphical representation of the equation,

$$\left(\frac{D_1}{K}\right)^3 - \left(\frac{D_1}{K}\right)^2 = \frac{q^2}{2gK^3}$$

where $q$ is the discharge per unit length of dam, or cubic feet per second per foot of dam. The notations are as indicated on the diagram.

![Diagram](http://ir.uiowa.edu/uisie/2)

Fig. 18. Curves showing the characteristics of the hydraulic jump.

The equation for the depth after the formation of the hydraulic jump is

$$D_2 = \frac{D_1}{2} + \left(\frac{2V_1}{g} \frac{D_1}{4} \right)^{0.5}$$

which, when applied in conjunction with the formula for depth above the jump stated above gives the curve "Depth below Jump, $D_2$."

http://ir.uiowa.edu/uisie/2
Several trial computations were made in the determination of apron elevation, but only the final one will be repeated here. Assuming the desired apron elevation to be 665.0, K will be 696.4 (pond elevation) — 665.0 = 31.4 feet; $K^{1.5} = 176.0$. For 32,000 cubic feet per second discharging through 4 tainter gates,

$$q = \frac{32,000}{4 \times 30} = 266.7 \text{ cubic feet per second per foot of spillway}$$

$$\frac{q}{K^{1.5}} = \frac{266.7}{178.0} = 1.52$$

For this value, $D_2$ is found to be 72 per cent of K as shown on the curve; or $0.72 \times 31.4 = 22.6$ feet = depth below the jump. Normal tailwater elevation at 32,000 cubic feet per second is equal to a stage of 9.7 feet above low water. (See Fig. 8.) Then 9.7 feet — 22.6 feet = —12.9 feet, the stage of the apron floor. Reducing this into terms of elevation, 677.9 (zero stage) —12.9 = 665.0, which was the value assumed in the beginning and is the correct elevation for the floor of the stilling basin. By trying several other rates of flow, it is found that the combination of discharge and depth of tailwater at 32,000 cubic feet per second presents the most serious condition and the stilling basin should therefore be designed to handle safely that quantity. The floor of the original stilling basin was at elevation 671.4, theoretically 6.4 feet too high.

46. Elevation of apron floor determined—This series of tests for the purpose of developing a new stilling basin was made on the large scale model in the glass flume, since it permitted a more thorough and detailed analysis of conditions affecting the problem than was possible in the general model. However, the design derived from these experiments was also tested in the latter model to observe its action when operated in conjunction with the entire structure.

Attention was first directed toward determining the best elevation of the apron floor and, incidentally, to check by model experiments the value given by the preceding theoretical analysis. To eliminate any possible effect of a deficient length of apron on the required elevation of its floor, the apron was extended 40 feet downstream making its total length...
120 feet. The stilling basin was tested at the original elevation 671.4, with the 120-foot length but with no marked change in the behavior, as indicated on Figure 19.

The next procedure was to lower the apron 5 feet to elevation 666.4. This change showed a remarkable improvement over both of the previous settings. (See Fig. 21.) The deep standing wave over the cribbing and riprap was almost entirely smoothed out, the tailwater flowing gently and uniformly over the stone protection and causing practically no disturbance of even the smallest stones. The bottom velocities passing over the sill were uniform and not excessive, as demonstrated by the paths and speeds of air bubbles entrapped in the water. Some washing in the sand bed below the apron was observed, but it did not progress to an appreciable depth, and at no time did it threaten to undermine the stone protection. By lowering the apron an additional amount of 1.5 feet to elevation 664.9, a further betterment in the performance of the stilling basin was effected. This improvement was, however, not very marked, and it was thought that little would be gained in the direction of added safety by a further reduction in the apron elevation. Furthermore, it was considered that the practical limit in the depth of excavation for the basin had been reached at this point.

47. Length of apron determined—The next step taken in the development of the stilling basin was to determine the length of apron which would most economically meet the requirements of satisfactory operation. Tests were made on lengths of 100 feet, and 90 feet, in addition to the 120 and 80 foot aprons which have already been described. The 100-foot length gave entirely satisfactory results, but with a 90-foot apron it was found that the distribution of velocities approaching the sill was very irregular and disturbed by eddies. This condition was due, no doubt, to the fact that sufficient distance was not allowed for the disturbances caused by the jump and by the baffle piers to become regular. Thus it was concluded inadvisable to construct an apron shorter than 100 feet, and that the cost of any extension beyond 100 feet would be unwarranted from the standpoint of added protection. On Figure 23 is shown the final design of the new stilling basin, which is also shown in operation on Figure 24.
Fig. 19. Stilling basin 120 feet long. Elevation of this floor same as in the original stilling basin (671.4). Discharge 29,000 cubic feet per second. Pond at stage of 18.5 feet, tailwater normal.

Fig. 20. Stilling basin 120 feet long without baffle piers. Discharge 25,000 cubic feet per second, other conditions the same as in Figure 19.
Fig. 21. Stilling basin 120 feet long with floor at elevation 666.4. Discharge 29,000 cubic feet per second, pond at stage of 18.5 feet, tailwater normal. Contrast this condition with that of Figure 16.

Fig. 22. Stilling basin 120 feet long with floor at elevation 666.4, baffle piers removed. Other conditions the same as in Figure 21. Note wave over riprap slightly deeper than in Figure 21.
Fig. 23. New stilling basin for constant pond regulation. Apron 100 feet long, elevation of floor 664.9. Note new type baffle piers.

Fig. 24. New stilling basin shown in Figure 23 in operation. Discharge 23,000 cubic feet per second, pond at stage of 18.5 feet, tailwater normal.
48. **Tests on baffle piers**—Throughout the entire series of development tests which have just been described, there was kept in mind the possibility of reducing or eliminating entirely the cost of constructing baffle piers in the stilling basin, without unduly sacrificing any valuable benefits which might be derived from their action. Practically every setup was tested both with and without the piers, and without exception the piers served a very useful purpose in reducing the velocities along the bottom of the apron and in concentrating the jump closer to the tainter gate sill. Current meter measurements were taken at three stations below the apron, with and without the baffle piers, to determine the effect of the piers on the velocity distribution in the channel over the riprap. The results are plotted on Figure 25. These measurements were made in conjunction with tests in the development of an apron under the second scheme of operation, and at that time the apron was set at elevation 667.9. It will be noticed that the velocities along the bottom of the channel immediately downstream from the sill are considerably slower when the baffle piers are used than when they are removed. The negative velocities indicate a bottom reverse roller, which, if not too violent, in a measure provides natural protection against undercutting the apron. This action also is most effective when the piers are in place. A comparison of Figures 21 and 22 shows the effect of the baffle piers in reducing the standing wave below the apron.

As shown on Figure 5 the baffle piers in the original design were constructed with a curved upstream face, which requires considerably more expensive formwork than would a rectangular section. Various rectangular shapes were tried out and the design shown on Figure 23 was found to be comparable in performance with those in the original design, with the added virtue of requiring less expensive concrete forms.

49. **Other stilling basin features—sloping apron, sills, etc.**—The investigations led to studies of various other details which might be adapted to the stilling basin in further increasing its economy and utility. An apron, sloping from a point on the floor immediately downstream from the second row of baffle piers to within a foot of the top of the end sill, has worked successfully in other stilling basins, but when ap-
Fig. 25. Effect of baffle piers in stilling basin on velocity conditions below the dam.
plied to the Hastings model it failed to produce satisfactory results. Substitutes for the baffle piers, such as sills of varied shapes and dimensions, were tested with a view toward economy in construction, but were abandoned because of unsatisfactory performance. Time did not permit studying the feasibility of using the toothed or dentated sills in the Hastings stilling basin, nor was it thought that these would demonstrate any particular advantage over the features incorporated in the final design.

To direct the jet discharged from the tainter gates onto the stilling basin floor a series of steps, 26 inches high and 10 feet wide, were constructed below the tainter gate sill.

50. **New stilling basin tested in the general model**—A model of the new stilling basin was constructed for the general model and studied under operation in conjunction with other units in the structure. The new basin was located downstream from gates 8 to 11, inclusive, numbering from the west end of the section. The original apron below gates 1 to 4 was left in place to permit making comparative studies. The action of the new basin proved to be far superior to that of the original. No standing waves occurred over the stone protection, the water flowing smoothly over the stream bed. The riprap, in the crib and below it, was left undisturbed, practically as it was placed in the model. Scour in the sand below the dam did take place, but did not progress to the point where the stone protection was endangered.

51. **Stilling basin side walls**—Because of the greater depth of the new basin there was a tendency for the tailwater to crowd into the jet ahead of the jump and contract the effective width of the basin. To overcome this objectionable feature, walls with crests at elevation 682.9, were built on each side of the stilling basin extending from immediately back of the tainter gate piers to 40 feet downstream.

52. **Final design for new stilling basin**—Thus the final design for a stilling basin for the Hastings dam to be operated under a full head as suggested by these model tests, resolved itself into the structure illustrated in Figure 26 which shows a section through the new stilling basin and its location with respect to the other features of the Hastings structure.
Fig. 26. Designs of new stilling basins for constant and for varying pond levels at Hastings dam.
X. EROSION BELOW ORIGINAL STILLING BASIN UNDER VARYING POND LEVEL

53. Operation requirements—In the second series of model tests on the Hastings lock and dam it was proposed to determine whether or not the original design contained adequate protection against toe erosion when operating under heads decreasing as the discharge increases, in accordance with the scheme set forth graphically on Figure 27. This method of

![Graph](http://ir.uiowa.edu/uisie/2)

Fig. 27. Stages required to be maintained at the Hastings dam for 9-foot navigation.

operation is designed to provide, at all rates of discharge up to 25,000 cubic feet per second, a minimum depth of 6 feet (ultimately a depth of 9 feet after the required dredging has been done) as far upstream as the Twin City lock and dam at Minneapolis. Above 25,000 cubic feet per second the
controlling factor is that a 10-foot depth (11-foot stage) be maintained over the upper miter sill (elevation 678.9 = 1-foot stage) in the Hastings lock, regardless of the fact that this will create a depth upstream greater than required for 9-foot navigation.

54. Original stilling basin inadequate—For these experiments the original apron was again replaced at elevation 671.4 in the glass flume and tested at various discharges under the conditions shown in the diagram of Figure 27. These tests indicated that the stilling basin, in all probability, would be satisfactory under normal conditions of flow, presupposing the most intelligent and meticulous care in operation to be observed at all times. Tests were also made under some abnormal conditions which might readily occur in practice, such as a low tailwater for the quantity of discharge passing through the dam. Such a condition would exist were it ever necessary, on a rising stage, to draw the pond down so rapidly that the tailwater could not build up at a corresponding rate, since the control is located about 4 miles downstream from the dam. The dependability of the stilling basin then became doubtful, although very serious reactions were not observed. However, there was the possibility that other unforeseen conditions might arise, such as the inability to operate one or more gates in the stilling basin, which would bring precarious consequences, and more especially if this should occur during a period of critically low tailwater. Then it would be equally disastrous to open gates outside of the stilling basin. Furthermore, the extreme care with which the dam must be operated even under normal circumstances in order to fulfill all the stipulations for its proper functioning, would be nothing short of impractical. Figures 28 and 29 illustrate the action of the original stilling basin under this method of operation.

55. Formation of sand bar below boule dam—If, when the tainter gates for the original stilling basin are discharging, the boule dam is closed, the body of water between the river lock wall and the stilling basin is set in motion, forming an eddy which moves upstream along the lock wall and back into the jet alongside of the stilling basin. Silt carried by the stream and caught in the roller will be deposited in a bar parallel to the lock wall immediately downstream from
Fig. 28. Original stilling basin operating under varying pond level. Discharge represents 6,000 cubic feet per second through two tainter gates, pond at stage of 8.7 feet, tailwater stage 2.0 feet.

Fig. 29. Original stilling basin as in Figure 28. Discharge 15,000 cubic feet per second through 4 gates, pond at stage of 14.1 feet, tailwater normal.
the boule pass. This formation can be seen on Figures 13, 14, and 15.

It is not unreasonable to expect that dredging of the deposit each year must be resorted to if the boule dam is to be used as a navigation pass.

The amount of silt deposited can be reduced by decreasing the quantity of water passing through the original stilling basin, which, under the conditions most favorable to the process—high head and low tailwater—could be done safely only by means of an additional stilling basin. It was thus proved quite conclusively that the original stilling basin was not satisfactory in performance even under the less trying scheme of operation, nor was it entirely an adequate protection against erosion below the dam. It was also apparent that, at this stage of the construction program, the only satisfactory solution to the problem would be the building of an additional stilling basin to carry a portion of the burden of operation. Construction of the new stilling basin was authorized immediately, and the tests were directed toward the development of a design which would meet satisfactorily all requirements under the proposed schedule of operation and under such deviations from that schedule as might later be found necessary.
XI. DEVELOPMENT OF NEW STILLING BASIN FOR VARYING POND LEVEL

56. Length of apron—The conditions imposed upon the proposed stilling basin were less severe than for the basin developed in the previous series of tests, and it was thought that the apron should not be appreciably shorter than that which proved amply safe with operation at full pond level. The time within which the design must be completed was short, and further investigations of a length of apron for the requirements of this stilling basin were not attempted. An apron 100 feet long was considered satisfactory. It only remained then to determine the proper elevation for the stilling basin floor.

57. Elevation of apron and final design—The table below shows the theoretical elevations of the stilling basin floor, or apron, required for a complete development of the hydraulic jump within the basin for various conditions under which it might be operated. These elevations are determined in the manner described under article 45.

<table>
<thead>
<tr>
<th>Discharge</th>
<th>Discharge</th>
<th>Amount</th>
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<tr>
<td>through</td>
<td>corresponding to</td>
<td>tailwater</td>
</tr>
<tr>
<td>stilling</td>
<td>Pond elevation</td>
<td>Tailwater elevation</td>
</tr>
<tr>
<td>basin</td>
<td>Feet</td>
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<td>694.0</td>
<td>10,000</td>
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<td>692.0</td>
<td>15,000</td>
</tr>
<tr>
<td>15,000</td>
<td>693.0</td>
<td>15,000</td>
</tr>
<tr>
<td>15,000</td>
<td>692.0</td>
<td>12,500*</td>
</tr>
</tbody>
</table>

A condition more serious than those enumerated in the table seems unlikely ever to occur, particularly in view of the fact that both the original and the new stilling basins will be in operation simultaneously. The last two conditions are apparently the most serious in the list. Tests were made on the new apron as developed in the previous series of tests, *A condition which would exist when the pond is being drawn down and before the tailwater has built up to normal.

http://ir.uiowa.edu/uisie/2
when set at elevation 668.4 and at elevation 667.4. Practically no difference in performance could be observed, both positions giving fully satisfactory results, and it was decided that a medium point would be an adequate depth. This stilling basin was designed in all details identical with the preceding one except that its floor was held at elevation 667.9 instead of 664.9. Detail drawings of the new basin are shown on Figure 26.

Figures 30 and 31 show the new stilling basin in operation
passing the total river flow, and on Figures 32 and 33 it is shown in operation passing one-half the total flow in the river, the balance assumed to be carried by the original stilling basin.

58. Location of new stilling basin—Construction of the masonry section of the Hastings lock and dam was carried out in two major steps. During the first step a single cofferdam enclosed the boule dam and a set of 6 adjoining tainter gates. This portion was practically completed at the time that the experimental results were obtained, and it became necessary
to open up the cofferdam to permit the spring flood to pass through. It was then too late to consider the construction of the new stilling basin adjacent to the original, however desirable this might have been from a practical as well as aesthetic viewpoint. The cofferdam enclosing the foundation for the remainder of the tainter gate section was constructed to permit the building of the new stilling basin downstream from gates 8 to 11 inclusive.

Fig. 34. Condition below new stilling basin designed for varying pond level after test in the small-scale model of the Hastings dam. Length 100 feet. Elevation of apron floor 667.4. Duration of test, 3 hours. Discharge for one hour, 15,000 cubic feet per second. Discharge for two hours, 20,000 cubic feet per second, tailwater corresponding to flow of 15,000 cubic feet per second.
59. Test on stilling basin in general model—A model of the new stilling basin was constructed at the proper location in the general model of the project and was tested through an entire season's operation, beginning with the winter low flow and carrying through into the spring flood. In this test both stilling basins were operated together, in the manner proposed for the full size structure, and there was no indication that the structure was not amply protected for any condition that can reasonably be expected to occur. On Figure 34 is shown the result of a severe test on the new stilling basin.

60. Attempted spreading of jet from stilling basin—A peculiarity observed in the behavior of both stilling basins, but slightly more characteristic of the newer one because of its greater depth of tailwater over the sill, was that the jet of water passing through the stilling basin tended to continue flowing downstream in a suppressed stream of comparatively high velocity rather than spreading out over a wider area of the stream bed at reduced velocity. The high velocity jet appeared to do no damage to the stream bed as long as the proper scheme of operation was adhered to, but if a spreading of the jet, and consequently a reduction of its velocity could be brought about, an additional measure of safety to the structure would have been gained. Several types of toothed sills designed to effect a divergence of the jet were tried out at the end of the apron with some success, but only at a sacrifice of uniformity in flow below the apron which tended to augment the condition that it was desired to avoid. Time did not permit investigating this problem further, although it was thought that a flaring apron could be devised that would accomplish the desired results.

61. Design of stepped sill for tainter gates—The tainter gates which do not discharge into either of the stilling basins are not intended to be operated at any time when the tailwater is not amply deep to protect the stream bed against scour. The proposed scheme of operation requires that these gates be brought into play at about 30,000 cubic feet per second discharge in the river, which should inflict no particular hardship on the toe protection under ordinary circumstances. If, however, it were necessary to operate these gates to sud-
denly lower the pool at low tailwater stage, or in the event that the stilling basin capacity should be impaired through accident, or otherwise, the protection provided below them might prove to be seriously inadequate. The cost of providing full protection against every emergency for all the gates did not appear to be economically justified, but experiments demonstrated that considerable further protection could be obtained at moderate cost by constructing a stepped sill at the lower edge of the tainter gate footing. Two types of square sills were tested in the general model in addition to

![Diagram](http://ir.uiowa.edu/uisie/2)

Fig. 35. Effect on downstream velocity conditions of stepped sill below those tainter gates outside of stilling basin.

the stepped sill; the latter appeared to function most favorably. The sill was constructed with two steps, each a foot high on treads 1.5 feet wide. It was also tested in the large-scale model, where a study was made of the velocity distribution in three vertical sections below the model, with and with-
out the stepped sill in place. The velocities were measured by means of a small Ott current meter. The results are plotted on Figure 35, showing how the sill throws the water at high velocity into the upper part of the stream instead of along the bottom of the channel, as is the case when no sill is used. The sill prevents the scouring jet from striking into the toe protection, and tests on the general model indicated that the erosion along the toe of the dam was not likely to occur even under extremely careless and unusual operation. It may be pointed out that the negative velocity occurring along the surface of the stream in section one, referring to Figure 35, when no sill was used, was due to a surface back roller at the point where the hydraulic jump was formed.
XII. MISCELLANEOUS STUDIES

62. Dredging at east end of spillway section—Reference was made earlier in this paper to a sand and gravel bar at the east bank of the river, which intersects the structure about 175 feet from the abutment. In forming the model river channel it was necessary to excavate to the elevation of the tainter gate sill in order to permit the water to reach the gates located in the bar. This experimental dredging can be seen on Figure 6. The gates which are affected will normally not be in operation except at floods of 35,000 cubic feet per second and over. In the course of the experimental work it was found that, with the entire spillway section open and passing a flood discharge, an eddy was formed upstream from the section placed in the sand bar. The roller moved upstream along the east bank of the cut and under certain conditions there was actually a reverse flow through the two end gate openings. In an attempt to improve the capacity of this section of the spillway by more smoothly guiding the water toward the gates, some additional dredging was done upstream from the dam. A slight improvement in the character of flow through the section was effected, the roller became less widespread, yet the gates in question did not perform with the same efficiency as did those in the main channel of the river. It is believed that in order to fully correct this condition it will be necessary to dredge out the bank upstream and downstream for several hundred feet to the full width of the spillway section. The channel conditions assumed in the model and the recommended dredging are shown on Figure 36.

63. Study of currents near the structure—It was one of the purposes of the Hastings model tests to study current conditions about the structure and particularly any appearance of disturbance in the flow which might affect the maneuvering of a tow in the approaches to the lock and to the boule pass. During the experimentation on the general model many varied conditions in operation were brought about, and at all times the regimen of flow was kept under careful surveillance.
Fig. 86. Extent of dredging carried out at east end of dam in the model, with recommended dredging (shown by broken contours) at Hastings dam to create proper flow conditions through all gates.
It was observed that, as long as the entire discharge was carried through the tainter gate section, there were no serious or objectionable cross currents in the lock approaches. Eddies formed above and below the lock but these caused a very slow rotary motion of the water and were thought to be unimportant.

When the normal flow in the river exceeds about 50,000 cubic feet per second, and the lock and boule dam are open in contemplation of their use as navigable passes, the velocity of the water flowing through them and the waves caused in flowing around the points of the lock wall and the boule pier create a condition that is believed to be too treacherous for successfully maneuvering a tow through the openings. Undoubtedly, it will be necessary to operate the lock whenever any shipping craft except the very smallest are to be passed through the Hastings structure. During extreme floods, backwater conditions may require the lock and boule dam to be kept open, in which case navigation will be curtailed for the period of high water, unless the lock gates are designed for operation against a head equal to the extreme swell head of the structure which amounts to about 1.5 feet. Figure 37 shows the method used in photographically recording current studies by floating sawdust on the water surface.

Fig. 37. Currents below Hastings dam. Discharge 20,000 cubic feet per second equally divided between the two stilling basins. Pond at stage of 12.0 feet, tailwater normal. Sawdust on water surface indicates current tendencies.
64. Calibration of Hastings dam—The general model of the lock and dam was exceptionally well adapted for making experimental determinations of the hydraulic capacity and head losses through the dam, and tests were made to obtain such data for all conditions of flow and operation that are likely to occur in practice.

The tainter gates in the general model were calibrated by fours; that is, successive tests were made with 4, 8, 12, 16, and 20 gates open. The results are shown on Figures 38 and 39. The 4 and 8-gate calibrations were made on the stilling basin sections. Two tests were made on 8 gates. One test was made on the original stilling basin section plus 4 additional adjacent gates, while the other test was made on the original and the new stilling basins which were at that time separated by two closed tainter gates. The capacity of
the latter was slightly less than where the eight gates were operated in an unbroken series.

Capacity tests on the general model for 4 and 8 gates were checked on the large-scale model in the glass flume. The results shown on Figure 38 indicate a remarkable agreement in performance of these two models of greatly different scale ratios, which leads to the belief that the calibrations can be applied also to the full-size structure with reasonable assurance of accuracy.

In all calibration tests the desired quantity of discharge was admitted into the model over the measuring weirs, and the tailwater elevation corresponding to that discharge, according to the natural rating curve, was created by regulating the tailwater control gate. When conditions in the model became constant, the elevation of the water surface of the

Fig. 39. Calibration curves of 12, 16, and 20 tainter gates.
pond was read on the point gage. All units outside of the section which was being calibrated were sealed to prevent leakage. The same procedure was used in the calibrations on the glass-flume model as on the general model.

65. Operation schedule—For the purpose of expediting the development of an operation schedule for use at the Hastings dam a test was made to determine the gate apertures required to pass certain quantities of discharge while the pond was maintained at the elevation required to ensure the project depth upstream. This test was made on the large-scale model in the glass flume.

On Figure 40 is shown the operation schedule recommended for the Hastings lock and dam. The scheme is, of course, subject to variation and changes that are dependent upon conditions which cannot now be predicted, such as the amount of the backwater effect of the discharge from the St. Croix river, whether the stage at St. Paul is rising or falling, the rate of change in stage at St. Paul, etc. It is thought, however, that this schedule will serve a useful purpose in aiding the construction of a more practical program of operation that will naturally develop itself during the first few seasons in which the Hastings project is in service.

The following table is submitted to supplement and clarify the graphical presentation of the proposed scheme of operation.
Fig. 40. Operation schedule recommended for the Hastings dam.
<table>
<thead>
<tr>
<th>Stage at St. Paul</th>
<th>Discharge</th>
<th>Pond stage at Hastings</th>
<th>Normal tailwater stage at Hastings</th>
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</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Gates open</td>
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<tr>
<td></td>
<td></td>
<td></td>
<td>8</td>
<td>1 to 4</td>
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<td>5</td>
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<td>13</td>
<td>to 7</td>
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<td>17</td>
<td>to 16</td>
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66. Photographs—Results of the model experiments on the Hastings lock and dam were recorded to a large extent photographically. Numerous photographs were taken of the models and testing apparatus, and practically every step taken in the process of developing the design of the new toe protection was photographed. About 700 feet of motion-picture film were taken of erosion tests and currents, where the water action was the interesting feature, and its behavior could not be recorded by a still picture. Many of the photographs are included as a part of this paper. All the photographs and motion picture films are available for reference at the District Engineer's office at St. Paul.