HYDRAULIC STUDIES OF A HIGHWAY BRIDGE

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HYDRAULIC STUDIES OF A HIGHWAY BRIDGE

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ABSTRACT

This report describes an investigation to determine the most efficient remedial measures toward prevention of scour due to periodic flooding of the Wabash River at Bridge 3 on the Illinois State Bond-Issue Highway Route 139 between Crossville, Illinois, and New Harmony, Indiana.

The investigation includes mainly the collection and analyses of field data and laboratory studies of a 1:50 scale hydraulic model. Results of the investigation reveal a most advantageous remedial measure which consists of (1) removal of timber from the upstream wooded island, (2) proper installation of parallel, permeable, stepped dikes on the upstream side of the bridge, (3) replacement of pile bents by piers, and (4) construction of a cut-off wall across the upstream end of the scour hole on the downstream side of the bridge. It is believed that this remedial measure can also be applied to other similar problems.
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NOMENCLATURE

A = area in sq. ft.

Am = area in sq. ft. in the model.

Ap = area in sq. ft. in the prototype.

D = diameter of pipe in ft.

d = depth of flow in ft.

dm = depth of flow in the model.

dp = depth of flow in the prototype.

F = Froude number.

Fm = Froude number of the model.

Fp = Froude number of the prototype.

g = gravitational acceleration of approximately 32.2 ft. per sec.

L = length in ft.

Lr = scale ratio.

nm = Manning's roughness coefficient of the model.

np = Manning's roughness coefficient of the prototype.

Q = discharge in cfs.

Qm = discharge in cfs. in the model.

Qp = discharge in cfs. in the prototype.

R = Reynolds number.
\[ R_m = \text{Reynolds number of the model.} \]

\[ R_p = \text{Reynolds number of the prototype.} \]

\[ S = \text{hydraulic gradient.} \]

\[ V = \text{velocity in ft. per sec.} \]

\[ V_m = \text{velocity in ft. per sec. in the model.} \]

\[ V_p = \text{velocity in ft. per sec. in the prototype.} \]

\[ V_{ol.} = \text{volume in cu. ft.} \]

\[ V_{ol.m} = \text{volume in cu. ft. in the model.} \]

\[ V_{ol.p} = \text{volume in cu. ft. in the prototype.} \]

\[ \nu = \text{kinematic viscosity in sq. ft. per sec.} \]
I INTRODUCTION

1 The Problem

The structure under investigation is the New Harmony Bridge No. 3, abbreviated hereafter as Bridge 3. It is one of the five overflow bridges located near New Harmony, Indiana, on the Illinois State Bond-Issue Highway Route 139 that links New Harmony, Indiana, and Crossville, Illinois (Fig. 1). The overflow bridges have been providing five flood openings along the earth embankment of the highway across the flood plain west of the Wabash River. The embankment, about 3.5 miles long and 20 to 25 feet high, is built of materials taken from a continuous borrow pit along the south side of the right-of-way. During flood periods the function of the overflow bridges is to permit continuous movement of flood waters over the flood plain to relieve excess flood load from the main channel. Except at Bridge 3, the performance of these venting structures has been proved to be fairly satisfactory.

The location of Bridge 3 is about one mile west of the main bridge across the Wabash River. This is a three-span girder bridge supported by piers built of concrete-capped piles and by abutments which have sloping aprons to protect the fill. The clear opening of the bridge is 137 feet measured between the tops of the abutments. After about eighteen years of service a deep hole, known as the "blue hole,
was developed on the downstream side of the bridge by the scouring action of successive flood waters. The size and depth of the hole have grown to such an extent that the bridge structure and approach fills are seriously endangered.

2 Situation Leading to Investigation

Since the completion of the bridges in 1933, scouring action at Bridge 3 was first observed in the February of 1937. It was noticed that the scour had developed at the immediate downstream side of the bridge and near the sloping apron of the east abutment. The situation was very critical, but prompt measures taken by the Illinois Division of Highways had saved the pile footings and abutments from a complete failure although the downstream sloping apron of the east abutment was unfortunately lost.

After the subsidence of the flood waters it was found that the scour hole had encroached to within 6 feet of the nearest pile bent and some scour had taken place under the bridge. There was evidence that the alert action by the Division of Highways forces had prevented the structure from serious damage. To avert further encroachment, sufficient rubble and old street paving were dumped under the bridge and along the north edge of the scour hole for a distance of 16 feet from the pile bent. This stabilization work had very likely stopped an otherwise almost irreparable damage at a later date.
In 1945 there was a flood in the Wabash River. A survey after the subsidence of the flood disclosed a hole about 250 feet in diameter on the downstream side of the bridge. The deepest portion of the hole, located about 120 feet from the nearest pile bent, was 35 feet lower than the ground surface under the bridge and 4 feet lower than the bottom of the bridge piles. Figure 2 shows the location of the hole and the profiles through the hole as they were observed in 1945 and in 1950. It will be noted that during this period the hole had been considerably deepened and enlarged. Movement toward the bridge was not great, however, undoubtedly because of the earlier stabilization efforts of the Division of Highways.

The volume of the hole below elevation 360 which is the average elevation of the hole periphery was about 29,500 cu. yd. in 1945. In 1950, the volume of the hole below elevation 360 was over 42,300 cu. yd., an increase of nearly 12,800 cu. yd. over the 1945 volume or an average of about 2,560 cu. yd. of material removed per year. Figure 3 shows the volume of the material removed from the hole as plotted against the date. By extending the line joining the two plotted points backward in time, it can be seen that the line intersects the abcissa for zero volume of the removed material at a date of 1933 which is the date of completion of the bridge. This, however, seems to be just coincidental. There is no doubt
that the removal of the material will continue under the present condition. The plot shows a trend which indicates that the complete failure of the bridge structure can be expected in a very near future. The situation has developed to such an extent that a thorough investigation of the problem is definitely necessary. Such investigation may lead to the development of a remedy that will save the structure from a complete failure and avoid costly maintenance in the future.

3 Objective of Study

The objective of the present investigation was twofold: (1) to ascertain the cause of the excessive scour downstream from Bridge 3, and (2) to develop a practical, reliable, and economical method of eliminating the existent dangerous scour condition.

4 Scope and Method of Study

The scope of the present study covers the following phases:

(1) Collection of field data, including flood records, topographical information, location and dimensions of structures, and other related information.

(2) Hydrologic analysis of data, including construction of hydrographs and rating curves and flood frequency analysis.

(3) Laboratory investigation, including design and construction of a hydraulic model, instrumentation and verification of the model, and
testing. The testing covers the original calibration of the model and revision tests on the model modified for various remedial measures. Silt tests were also conducted to determine the scour condition.

(4) Study of remedial measures to determine a suggested solution for the problem.

The general method of study consists of analyzing the available data by both hydrologic and hydraulic approaches and determining the solution of the problem by laboratory investigation of a fixed-bed, undistorted hydraulic model.

5 Principal Findings

The present investigation leads to the following major findings:

(1) Cause of the blue hole.

(2) Flood situation at Bridge 3.

(3) A sound solution of the problem based on factual field data and laboratory investigation.

(4) Feasibility of a laboratory investigation and applicability of the results to other similar problems.

6 Acknowledgments

The studies reported in this publication were entitled Project IHR-11, "Hydraulics of Flow at Bridges," a cooperative investigation under the Illinois Cooperative Highway Research Program. This
investigation was conducted by the Hydraulic Engineering Staff of the Department of Civil Engineering and was sponsored by the Division of Highways of the State of Illinois.

The following persons were responsible for the general administrative supervision of the projector at the time the investigation was being carried out:

For the University of Illinois:

Dean W. L. Everitt, Director of the Engineering Experiment Station.
Professor W. C. Huntington, Head of the Department of Civil Engineering.
Ellis Danner, Professor of Highway Engineering, in charge of the Illinois Cooperative Highway Research Program.

For the Division of Highways:

F. N. Barker, Chief Highway Engineer.
W. L. Esmond, Engineer of Research and Planning.
W. E. Chastain, Sr., Engineer of Physical Research.

The direct supervision and technical direction of the investigation were provided by Professor J. J. Doland, Project Supervisor and Professor John C. Guillou, Project Investigator.

Technical advice and suggestions for work programming were provided for the studies covered by this report by a Project Advisory Committee consisting of the following members:

J. J. Doland, Professor of Hydraulic Engineering, University of Illinois.
W. L. Esmond, Engineer of Research and Planning, Illinois Division of Highways.
R. W. Hansen, Bridge Engineer, Illinois Division of Highways.
Thomas H. Thornburn, Research Professor of Civil Engineering, University of Illinois.
R. B. Walters, District Highway Engineer, Illinois Division of Highways.

The laboratory phases of the investigation including design, construction and testing, the preparation of a preliminary report as well as the supervision of assisting personnel was the responsibility of Mr. Irby J. Hickenlooper, Research Associate in Civil Engineering. A large portion of the laboratory work was capably performed by Mr. Mulford Martin, Engineering Draftsman, with the aid of Messrs. D. G. Rinck, H. E. Hofstetter, and G. A. Shunneson, students in the Department of Civil Engineering and R. T. Henry, student in the Department of Mechanical Engineering.

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II ANALYSIS OF THE PROBLEM

7 Field Data Available

The field data available for the analysis of the present problem are as follows:

(1) Stream Flow Data The data include stream flow measurements made from 1935 to 1950 by the Louisville District Office of the Corps of Engineers for river stages exceeding 18 ft. at the New Harmony gage. The gage is located near the main bridge span across the Wabash River, a mile east of Bridge 3. These measurements include discharges and mean and maximum velocities of flow through Bridge 3, and stage data at the New Harmony gage during the time of measurement.

(2) 1944 Flood Data These include water surface elevations at 38 reference points along the length of the highway embankment and a map showing spotted velocity observations and directions of flow for the high water of April 1944 as observed by the Corps of Engineers.

(3) 1950 and 1951 Flood Data These data were observed by the University personnel. They include:

(a) Upstream and downstream water surface elevations at Bridge 3 obtained during high waters of 1950 and 1951.
(b) Photographs of the problem area during the high water of 1950 and after the subsidence of the high water (Fig. 4). These photographs include not only the views taken during different periods of high water and the pictures of topographic details, but also views of sediment deposition that has taken place at or near the edges of the scour hole, and the view of the upstream slope of the west approach fill.

(c) Observations of surface flow patterns during high water periods of 1950 and 1951, and observations of significant topographic features in areas contiguous to Bridge 3.

(4) Scour Contours These include a contour map of the blue hole during the summer of 1945 and a contour map showing the highway embankment and areas contiguous to the scour hole in 1950. Both maps were prepared by the Illinois Division of Highways.

(5) Bridge 3 Design Drawing This drawing was prepared by the Illinois Division of Highways, showing construction and dimensions of the Bridge 3 structure.

8 Hydrographs for the 1943 Flood

Using the flow data collected by the Corps of Engineers, hydrographs in semi-log plots (Fig. 5) were prepared for discharges through Bridge 3 and Bridge 4 and for the total discharge through the flood.
plains as well as the main channel during the flood of May 1943. These hydrographs cover stages only greater than 18 ft. at the New Harmony gage, since for lower stages the data are not available. It was noted later in the study that the peak discharge through either Bridge 3 or Bridge 4 was less than 5 per cent of the total. The average discharge per foot of the bridge length at the peak of the flood, however, was found to be 112.1 cfs for Bridge 3, 89.7 cfs for Bridge 4, and 73.3 cfs for the main channel bridge. The highest discharge per foot of the bridge at high water stages is probably a major factor responsible for scouring at Bridge 3.

9 Rating Curves

Using the hydrograph data of 1943 at Bridge 3 as described in the preceding article, a mean rating curve was prepared as shown in Fig. 6A. In plotting this curve, all discharges at Bridge 3 were plotted with reference to the New Harmony gage and an average curve was drawn for these points. This average curve represents the mean rating curve at Bridge 3 with reference to the New Harmony gage. Similarly, the mean rating curve for the total flood discharge is also shown in Fig. 6B for the sake of reference.

In order to obtain a rating curve at Bridge 3 with reference to the upstream and downstream stages, use was made of the Corps of
Engineers flood surface profile data for April 19-22, 1944 and the Illinois Division of Highways data for January 19, 1950. Thus, the water surface elevation at the New Harmony gage can be plotted against the water surface elevations at downstream and upstream sides of Bridge 3 as shown in Fig. 7. The high and low pairs of the plotted points in Fig. 7 are from data obtained in 1950, and the middle pair of points are from data obtained in 1944. The 1950 data are believed to be more suitable for the present study since they were obtained after the major topographical changes had occurred. From the rating curve of Fig. 6A for Bridge 3 with reference to the New Harmony gage and the stage relationship of Fig. 7, the rating curve for Bridge 3 with reference to the upstream and downstream stages at Bridge 3 can be readily obtained as shown in Fig. 8.

10 Cause of the Blue Hole

Field data and observations indicate that the major cause of the scour at Bridge 3 was the curvilinear nature of the flow through the bridge at a relatively high discharge per unit length of the bridge. A large portion of the flow approached the bridge in a direction parallel to the roadway embankment. When the bridge opening was reached, the flow turned sharply toward the opening, moved across the curved shoulder of the abutment apron, and plunged toward the downstream
side of the abutment, resulting in great turbulence. The approach flow in the form of a prism moved freely near and in parallel to the embankment for a distance of 150 ft. on each side of the bridge opening. The flow was actually divided into two main segments by a timbered area near to and upstream from the bridge opening. These main segments of flow approached the bridge, met each other at angles under and downstream from the bridge, and thus created turbulence and hydraulic forces which are responsible for the scour.

As mentioned previously, a continuous borrow pit was made on the south side of the roadway. Consequently, the relatively stable, vegetated topsoil was stripped off the borrow pit area and a more easily erodible surface was exposed. In the area immediately downstream from the bridge structure, the exposed erodible surface was apparently not sufficiently resistant to the hydraulic forces of the turbulence created by the converging stream flowing toward the Bridge 3. As a result, scour in the area became aggravated.

The hydraulic forces of the turbulent flow have developed scour and eventually caused the formation of the blue hole. Coarse material scoured from the hole was carried away and deposited at the downstream side of the hole. The size of the scoured material ranges from fine sand to rubble of ten or twelve pounds in weight. Much of the
heavier material consists of paving bricks or brick fragments which had been used for stabilization at the bridge and has been found to deposit as far as 350 ft. downstream from the structure. As would be expected, the finer material was carried away and deposited farther from the edge of the hole, and the extent of the deposition was abrupt and distinct. Deposition of various types of material can be seen in Photos 3, 4, and 5 of Fig. 4.

11 Related Information

(a) Comparative Flow Condition at Bridge 4 Bridge 4 is located about 3,600 ft. west of Bridge 3. As far as the size and construction are concerned the two structures are practically identical. However, little scour has been experienced at the site of Bridge 4. The difference in scouring condition may be due to the difference in flow characteristics. At very high flood stages the discharges through Bridge 4 have been found to be less than those through Bridge 3; but for moderate flood stages, they were greater. The approach area to Bridge 4 is free of trees; whereas the approach immediately upstream from Bridge 3 is timbered area which divides the flow, resulting in converging currents and the accompanying turbulence. By comparison with the flow condition at Bridge 4, it seems that the high discharge at peak stages and the turbulence resulting from the existence of the
timbered area may have been the major responsible factors causing the scouring at Bridge 3.

(b) Reference to the Operation of the Mississippi River Floodways Reference is made to the operation of the Mississippi River Floodways in which the formation of a sizable scour hole at a levee crevasse is a normal and usual occurrence. The process of forming the scour hole is as follows: (1) a break occurs in the levee, (2) a hole is scoured on the downstream side of the levee crevasse, (3) the width of crevasse is enlarged, (4) the hole is enlarged by scour on the unstable upstream edge of the hole, and (5) the upstream edge of the scour is moved upstream through the levee and onto the adjacent land. (See Ref. 1 in Appendix 4)

It is noted that the opening at Bridge 3 is similar to a crevasse in a levee with paved side slopes. At Bridge 3, the paved toe of the roadway approach fill prevents a widening of the opening and the huge quantities of rubble placed under the bridge for maintenance have stabilized, at least temporarily, the floor of the opening and thus are retarding the rate of progression of the hole toward the bridge. If the normal development of a crevasse were allowed at Bridge 3, the hole would have moved upstream through the bridge opening. Since the elevation of the bottom of the hole is 10 feet below the elevation of the
bottom of the 50-foot piling, such a development would undoubtedly be
disastrous to the bridge.

(c) Reference to the Scouring at Mitchell Creek Bridge The
Mitchell Creek Bridge is on the Federal-Aid Secondary Route No. 15A
between Cowden and Herrick, Illinois. The bridge is 130 ft. long. It
has open-abutments and a three-span structure of steel beams con­
structed on top of concrete-capped column bents. The bents are sup­
ported below the stream floor on 24-ft. long, concrete-capped wood
piles. When the bridge was first completed, the stream floor elevation
was about 3 ft. above the concrete cap of the piles. Since then, scour
has occurred and a blue hole is formed. The present elevation of the
hole bottom at the center bent is about 8 ft. below the cap of the piles.
The deepest portion of the hole is found directly below the bridge at a
depth of about 12 ft. above the base of the piles.

12 Discussion of Various Suggestions for Improvement

Several suggestions have been made proposing remedial
measures to halt scour at Bridge 3 other than those to be tested in the
laboratory. These suggestions deserve an opinion, and thus a brief
discussion of the proposed measures are given as follows:

(1) Natural Filling of the Blue Hole It has been questioned
that whether or not the blue hole may be filled up by natural deposition
over a period of years after the remedial measure to stop scour has been made effective. However, the possibility of such natural siltation seems to be extremely unlikely if not impossible. The possibility arises only due to the fact that the lower portions of the blue hole will be filled by sloughing or shifting of material from higher areas of the hole. Actual filling of the hole by deposition of upstream material is hardly possible.

With reference to the flood of March 1944 as recorded by the Corps of Engineers the velocity of the flood plain flow at about 500 ft. upstream from Bridge 3 was 1.1 ft. per sec. When the depth of water was 6 ft. the velocity was less than 2.0 ft. per sec. in most locations over the flood plain. As flow approached Bridge 3, the velocity increased to a maximum at the bridge opening. Velocities in the hole were well in excess of 2.0 ft. per sec. and would continue to be so even if the remedial measure were made effective. Any material carried to the bridge site by flood plain velocities less than 2.0 ft. per sec. will be carried on through or across the hole to be deposited, if at all, on the flood plain well downstream from the hole.

(2) Artificial Filling of the Blue Hole To completely fill the scour hole, as has been suggested, without recourse to further remedial measures, would merely start again the cycle of the process of
blue-hole formation. Due to lack of compaction, cohesion, and vegetated overburden of the filling material comparable to natural ground conditions, it is to be expected that the blue-hole process would recur at an increasing rate. And the lack of the natural resisting forces to scour would only make the situation worse.

(3) Closing the Bridge Opening Consideration has been given to the feasibility of completely closing Bridge 3 and eliminating it entirely as a venting structure. It is believed that such action would increase the water load of Bridge 4 to such an extent that the blue-hole process would very probably start at the site of Bridge 4 and would progress at a more rapid rate than has been manifested at Bridge 3.

13 Approach by Hydraulic Model

After observation of the prototype condition during the flood of January 1950, it was believed that the most reasonable way of studying the problem would be the use of a hydraulic model. The hydraulic model is a system, similar to the prototype system in certain exact relationships, which for present purposes could be altered so as to predict future action of the prototype from the model. The hydraulic model may be classified into two general types with respect to bed-load movement; namely, the fixed-bed model and movable-bed model. Either of the two types may be further described as distorted scale or
undistorted scale. Generally speaking, however, a movable-bed model requires a distorted scale. (For further information see Ref. 2.)

The procedure to be followed before designing a model for the Bridge 3 problem was to assemble all prototype data and then to determine the feasibility of construction of a distorted, movable-bed model. After examining the data available for this problem, however, it became apparent that information available to verify the distorted, movable-bed model within a necessary limit of accuracy was insufficient. Furthermore, the cost of a movable-bed is very high, several times that of a fixed-bed model, due mainly to the large amount of verification time required and the vastly increased testing effort. In the present problem it was apparent that the advantages of a movable-bed model would not justifiably warrant the high cost involved. On the other hand, the data obtained from a fixed-bed model would be quite satisfactory for developing a solution of the present problem. It was expected also that the tests of a fixed-bed model would yield a clear picture of the hydraulic conditions at Bridge 3. In other words, flow patterns, velocities, and scouring capacities of flood waves as observed from the model would be sufficient to indicate the present condition of the structure as well as the condition after any remedial measure has been made. It was further believed that after the laboratory
investigation by hydraulic model was completed a sound basis for
analysis and comparison of the hydraulic advantages of various reme-
dial measures could be obtained, and such information should be ap-
pllicable also to other similar problems.
14 Design of Model

As mentioned previously, a model may be designed as either a movable-bed model or a fixed-bed model. Owing to the meagerness of field data and the increased cost and effort for the construction of a movable-bed model, a fixed-bed undistorted model was adopted for the use of the laboratory investigation. The design of the model is based on the conditions described below:

(1) The natural stream flow is generally turbulent. In order to obtain similarity in a model of turbulent flow it is necessary that the model flow also be turbulent. By the principle of fluid mechanics, this means that the Reynolds number

\[ R = \frac{VL}{\nu} \]  

be greater than a critical value. In this equation, \( V \) is the velocity in ft. per sec., \( \nu \) is the kinematic viscosity of water, approximately \( 10^{-5} \) sq. ft. per sec. on the average, and \( L \) is a characteristic length in ft. For flow in pipes, the pipe diameter \( D \) is taken as the characteristic length, and the upper critical Reynolds number is about 3,200 depending on the roughness and pipe diameter. For flow in wide open channels, the depth of flow \( d \) is taken as the characteristic length. The depth of
flow in a wide open channel is equal to the hydraulic radius, while the
diameter of a pipe is equal to four times its hydraulic radius. Therefore, for the same hydraulic radius, the corresponding upper critical
Reynolds number for flow in wide open channels is about \( \frac{3,200}{4} \) or
800. The flood-plain channel under consideration has a relatively
shallow depth as compared to its width, and hence it can be considered
as a wide open channel. Consequently, in order to maintain a turbulent
flow in the model channel, it is required that

\[
\frac{R}{V} = \frac{V_{m}d_{m}}{V} > 800
\]  

(2) in which the subscript \( m \) indicates the model condition.

(2) The flow at Bridge 3 is influenced primarily by forces of
gravity and inertia. By the principle of fluid mechanics, the Froude
number of the simulating model should be equal to that of the proto-
type. The Froude number is defined as

\[
F = \frac{V}{\sqrt{gL}}
\]

in which \( V \) is the velocity of flow in fps, \( g \) is the gravitational accel-
eration which may be taken as 32.2 ft. per sec. per sec., and \( L \) is a
characteristic length or the depth of flow, \( d \), in case of channel flow.

Using subscripts \( m \) for model and \( p \) for prototype, it is required that
\[
\frac{F_m}{F_p} = \frac{F}{F_p}
\]  

or

\[
\frac{V_m}{\sqrt{gd_m}} = \frac{V}{\sqrt{gd_p}}
\]

At Bridge 3, the minimum depth and velocity of flow upstream from the bridge are respectively \(d_p = 2\) ft. and \(V_p = 1.5\) fps. Substituting these values in Eqs. 2 and 5, the following are obtained

\[
V_m \frac{d_m}{V_p} > 800 \times 10^{-5}
\]

and

\[
\frac{V_m^2}{d_m} = \frac{1.5^2}{2}
\]

Solving Eqs. (6) and (7), \(d_m > 0.0385\) ft. Therefore, from the hydraulic point of view, the scale ratio of the model should be about \(2/0.0385 = 52\).

The laboratory facilities available for the construction of the model are: A space of 21 ft. by 31 ft., a maximum rate of flow \(Q_m = 5\) cfs., and a calibration capacity for flow of \(Q_m = 1\) cfs. It was also desirable that the model include an area of 400 to 500 ft. to each side of the bridge centerline along the roadway and 600 to 800 ft. of the approach areas for simulating the prototype flow at the bridge and blue
hole. With all these considerations, an undistorted model scale ratio of 1:50 was adopted.

After the scale ratio is decided, the relationships between the model and prototype dimensions may be computed by the following formulas, and the computed values are shown in Table 1. In these formulas, \( L_r \) represents the scale ratio, being equal to 50, and subscripts "m" and "p" indicate respectively the model and prototype conditions.

### Length

\[
d_p = d_m L_r = 50 \, d_m \quad \text{or} \quad d_m = \frac{d_p}{50}
\]

### Area

\[
A_p = A_m L_r^2 = 2500 \, A_m \quad \text{or} \quad A_m = \frac{A_p}{2500}
\]

### Volume

\[
Vol_p = Vol_m L_r^3 \quad \text{or} \quad Vol_m = \frac{Vol_p}{125,000}
\]

### Velocity

\[
V_p = V_m L_r^{0.5} = 7.07 \, V_m \quad \text{or} \quad V_m = \frac{V_p}{7.07}
\]

### Discharge

\[
Q_p = Q_m L_r^{2.5} = 17,680 \, Q_m \quad \text{or} \quad Q_m = \frac{Q_p}{17,680}
\]

### Roughness

\[
n_p = n_m L_r^{1/6} = 1.92 \, n_m \quad \text{or} \quad n_m = \frac{n_p}{1.92}
\]
Derivations of the preceding equations are given in Appendix 1.

15 Model Construction

A hydraulic model of the problem under investigation was built in the Hydraulic Engineering Laboratory of the University. For creating a space for the model, a monolithic reinforced concrete retaining wall 21 in. high and 4 in. thick was first laid directly on the laboratory floor. For waterproofing purposes, a chemical compound was used in the construction. At both upstream and downstream of the model, a low weir wall at each end was constructed at a position 15 in. inside the retaining wall. At the upstream end, the purpose of the wall was to provide a stilling basin for insuring an even distribution of the flow supply. At the downstream end, the wall formed a discharge channel leading to a 90° V-notch weir where the rate of flow was to be measured.

The model proper was constructed of concrete in courses and in two layers. The concrete for each course of the bottom layer was placed in an area of one or two feet wide and as long as the width of the model. The placing of concrete was made from both the upstream and downstream ends of the model and progressed toward the center section where the model bridge and roadway were to be located later. The bottom layer consisted of an inexpensive but very stable mix of
Table 1 Relationships between model and prototype dimensions

<table>
<thead>
<tr>
<th>Item</th>
<th>Prototype value</th>
<th>Model value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$L_r$</td>
<td>50</td>
<td>1</td>
</tr>
<tr>
<td>Total width, ft.</td>
<td>1,000</td>
<td>20</td>
</tr>
<tr>
<td>Total length, ft.</td>
<td>1,400</td>
<td>28</td>
</tr>
<tr>
<td>Min. depth upstream from bridge, ft.</td>
<td>2</td>
<td>0.04</td>
</tr>
<tr>
<td>Min. velocity upstream from bridge, fps.</td>
<td>1.5</td>
<td>0.212</td>
</tr>
<tr>
<td>$R = \frac{Vd}{\gamma}$</td>
<td>$3 \times 10^5$</td>
<td>848</td>
</tr>
<tr>
<td>Max. Q, cfs.</td>
<td>17,000</td>
<td>0.962</td>
</tr>
<tr>
<td>$n$</td>
<td>0.025</td>
<td>0.0130</td>
</tr>
<tr>
<td></td>
<td>0.030</td>
<td>0.0156</td>
</tr>
<tr>
<td>Bridge opening c. to c. of pile cap, ft.</td>
<td>144</td>
<td>2.88</td>
</tr>
<tr>
<td>Single span length, ft.</td>
<td>48</td>
<td>0.96</td>
</tr>
<tr>
<td>Square pile dimension, ft.</td>
<td>1.33</td>
<td>0.027</td>
</tr>
<tr>
<td>Blue hole depth below elevation 362</td>
<td>42</td>
<td>0.84</td>
</tr>
<tr>
<td>Crown of road to bottom of blue hole</td>
<td>65</td>
<td>1.30</td>
</tr>
</tbody>
</table>
1:7 sand-cement mortar, and it was poured to within about 2 in. of the desired model surface. The top layer consisted of a 1:4 mix of Zonolite concrete, and it was placed as nearly to the grade as possible after the bottom course had set sufficiently to provide a firm base. Zonolite concrete is a light-weight aggregate concrete composed of expanded mica aggregate and Portland cement. This material was used for the construction of the top layer because it can be easily molded and shaped to reproduce required geometrical features of the prototype with satisfactory exactness.

The surface of the top layer was placed approximately to grade by a movable form board which was set to the desired elevation by means of a dumpy level. As soon as the top layer had set for a sufficient period of at least 15 hours, contours were drawn to simulate those of the prototype. The top layer was placed to a thickness a little in excess of that required so that by using a disc sander and a plane the model surface could be cut to exactly the required grade. The entire surface of the model was cut to within ±0.005 ft. of the model elevation values.

The water supply for the model testing was provided by a constant head tank whose skimming surface was nearly 12 ft. above the model surface under the model bridge. Water from the constant head
tank was led through an 8-inch supply line to an 8-inch control valve near the upstream end of the model. From the valve the flow continued through an 8-inch header pipe which extended over the model and through 6-inch tees and reducers to three 4-inch downcomers. Each downcomer was fitted with a 4-inch tee at its lower end. The tee was connected to two 3-foot-long nipples which provided an even distribution of flow to the model supply channel. The outflow from the model was pumped back to the head tank by centrifugal pumps.

Having decided upon the model scale ratio, the prototype bridge dimensions and ground elevations were converted to model values according to the information from the bridge drawing and contour map prepared by the Illinois Division of Highways. The bridge, bridge piling, and pile caps were made to scale of wood embedded in the Zonolite surface in the location indicated by the prototype data.

With the aid of the photographic data obtained in the field, the topography and vegetation of the prototype were simulated. In the vicinity of the bridge and hole, trees six inches and over in diameter were simulated by using dowel pins of proper diameter as determined by the scale ratio. Smaller trees and brush were simulated by means of hardware cloth which is cut into strips, coated with cement paint to give roughness and body, and placed in accordance with the prototype
topography. The roughness of corn stubble fields over which a major portion of the water approaches the bridge was assumed to have a Manning's roughness value of $n_p = 0.025$ which corresponds to a model value of $n_m = 0.013$. The roughness of the light-aggregate concrete surface seemed to be very close to this value. However, some additional model roughness was added to simulate the variability of the prototype roughness.

16 Instrumentation

Rates of flow through the model were measured by means of a $90^\circ$ V-notch weir and tank, having a maximum capacity of slightly over 1 cfs, placed at the end of the model discharge channel.

In order to obtain water-surface elevations over the model, 276 large-head galvanized nails were driven into the model surface until the nail heads were just flush with the surface. A model elevation corresponding to the prototype value was then established for each nail head. Twelve point gages attached to a movable truss were used to obtain model-depths over the nail heads. The model water depth at each nail was converted to a prototype value which was added to the previously established prototype elevation of the nail head to obtain the water-surface elevation.

A small propeller-type velocity meter was used to determine velocities of flow over strategic parts of the model.
Model Verification

Verification is a process to adjust the model to behave in a manner similar to the prototype. By this process the model will be made not only geometrically similar but also hydraulically similar to the prototype, and thus it will produce reliable data.

To obtain patterns of surface flow similar to the prototype it was found necessary to restrict the model water supply to certain sections of the upstream supply channel and to direct the major portion of the flow to move from a northeast direction toward the bridge. This pattern corresponds to the overflow from an upstream reach to the main channel of the Wabash River. Also, a roughening of the downstream end of the hole was found necessary to simulate properly the rotating-cell movement of the prototype flow.

In making verification runs emphasis was made to reproduce the rating curve relationship (shown by the dotted lines in Fig. 8) in the model. To do this it was found necessary to introduce a control, made of removable concrete bricks, at the downstream end of the model immediately upstream from the discharge channel. This control was adjusted so that the rating curve passed through the lowest and highest pairs of values that represent the 1950 high water flow.
After verification the patterns of flow in the calibration tests were found similar in appearance to those observed in the prototype for corresponding discharge rates, being characterized by pronounced rotating-cell movement on each side of the jet of flow through the hole. The cell on the east side of the hole was very large and moved in a counter-clockwise direction as driven by the jet of water moving through and across the hole. The cell on the west side of the hole moving in a clockwise direction was smaller and had a tendency to form two rotating cells. Rattle-snake patterns of surface flow downstream from each pile bent were also present in the model as they were in the prototype. Unless otherwise stated, the results of model tests will be presented in terms of prototype values.

18 General Procedure of Testing

The testing of the model consisted mainly of calibration tests and revision tests. The calibration tests were made to ascertain the existing conditions of the prototype as they appeared in the model. The revision tests were made to predict the conditions of the prototype due to the proposed revision as a remedial measure for the scour situation. The conditions to be observed in the tests included the water surface elevations, velocities of flow, and scour development at various rates of flow through the bridge opening.
Water surface contours and profiles at different tests and the velocity-discharge curves were determined. However, they are not shown here in order to reduce the volume of this report.

Velocity measurements were recorded across the bridge opening at different stations which were established by large-head nails for the measurement of water-surface elevations. These data were represented by velocity-discharge curves. The velocity is the mean velocity across the waterway area of the bridge, and it is equal to the rate of flow divided by the waterway area at 20 ft. north of the bridge centerline.

The scour development was determined qualitatively by introducing a given volume of sand in the model and measuring the quantity of sand moved during a fixed period of time. The movable material used in such silt tests was a uniformly-graded quartz sand. The procedure of the silt test is as follows: (1) The blue hole was first filled with sand to a plane of the 340.0-ft. elevation, and the sand was carefully leveled; (2) the model was slowly filled with water without disturbing the sand, and the sand was again checked for level; (3) the model was then run for 120 min. at a predetermined rate of flow for the test; (4) water was carefully drained from the model and photographic records were made for the resulting scour condition and
deposition pattern; (5) the sand above the 340.0-ft. elevation was removed from the area of the hole; (6) the sand remaining below elevation 340.0 was removed from the hole and was weighed after being drained in a sieve box for 15 min.; and (7) the sand that had been washed to above the 340.0-ft. elevation was added to the sieve box and was weighed to determine the total weight of sand used in the test. The weight of sand remaining below elevation 340.0 was plotted against the rate of flow maintained for the test. This plot provides the relationship in model between the weight of material remaining below elevation 340.0 and the rate of flow. The difference between the weight of all the sand used and the weight of sand remaining below elevation 340.0 was equal to the weight of the sand above elevation 340.0. The weight of sand above elevation 340.0 divided by the weight of all the sand used gave the per cent of material removed from below elevation 340.0. This per cent of material removed was then plotted against the prototype value of the rate of flow (Fig. 9).

Silt test data were obtained only for the calibration tests and for some revision tests. Other revision tests had obvious disadvantages, and hence their silt tests were not conducted.

19 Calibration Tests

The purpose of calibration tests is twofold: (1) To ascertain as
nearly as possible the detailed hydraulic characteristics in the vicinity of Bridge 3 as they occur during flood periods under the existing control conditions, and (2) to establish a plane of reference for comparison of the results of various model revisions. The procedure of calibration tests covered the full range of prototype bridge discharge and consisted of the determination of water surface elevations, velocities at and near the bridge opening, and the scouring capacity in the blue hole.

Water surface elevations were determined at 276 points over the model for prototype rates of flow ranging from 3,450 to 19,750 cfs. at intervals of about 1,000 cfs. From the measurement of water-surface elevations, contours and profiles of water surface were drawn for each rate of flow studied.

Average values of meter velocities measured across the bridge opening 20 ft. north of the bridge centerline and the mean velocity, \( Q/A \), were studied. It was noted that the mean velocity of flow through the bridge rises to a maximum at a discharge of about 13,000 cfs. and then decreases as discharge increases. For discharges greater than 13,000 cfs. the velocities are lower than the maximum velocity at 13,000 cfs. and a corresponding water surface elevation at 20 ft. north of the bridge centerline was 376.6 ft. The average values
of meter velocity were found to be quite erratic and deviate considerably from the corresponding values of mean velocity. The reasons may be: (1) The meter velocities were taken in the direction of flow irrespective of the alignment to the bridge opening; (2) at some points the flow was either too slow or too shallow for meter measurement, or was reversed in direction; and (3) the movement of water was in some cases so turbulent and surging that it was impossible to obtain an exact average of the meter velocity.

It was found that the flow along the sloping apron in the bridge opening moved in an upstream direction. Until almost in line with the face of the north sloping fill, this flow was swept downstream by the curvilinear flow near the intersection of the sloping apron beneath the bridge and the upstream sloping apron. This phenomenon occurred at both the east and west ends of the bridge opening for all rates of flow, but it was much more pronounced at the east end.

In order to confirm the above and other phenomena in the model, confetti was introduced into the upstream portion of the model and then carried through the model on the water surface to delineate the surface flow pattern in the model. It was observed that the confetti was carried downstream toward the roadway embankment, curving slowly toward the bridge opening. Much of the confetti was borne very close to the
roadway embankment before being turned sharply and carried parallel and close to the roadway toward the bridge opening. By the time this roadside flow had reached the bridge opening its velocity was sufficiently high to carry the confetti to the first pile bent or beyond and into the center span before being swept on downstream by the larger volume of the major flow. The major portion of the flow, approaching the bridge opening at an angle and separated by the upstream timber "island," was concentrated still further in the middle span due to the influence of the flow along the roadway. Apparently, a predominant part of the total flow had passed through the middle span.

The roadside flow continued to pull along water that lay in the outer spans adjacent to the toe of the approach sloping apron. The continual removal of this water had caused flow near the sloping apron to move in an upstream direction. This phenomenon was dramatically demonstrated in the model by the movement of confetti in an upstream direction along the toe of the approach sloping apron.

Nearly all the confetti was carried through the center bay of the bridge opening and on to an area near the downstream end of the hole before being caught in a rotating-cell movement that turned the flow of confetti sharply and brought it back upstream near the periphery of the hole. This rotating-cell movement on each side of
the main flow from the bridge opening was developed for all rates of flow. The movement was maintained and driven by the main flow which had a jet-like character due to the concentration of flow through the center bay of the bridge opening. The paths of confetti movement for the 8,000 cfs. and 17,000 cfs. runs are shown respectively in photographs 1 and 2 in Fig. 10.

As discussed previously, silt runs for six different rates of flow yielded a quantitative relationship between rates of flow and the scouring capacity of the moving water in the blue hole under the influence of the existing controls. The relationship between percentage of material moved and the rate of flow is shown in Fig. 9. Views of scouring conditions after the 8,000 cfs. and 17,000 cfs. runs are shown respectively in photographs 3 and 4 in Fig. 10.

20 Revision Tests

After the calibration tests, various modifications of the model were made in order to simulate remedial measures that will prevent further scour resulting from the complex nature of the flow in the area immediately downstream from the bridge structure, and will avoid particularly further expansion of the blue hole on its upstream edge.

Fourteen different revisions designated in order by letters from A to N were proposed and the corresponding tests were conducted.
In the revision tests, one of the most important factors in determining the efficiency of the proposed remedial measure was the visual observation of the flow pattern.

To help in studying the surface flow patterns and in recording them by photographs, confetti was introduced at places far upstream from the bridge and carried downstream by the flow through the bridge opening. As the confetti was moved along by the water, velocity changes and flow patterns could be easily seen. Photographs were taken for every rate of flow for all revision tests as they were taken in calibration tests. The velocity of flow can be interpreted from the distance shown on the photographs for which the confetti particles have moved during the time of photographic exposure. Flow patterns are shown on the photographs by the location and distribution of confetti particles. These photographic records include two 400-foot reels of 16-mm. motion pictures which recorded each of the calibration tests as well as the proposed revision tests. The flow patterns are of course more obvious on the motion pictures. Detailed descriptions of the revision tests are given in Appendix 2.

In proposing the revisions, several considerations were made. It is believed that the scour at Bridge 3 was caused by the excessive turbulence of the water under and downstream from the bridge. This
turbulence was developed by the meeting of the curvilinear approach flows and by the concentration of these flows at the center bay of the bridge opening. The curvilinear nature and concentration of the approach flows were caused partly by the timbered island upstream from the bridge and partly by flows approaching the bridge opening in a direction parallel to the roadway embankment. To aid in reducing curvilinear flow one phase of the remedial work was the removal of all trees from the timbered island for a distance of 300 ft. upstream from the bridge centerline. This measure was taken in all revisions.

Except for Revision M all revisions utilized a vertical cut-off wall parallel to and at 45 ft. downstream from the bridge centerline. This cut-off wall across the upstream end of the hole was designed with a purpose to prevent movement of the hole in an upstream direction and to allow discharge at a more or less constant depth into the area of the blue hole. The elevation of the top of the cut-off wall was set at 361.75 ft.

All revisions utilized permeable dikes upstream from the bridge except Revision L which had no dikes. Permeability was considered an essential specification of the dikes and hence all dikes were constructed to possess this characteristic. One purpose of the dikes was to decrease the effect of the side flows approaching the bridge.
| REVISION | UPSTREAM DIES | BENTS | CUT-OFF WALL | FLOW DISTRIBUTION THROUGH OPENING | MODIFIED WATERSHED BETWEEN MAIN DIES AND SLOPING APRONS | HAVING DIVERGING UPSTREAM ENOS |
|---------------------|---------------------|---------------------|---------------------|------------------------------------------------|------------------------------------------------|
| **A** | SAME AS REV. C EXCEPT THAT DIES BETWEEN THE BRIDGE OPENING AND SLOPING APRONS WERE NOT MOVED |Normal | Normal | No change | IMPROVED | NO Observable |
| **B** | USING 100-Ft. LONG CONVERGING DIES | Normal | Decentralized | Little change | Decreased 2% | NO Observable | Normal | Normal | NO | - |
| **C** | USING 75-Ft. LONG CONVERGING DIES | Normal | Concentrated | Decreased 3% | Reduced | NO Observable | Normal | Normal | NO | - |
| **D** | USING 75-Ft. LONG PARALLEL DIES | Normal | SATISFACTORY | LESS THAN NORMAL | INCREASED 25% | Reduced | Normal | Normal | NO | - |
| **E** | USING 100-Ft. LONG PARALLEL DIES | Normal | SATISFACTORY | LESS THAN ORIGINAL | INCREASED 15% | Reduced | HIGH TURBULENCE | DECREASED | NO | - |
| **F** | SAME AS REV. C EXCEPT DIKES EXTENDED WITH SLOPING TOP | Normal | SATISFACTORY | IMPROVED | LITTLE CHANGE | SLIGHTLY HIGHER THAN THE ORIGINAL | REDUCED 2.5 TIMES THE ORIGINAL AT X,000 CFS. | SATISFACTORY IN REDUCING HIGH VELOCITY AT DIKE ENDS |
| **G** | UPSTREAM ENOS OF REV. G WERE SWUNG OUTWARD | Normal | SATISFACTORY | IMPROVED | Slightly higher at the original | REDUCED 2.4 TIMES THE ORIGINAL AT 15,000 CFS. | SATISFACTORY IN REDUCING DISCHARGES BELOW 12,000 CFS. | ABOVE 17,000 CFS. |
| **H** | USING WALLS ADDED TO ENOS OF REV. F | Normal | SATISFACTORY | IMPROVED | REDUCED | HIGH VELOCITY | LESS THAN THAT IN REV. G | MAX. SCOUR GREATER THAN REV. K BUT LESS AT LOW DISCHARGES |
| **I** | MODIFYING REV. F BY LONG SLOPING TOP | Normal | SATISFACTORY | IMPROVED | REDUCED | HIGH VELOCITY | LESS THAN REV. F | MAX. SCOUR GREATER THAN REV. F |
| **J** | NO | BENTS | SATISFACTORY | IMPROVED | REDUCED | HIGH VELOCITY | LESS THAN REV. M | MAX. SCOUR GREATER THAN REV. M |
| **K** | NO | BENTS | SATISFACTORY | IMPROVED | REDUCED | HIGH VELOCITY | LESS THAN REV. M |
| **L** | NO | P I E R S | SATISFACTORY | IMPROVED | REDUCED | HIGH VELOCITY | LESS THAN REV. M |
| **M** | NO | P I E R S | SATISFACTORY | IMPROVED | REDUCED | HIGH VELOCITY | LESS THAN REV. M |
| **N** | NO | P I E R S | SATISFACTORY | IMPROVED | REDUCED | HIGH VELOCITY | LESS THAN REV. M |
opening normal to its centerline. The permeability of the dikes allowed, therefore, the side flows to enter the main flow with greatly reduced volume and velocity, and thus to decrease the hydrostatic head in the design of the dike structure.

The fourteen proposed revisions consist of various arrangements of permeable dikes and cut-off walls. Table 2 shows the arrangements for all revisions and summarizes the descriptions and testing conditions and results for each revision.

21 Practical Considerations

Laboratory tests indicate that scour immediately downstream from Bridge 3 may be substantially reduced by the effective use of a combination of following remedial measures:

1) The removal of timber from the upstream wooded island.

2) The proper installation of suitable dikes on the upstream side of the bridge.

3) The transformation of pile bents into piers.

4) The installation of a cut-off wall on the downstream side of the bridge.

The practical considerations for these remedial measures are described as follows:
With reference to remedial measure (1), the removal of the timber from the upstream wooded island of the model allowed the water to approach the opening in a direction more nearly normal to the centerline of the bridge and hence reduced the curvilinear nature of the approach flows.

With reference to remedial measure (2), all dikes tested in the laboratory had a thickness of four feet and were permeable. Dikes of a permeable nature were considered extremely desirable for several reasons. Had impermeable dikes been used, flow approaching the opening and parallel to the roadway would have been forced around the upstream ends of the dikes, until the dikes were overtopped. This would have caused increased velocities at the upstream ends of the dikes, which means greater danger of scour. An impermeable dike would have a greater pressure differential across the dike for its full length, thus requiring more expensive construction to protect the structure from lateral tipping.

Permeable dikes will serve the purpose of (1) directing approach flows to a direction normal to the bridge centerline, and (2) reducing flows, which approach the bridge opening in a direction parallel to the roadway, without increasing the danger of scour at the upstream ends of the dikes. A permeable dike 4 ft, wide may be
constructed in the following steps: (1) Driving pairs of steel posts at intervals of 4 ft. apart for the full length of the dike, (2) placing heavy wire mesh inside each line of posts for the length of the dike, (3) tying each pair of posts securely together, and (4) filling the space between the wire mesh with stone to the desired elevation. When the dikes are built of steel and stone, the corn stalks and other organic debris caught in the structure can be removed by burning during low water stages, thus reducing maintenance to a minimum.

The most effective dikes tested in the laboratory (Revision J) were placed parallel to, and symmetrically about, the centerline of the bridge opening and 71 ft. therefrom. The dikes were on the north side of the bridge, beginning at the top of the face of the sloping apron and extending upstream nearly 125 ft. They were so stepped that the top of each dike was at three levels, progressively lower in an upstream direction. The advantages of using stepped dikes are twofold; namely, first, decrease in flow around the ends of dikes by overtopping dikes at steps of lower level; and secondly, economy in construction gained by decreased design load and material requirements.

With reference to the remedial measure (3), piers instead of pile bents are helpful in reducing scour since they aid in the proper alignment of flow through the bridge. However, the decrease in scour
due to piers is not great. From the practical viewpoint, therefore, whether the benefit to be derived from this measure would justify the additional cost of construction should be considered.

With reference to the remedial measure (4), the placing of a vertical cut-off wall 45 ft. from and parallel to the centerline of the bridge on the downstream side was undoubtedly of great benefit in the reduction of scour. The corresponding prototype length of the cut-off wall was 93 ft. which is the same as the distance from toe to toe of the approach fills. This length of cut-off wall, however, would not be adequate for the prototype because of the erodible nature of the soil. A better dimension for the cut-off wall would be 150 ft. placed 45 ft. from and parallel to the centerline of the bridge. The center of the wall should be on the centerline of the bridge opening. The top of the wall should be at elevation 362.0 ft. across the central part of the opening and follow the natural ground profile at the ends. All areas north of the wall that are below the elevation of the top of the wall should be filled with a durable material and leveled with the top of the wall to prevent erosion and converging of the flow. The bank at each end of the wall should be protected by means of rubble fill to check erosion caused by wind generated waves. A sketch of such a cut-off wall is shown in Fig. 11.
Erosion from wave action caused by wind has been quite active at the blue hole of Bridge 3. The most active areas of such erosion are located along the northerly bank of the hole on each side of the bridge opening. This erosion encroachment, if not stopped, would endanger the ends of the cut-off wall, and consequently the roadway fill. Photographs in Fig. 12 illustrate the erosion in progress.

It should be noted that the use of either the cut-off wall or the permeable dikes alone is not recommended because the silt tests indicated a considerably greater degree of reduction in scouring when they were used jointly, although tests of each used singly also showed an appreciable amount of reduction in scouring capacity. The merit of dikes, cut-off wall, piers, bents, and their various combinations will be discussed in detail in Appendix 2.
IV RECOMMENDATION

22 Solution of Bridge 3 Problem

Based upon laboratory investigation and field information the following remedial measures are recommended for stopping the threat of destruction to Bridge 3 caused by the dangerous scour that occurs periodically with existing control conditions:

(1) Clear the upstream wooded area of all timber for a distance of at least 350 ft. from the centerline of the bridge. It should be noted that the recommendation calls for a minimum clear distance of 350 ft. while a corresponding distance of 300 ft. was used in the model. The additional length of 50 ft. is recommended in consideration of the possible deposition of debris in the area in future.

(2) Construct permeable, stepped dikes made of steel and stone on the upstream side of the bridge parallel to and symmetrical about the centerline of the bridge opening and 71 ft. therefrom. These dikes should commence at the faces of the approach fills and extend northward to points 150 ft. from the centerline of the bridge. The top of the northerly 55 ft. of each dike should have an elevation of 374.0 ft. and the remaining length of each dike to the face of the approach fill should have an elevation of 378.9 ft. The upstream end of each dike should have rubble placed around its toe. A sketch of this dike arrangement is shown in Fig. 13.
(3) Construct a 150-ft. long cut-off wall of sheet-steel type on the downstream side of the bridge 45 ft. from and parallel to its center-line. The central section of cut-off wall should have a top elevation of 362.0 ft. The top of the end sections of cut-off wall should follow the ground profile. The ends of the wall should be protected with rubble fill. A sketch of this cut-off wall arrangement is shown in Fig. 11.

In addition to the above, vigilance should be exercised in the observation of increased scour particularly at the downstream end of the scour hole. 

23 Future Studies

The present investigation has led to an effective and practical solution of the problem to prevent further scour at Bridge 3. As this solution has been mostly developed and verified in the laboratory, the next step should be the application of the recommended remedial measure to the prototype structure and the examination of the actual performance in order to obtain a field verification.

In the laboratory the investigation was made on a fixed-bed model and thus the information obtained for scour situation is only qualitative. In order to evaluate the quantitative nature of scour, a movable-bed model may be constructed for future studies if expenditure of more funds can be shown to be justifiable. Also, a general
study of the hydraulics of flow at bridges is recommended on a bridge model without the blue hole, and thus, the hydraulic effect due to an individual measure, such as the dikes, cut-off wall, etc., can be fully evaluated.

The problem under investigation is an individual case. The results thus obtained, however, are believed to be also applicable to other similar problems. It is recommended that information from other similar cases in the State should be collected and a study of the proposed remedial measure when applied to these cases be made. Thus, a comprehensive understanding of the hydraulics of flow at bridges as well as a more general remedial measure to scour at bridges can eventually be obtained.
APPENDIX 1 DEVELOPMENT OF MODEL EQUATIONS

A1-1 Model Theory

The relationships between the model and prototype dimensions may be developed by means of the dimensionless numbers, such as the Froude number and the Reynolds number. These numbers can be derived in fluid mechanics by the use of the principle of dynamic similarity and from a consideration of the physical forces, such as forces of gravity, viscosity, surface tension, etc., with respect to the resisting force of inertia. The derivation of the model equations assumes that for equal values of dimensionless numbers the corresponding flow patterns in model and prototype are similar. Thus, if model and prototype are exactly similar in hydraulic characteristics, the dimensionless numbers for model and prototype must be equal.

Since the flow at Bridge 3 is affected primarily by forces of inertia and gravity, the corresponding dimensionless number is the Froude number. Thus, the Froude number of the model must be equal to that of the prototype. Using subscripts \( m \) for model and \( p \) for prototype, it is required that

\[
\frac{V_m}{\sqrt{g d_m}} = \frac{V_p}{\sqrt{g d_p}} \tag{5}
\]

in which \( V_m \) and \( V_p \) are velocities of flow in ft. per sec., \( g \) is the
gravitational acceleration which is the same for both model and prototype, and \( d_m \) and \( d_p \) are depths of flow. Equation (5) may also be written as

\[
\frac{d_p}{d_m} = \frac{v_p^2}{v_m^2}
\]  
(A1-1)

A1-2 Length, Area, and Volume Relationships

By definition, a length in the prototype divided by the corresponding length in the model is equal to the scale ratio, or

\[
\frac{d_p}{d_m} = L_r
\]  
(A1-2)

or

\[
d_p = d_m L_r
\]  
(8)

Since area \( A \) and volume \( Vol \) are respectively the square and cube of the linear dimension, the following model equations can be written

\[
A_p = A_m L_r^2
\]  
(9)

and

\[
Vol_p = Vol_m L_r^3
\]  
(10)

A1-3 Velocity Relationship

From Eqs. A1-1 and A1-2, the velocity relationship is obtained as
\begin{align}
V_p &= V_m L_r^{0.5} \tag{11} \\
\text{Al}-4 \quad \text{Discharge Relationship} \\
\text{The rate of flow or discharge } Q \text{ is equal to the product of the area and the velocity. Thus, the product of Eqs. (9) and (11) gives} \\
A_p V_p &= A_m V_m L_r^{2.5} \\
\text{or} \\
Q_p &= Q_m L_r^{2.5} \tag{12} \\
\text{Al}-5 \quad \text{Roughness Relationship} \\
\text{The Manning equations for prototype and model are respectively} \\
V_p &= \frac{1.49}{n_p} R_p^{2/3} S^{1/2} \tag{A1-3} \\
\text{and} \\
V_m &= \frac{1.49}{n_m} R_m^{2/3} S^{1/2} \tag{A1-4} \\
in \text{which } V_p \text{ and } V_m \text{ are velocities in ft. per sec., } n_p \text{ and } n_m \text{ are coefficients of roughness, } R_p \text{ and } R_m \text{ are hydraulic radii, and } S \text{ is the hydraulic gradient, which is the same in prototype and model. Dividing Eq. A1-3 by Eq. A1-4 and solving for }\frac{n_p}{n_m}: \\
\frac{n_p}{n_m} &= \frac{V_m}{V_p} \left( \frac{R_p}{R_m} \right)^{2/3} \tag{A1-5} \\
\text{It should be noted that the hydraulic radius is in a linear dimension, or}
\[ \frac{R_p}{R_m} = L_r \] Thus, with the aid of Eq. 11, the ratio of the roughness coefficients may be written as

\[ \frac{n_p}{n_m} = \frac{1}{L^{0.5}} (L_r)^{2/3} = L_r^{1/6} \]

or

\[ n_p = n_m L_r^{1/6} \quad (13) \]
APPENDIX 2 REVISIONS

Sketches of the arrangement and summarized features of all revisions are shown in Table 2.

A2-1 Revision A

This revision was the first step toward the search for a most efficient measure of directing the flow of water through the bridge opening in a direction parallel to the centerline of the opening.

It has been noted that the waterway areas provided for the water prisms adjacent to and above the toe of the approach sloping apron were occupied largely by backwater moving in an upstream direction. Consequently, the areas were not utilized for positive flow through the bridge. In this revision, dikes were placed through the bridge opening above the toes of the sloping aprons and then directed divergently in the upstream direction at an angle of 7-1/2° with the normal to the bridge opening. Permeable cut-off dikes were also placed between the main dikes and the sloping apron on both the upstream and downstream sides. A drawing of the revision and the photographic views are shown in Fig. A-1.

Although this revision improved the flow condition through the bridge, it was not a satisfactory solution because the major flow was still concentrated in the middle span and tended to jet into the area of
the blue hole. Furthermore, the rotating-cell movement which was so noticeable in the calibration tests of the model in original condition showed almost no diminution.

A comparison between the rating curves for Revision A and the original rating curves is shown (Fig. A-1). It will be noted that at a discharge of 14,000 cfs. the difference in elevation between the upstream and downstream water-surface elevations has increased from 2.3 ft. in the original condition to 3.6 ft. in the revision. This indicates a decrease of about 13% in the capacity of the opening when the upstream water-surface elevation corresponds to 378.0 ft.

Revision B

In Revision A too much restriction seemed to be developed in the bridge area. For this reason the portion of the dikes of Revision A which were parallel and within the bridge opening were removed. The remaining dikes on the downstream side of the bridge constituted the remedial measure in Revision B.

From the rating curves for Revision B, it was found that the difference between the upstream and downstream water-surface elevations was less than that of Revision A, but it is still 0.6 ft. greater than the corresponding difference in the original condition at 14,000 cfs. Thus, the capacity of the bridge opening was still less
than the original capacity. When the upstream water-surface elevation was at 378.0 ft., the dike arrangement in Revision B has reduced the capacity of the opening by nearly 10%.

In this revision the flow was concentrated in the middle bay and jetted into the area of the blue hole, thus aggravating the rotating-cell movement of the water in the downstream area on each side of the jet. Pronounced backwater conditions were also found to exist along the sloping apron, apparently aided by the permeable dike with cut-off walls.

A2-3 Revision C

Since Revision B had the disadvantage of causing increased backwater in the bridge area and continued concentration of flow through the center bay, Revision C was proposed to remedy the condition by placing 100-ft. long straight but converging dikes at the upstream side of the bridge opening. The dikes were started at the intersection of the tops of the north-south and east-west sloping aprons, converged upstream, and terminated at points 54.5 ft. from the center-line of the bridge opening. These terminating points were approximately at the same location of the upstream ends of the dikes in Revision B. The purpose of this arrangement was to allow the water in the outer spans to flow freely downstream and thus avoid the backwater created in Revision B.
From the rating curves it was found that the difference between the upstream and downstream water-surface elevations is very little more than that of the original, and that the capacity of the opening was reduced only about 3% when the corresponding upstream water-surface elevation is 378.0 ft.

In this revision, the flow through the bridge opening was decentralized and the outer spans were made to carry more flow. A good distribution of flow was observed, as confetti was found to be distributed in all bays. One disadvantage of this revision, however, is the relatively high velocities of flow that were found at the upstream ends of the dikes.

A2-4 Revision D

To decrease the high velocities that existed at the upstream ends of the dikes of Revision C, a wider opening was provided in Revision D by shortening each dike by 25 ft. at the upstream end.

The rating curves for this revision are very nearly the same as those for the original tests before dikes were introduced, indicating almost no change in capacity.

This revision was not a particular improvement over Revision C because a large portion of the flow is still concentrated through the two westerly bays of the opening.
A2-5 Revision E

To distribute flow more uniformly across the full width of the bridge opening, the 75-ft. long dikes of the previous revision were made parallel with the centerline of the opening and at a distance of 142 ft. apart. This revision has produced a satisfactory distribution of flow through the bridge opening.

The rating curves of this revision indicate that the capacity of the opening was increased by about 2.5% at an upstream water-surface elevation of 378.0 ft. Also, it was noted that for rates of flow above 12,000 cfs. the difference in the upstream and downstream water-surface elevations was less than that of the original tests.

The two westerly bays of the opening still carried more than their proportionate share of water as indicated by higher water-surface elevations. In the area of the blue hole the western zone of contact between the inflow from the bridge and the relatively still water on the side contained many vortices which indicated considerable turbulence in that area.

A2-6 Revision F

This revision is a modification of Revision E by extending the 75-ft. long, parallel dikes to a length of 100 ft. This was done to allow more time for the inflow to the dike area to distribute evenly across
the bridge opening and thereby decrease the amount of flow that had been carried by the two westerly bays in Revision E. It was expected that this action would decrease the downstream turbulence in the area of the blue hole.

The rating curves of this revision were found to be very nearly the same as those for Revision E. The capacity of the opening was higher than the original by about 2% at a water-surface elevation of 378.0 ft.

In Photograph 2 of Fig. A-2, well distributed parallel flow can be seen over the entire bridge opening. The streaks of confetti indicate further that the velocities are relatively high around the upstream ends of the dikes. The excessive velocities at the upstream ends of the dikes were found to be the only disadvantage of this revision.

A2-7 Revision G

The converging dikes of Revision C and the parallel dikes of Revision F have both produced satisfactory hydraulic characteristics through the bridge opening and on the downstream side in the area of the blue hole. The common disadvantage of both revisions was the excessive velocities around the upstream ends of the dikes. Further revisions were therefore needed for reducing these excessive velocities.
In Revision G (Fig. A-3) the dike arrangement was made the same as that of Revision C except that the dikes were extended further upstream with the height of dikes decreasing at an angle of 30° to the horizontal, starting at points 100-ft. from the downstream ends of the dikes. Revision G was given more extensive tests than previous revisions because it was believed to be a possible solution to the problem. Hence, silt tests were made on this revision.

The rating curves for Revision G are similar to those of Revision C. The difference between the upstream and downstream water-surface elevations was 0.5 ft. greater than that for the original tests at a rate of flow of 14,000 cfs. The capacity of the bridge opening was less than the original capacity by 7.5%. A small reduction in capacity is allowable since lower velocities through the bridge opening would then prevail. However, 7.5% is a greater reduction than desirable.

The distribution of flow across the bridge opening was satisfactory as was the flow further downstream in the area of the blue hole. Photograph 2 in Fig. A-4 shows that the large rotating-cell movement that existed on the east side of the blue hole in the pre-dike tests has been completely eliminated, but the small rotating-cell movement on the west side of the hole continued to form. The relative velocities
over the model are indicated by the length of the streaks developed by confetti. Accordingly, it will be noted that in the constricted area at the entrance to the dikes, relatively long streaks of confetti can be seen, indicating relatively high velocities in this area.

The silt tests were conducted in the same manner as those of the original tests. Photographs 3 and 4 in Fig. A-4 show the results of scour for two different rates of flow for Revision G. When these photographs are compared with those taken at the original testing conditions (Photographs 3 and 4 in Fig. 10), the great decrease in scour will be immediately noticed. It will be noted that the scouring capacity of the flows under original conditions has exposed the bottom of the blue hole in the model and carried quantities of material from the hole for depositing on the nearby higher land. The scour that was observed under the diked condition of Revision G was, however, of comparatively minor proportion.

For the silt tests, the per cent of material moved is plotted as the ordinate against the discharge as the abscissa (Fig. A-3). This plot indicates that for discharges above 10,000 cfs., the scour developed under the original condition was at least three times as great as that under the condition of Revision G.
A2-8 Revision H

The occurrence of relatively high velocities across the entrance to the dikes of Revision G was a disadvantage which led to the proposal of Revision H. In modifying Revision G for Revision H, the upstream ends of the converging dikes were swung outward, away from the centerline of the bridge opening, until they were 170 ft. apart at points 100 ft. upstream from the top of the sloping apron, thus converting them to diverging dikes.

The rating curves of this revision indicate that this dike arrangement gives the highest hydraulic efficiency in the capacity of openings tested. It was found that at a discharge of 14,000 cfs., the difference between the upstream and downstream water-surface elevations was only 1.7 ft. as compared to the difference of 2.3 ft. of the pre-diked condition. Also, it was noted that when the upstream water-surface was at elevation 378.0, the increase in capacity was about 3.3%. This increase in capacity is an undesirable factor, since the bridge opening must maintain high average velocities.

The tests indicated that the west half of the opening carried a major share of the flow, and that higher velocities had extended into the area of the blue hole.
Silt tests were conducted on Revision H. The result showed that sand deposition was more extensive than that developed under the conditions of Revision G. It was found that the per cent of material moved in the present revision had reached a maximum of 20% as compared to 16.5% in Revision G. However, the improvement over the original or pre-diked condition was nearly as great as that of Revision G.

Revision I

The parallel dikes of Revision F produced good hydraulic characteristics except for the high velocities that existed at the upstream ends of the dikes. Revision I is a modification of Revision F. To the ends of the 100-ft. long, straight dikes of Revision F were added wing wall extensions that flared away from the centerline of the bridge opening at an angle of 60°. The rating curves of this revision are close to the pre-diked rating curves. This indicates that the bridge opening had about the same capacity as the original.

The results of two silt tests indicated little alteration from the respective tests of Revision G. The material moved was found to reach a maximum of 15% at a discharge of 12,000 cfs. At the same discharge, the original tests have produced a value of 43% of material moved.
Revision J (Fig. A-5) was a further modification of decreasing the velocities at the upstream ends of the dikes. Revision J was composed of two long, straight, stepped dikes 142 ft. apart, equidistant from, and parallel to, the centerline of the bridge opening. In elevation each dike had a four foot step, at points 50 ft. and 100 ft. from the upstream end. The upstream end of the dike was located 125 ft. north of the top of the upstream sloping apron. The top of the upstream 50 ft. of dike was at elevation 370.0, the top of the next 50 ft. was at elevation 374.0, and the top of the remaining distance to the sloping apron was at an elevation of 378.0.

The rating curves of this revision were very close to the original rating curves up to a rate of flow of 14,000 cfs. For discharge rates above 14,000 cfs., the capacity of the bridge opening was slightly greater than the pre-diked conditions. Photographs 1 and 2 in Fig. A-5 show the patterns of flow for the 8,000 cfs. and 12,000 cfs. runs. The pattern of approach flow is especially noticeable for the 8,000 cfs. run.

The silt tests indicated that a 14 per cent of material moved at a rate of flow of 11,130 cfs. At this discharge the original per cent of material moved was 34% or 2.4 times greater than the revised condition. A curve in Fig. A-5 shows the variation in per cent material
moved with respect to the increased discharge. At a discharge of 19,000 cfs, the per cent material moved reached a high value of 16.5%. This discharge exceeds by nearly 3,000 cfs or the highest recorded flow through Bridge 3. The results of the silt tests are shown by photographs 3 and 4 in Fig. A-6.

Revision K

In all revisions thus far discussed, two components have been added to the original condition; namely, the set of permeable dikes on the upstream side of the bridge and the cut-off wall on the downstream side. To determine the relative effectiveness of each component of the remedial measure, tests were run with only one element in operation at any given time.

For Revision K the dikes were entirely removed, leaving only the cut-off wall. The top of the cut-off wall, at some places, was made higher than the ground upstream from it. In these locations enough fill was placed back of the cut-off wall to give a level surface.

The rating curve with reference to the upstream water surface of Revision K was found to be closely resembled to that of the original. The downstream rating curve is slightly higher than that of the original, indicating that the water surface at the upstream end of the blue hole, and hence the water surface along the roadway fill, were
higher than those in the original. The water surface profiles confirm this deduction. The reason for this change was that the cut-off wall has prevented the bridge discharge from diving or plunging into the hole and thus lowering the water surface. The movement of the confetti as observed in the test indicated that the primary flow was through the center span.

Two silt tests were conducted in Revision K. When the results of the tests were compared with those of the original, a great reduction in scour was noticed. On the other hand, when compared with the silt-tests of previous revisions, it was seen that the scour which occurred in Revision K was relatively high.

A2-12 Revision L

In all of the revisions up to this point, the concrete pile bents of the original structure were left in the natural condition. For Revision L the bents were made to simulate piers instead of piling and the cut-off wall was left on the model. As in Revision K, the dikes were entirely removed. The only difference between Revision K and Revision L was that Revision L had piers instead of pile bents.

The rating curves indicated that the capacity of the bridge was 7% greater than the original at a discharge of 6,000 cfs. and then decreased with an increase in discharge up to 16,000 cfs. Apparently the piers helped to increase the flow at the lower discharges.
The silt tests indicated that more scour occurred when piers were substituted for pile bents. However, the result showed that while the maximum scour was greater for Revision K, the scour at the lower discharges was higher for Revision L. This indicates that for discharges above 15,000 cfs, the piers are advantageous, but for discharges less than 15,000 cfs, the piers tend to increase scour.

A2-13 Revision M

For Revision M (Fig. A-7) the straight, parallel, stepped dikes of Revision I were placed on the model, the cut-off wall was removed, and piers were used instead of pile bents. The purpose of this revision was to determine the value of permeable dikes and pier bents without the cut-off wall. This was the only revision in which the cut-off wall was not used.

A study of the rating curves for Revision M (Fig. A-7) will show that up to 10,000 cfs, the curves approximate closely to those of the original, but that at discharges above 10,000 cfs, the capacity of the opening was increased. The confetti in the photographs 1 and 2 of Fig. A-8 indicates the pattern of flow. It should be understood that the confetti was stopped by dikes until the latter were overtopped, and thus the flow through the dikes cannot be shown.
Results of two of the silt tests are shown in photographs 3 and 4 in Fig. A-8. It is apparent from these photographs that the scouring was relatively severe without the aid of the cut-off wall. The graph for per cent of material moved versus discharge (Fig. A-7) indicates more clearly the extent of scour. For discharges less than 10,000 cfs. the dikes and pier bents were not of great value, but for discharges above 10,000 cfs. they were effective in reducing scour.

A2-14 Revision N

The component parts of Revision N (Fig. A-9) include: (1) the straight, parallel, stepped dikes like those of Revision J, (2) the cut-off wall across the upper end of the blue hole as in all revisions except M, and (3) piers instead of pile bents. Revision N was identical to Revision J, except that in the latter the bents were of concrete piling. By comparing the results of Revision J with those of Revision N, the effect of piers can be easily seen.

The rating curves of Revision N (Fig. A-9) are nearly identical with those of Revision M.

By comparing the rating curves of Revision N with those of Revision J and with those of Revision M, it can be seen that neither cut-off wall nor piers have any appreciable effect upon either upstream or downstream water surface when the permeable dikes are in place.
As will be noticed, Revision N is also identical with the original rating curve except that at higher discharges the bridge opening operates at a slightly greater efficiency under the conditions of Revision N.

The flow patterns in Revision N are shown by photographs 1 and 2 in Fig. A-10. The large rotating-cell that was on the east side of the blue hole under original conditions was almost non-existent. However, these continued to be two small rotating cells on the west side of the hole. Obviously, floating particles, such as confetti, can indicate only the surface pattern of flow.

By comparing the silt-tests photographs of Revision N (No. 2 and 3 in Fig. A-10) with those of Revision M (Fig. A-8), it can be seen at once that the insertion of the cut-off wall decreased the scour appreciably. The function of the cut-off wall can be more clearly understood by study of the graph for per cent of material moved versus discharge (Fig. A-9) and by making comparison of it with the corresponding graph of Revision M.

The effect of piers replacing the pile bents may be seen by comparing the graph for material moved as shown in Fig. A-9 with the corresponding graph of Fig. A-5 of Revision J. It will be noted that a slightly less scour occurred when pier bents were used, especially at discharges below 12,000 cfs. and above 17,000 cfs. Flows above 17,000
cfs. were not of special interest because the maximum recorded flow through Bridge 3 is only 16,300 cfs. At discharges less than 12,000 cfs, the effect of piers was favorable. For example, at 10,000 cfs, the use of piers instead of pile bents causes the per cent of material moved to drop from 13% to 11%.

A2-15 Extended Tests on Revision J

When considering both the hydraulic and economic aspects of all fourteen revisions (see Table 2), Revision J was found to be most practical. Since abridged tests were conducted upon all revisions, it was considered necessary that a more extensive and thorough investigation be made of Revision J. For the convenience of identification, Revision J is referred to as Revision J' when it was subject to more extensive tests.

As it may be recalled, Revision J was composed of three distinct features: (1) Two dikes on the upstream side of the bridge, (2) pile bents, and (3) a cut-off wall on the downstream side of the bridge. The dikes were straight, parallel, stepped dikes of a permeable character, and were placed symmetrically about the centerline of the bridge opening and 71 feet therefrom. The pile bents tested were in the condition as they exist. The cut-off wall was a vertical wall placed 45 ft. south of, and parallel to, the bridge centerline. Fill was
made in places where necessary on the upstream side of the cut-off wall to form a level surface.

Calibration tests same as those for Revision J were conducted upon Revision J'. Silt tests for Revision J were not made since the silt tests of the original model were rather thoroughly conducted.

Water surface contours for Revision J' for the 8,000 cfs. and the 17,000 cfs. runs were determined. With reference to the rating curves for Revision J' (Fig. A-11), it is seen that the upstream water surface elevations for 8,000 cfs. and 17,000 cfs. were respectively 375.35 ft. and 382.60 ft. at a point 200 ft. west of the west bridge abutment. The three steps on the dikes were at elevations 370.0, 374.0, and 378.0. Thus, at a discharge of 8,000 cfs. the second step of the dikes was overtopped, and at 17,000 cfs. the entire length of dikes was overtopped. In comparison with the original contours for the corresponding discharges, it will be noted that the steepness of the hydraulic gradient was much decreased in Revision J and that the curvilinear character of the approach flow was suppressed and moved upstream from the bridge opening.

The water surface profiles for the centerline of the bridge opening and for the east-west range 20 ft. north of and parallel to the bridge centerline are shown in Fig. A-11 for both the original and
Revision J'. It will be noted that the centerline profile of Revision J' was not as steep in the approach to the bridge as was the profile of the original. This indicates less velocity variation along this line. The lowest water surface elevation for Revision J' in the bridge area for a discharge of 8,000 cfs. was not greatly different from the same of the original, but was formed with a more mild and uniform approach. At higher discharges the original tests indicated a much steeper hydraulic gradient, dropping to a low point just downstream from the bridge centerline. When comparing the profiles for 17,000 cfs., it will be seen that the profile for the original was lowered to an elevation of 380.10, or 0.5 foot lower than the surface elevation at 65 ft. further downstream. The relative smoothness of the transition for Revision J' is striking.

The east-west profiles (Fig. A-11) indicate that the distribution of flow across the bridge opening was greatly improved by Revision J'. It will be seen that the water surface in the center of the opening was higher than that near the approach fills in both instances. However, the difference between the highest and lowest water surfaces across the opening was 0.9 ft. for the original as compared to 0.2 ft. for Revision J' at 8,000 cfs. The same comparison for a discharge of 17,000 cfs. is again 0.9 ft. for the original as compared to 0.55 ft. for Revision
The most extreme difference between the water surface of the central section and that near the approach fills at 12,000 cfs. was 1.7 ft. in the original against 0.25 ft. for Revision J'.

The average values of meter velocities across the bridge opening at 20 ft. north of the bridge centerline were plotted against discharges (Fig. A-11). The corresponding values from the original tests are also shown in the figure. The accuracy of meter velocities at certain points is questionable because of the turbulence and fluctuation of velocities at those points with respect to time. Average meter velocities for Revision J' are lower than the original at discharges greater than 13,000 cfs. Higher average velocities appear to occur in Revision J' for discharges less than 13,000 cfs. During the calibration or original test for discharges less than 13,000 cfs, the flow conditions were such that at least one and usually two velocity stations out of seven had such low values that they were below the range of the velocity meter. The calibration values plotted were for the average of all seven velocity stations and hence the average value was low. This is the reason for the change in position of the two curves at discharges less than 13,000 cfs.

The mean velocities of flow, \( V = \frac{Q}{A} \), were plotted against discharge as shown in Fig. A-11. The corresponding plot for the
original test is also shown for comparison. The curve for Revision J follows the original closely except that it has a slightly greater and later peak. The flow through the bridge opening was found to be well distributed for Revision J. There were no areas of backwater in the bridge opening and all bays carried a relatively equal flow, proportionate to their cross sectional areas.

The photographs (Fig. A-12) indicate a part of the surface flow by means of confetti. It should be noted, however, the indication by confetti is indicative of the surface flow pattern only as the confetti does not show the nature of the sub-surface flow. Since the layer of flow near the stream bed is the portion of flow responsible for scour, its prime importance is evident. The observation on sub-surface flow was made by dropping a short piece of string into the water, and thus the direction of flow at any depth could be seen in any location desired. The tests indicated that the surface flow over the dikes was almost perpendicular to the centerline of the opening for a short distance from the dikes, but underneath this top layer of flow there appeared another layer that consistently moved parallel to the centerline of the opening and hence parallel to the dikes. The parallel flow of the lower layer existed from dike to dike across the entire width of opening. The cross flow and turbulence of the surface layer did not
extend either to the stream bed or to a great distance beyond the dike. This is a very significant advantage obtained by the use of dikes.

The silt tests of Revision J have been discussed before. The resulting graph for per cent material moved versus discharge is compared with similar graphs of the original test and the tests for Revisions N, K, and M (Fig. A-13). It will be recalled that Revision N was the same as Revision J except that the former had piers and the latter had pile bents. Revision K had no dikes but pile bents and cut-off wall. Revision M had no cut-off wall but stepped dikes and piers. It will be seen that neither dikes alone nor cut-off wall alone gave an adequate solution to the scour problem, but by using both dikes and cut-off wall the scour was reduced to a reasonable extent. It should be noted again that the results shown by silt tests were qualitative only and did not indicate the quantities of prototype material moved. However, from such tests the degree of reduction in scouring capacity can be detected.
APPENDIX 3 FLOOD FREQUENCY STUDY

In the design of hydraulic structures it is sometimes desirable to know the frequency of occurrence of a flood of given magnitude. A curve showing the relation between flood magnitude and probable recurrence interval will aid the engineer in preparing a most economic design, since it will provide a means of balancing the benefit arising from adequate flood prevention against the expenditure for the structure. The recurrence interval is the average period in years between the occurrence of a flood of given magnitude and an equal or larger flood. It should be noted, however, that once the recurrence interval is determined, it does not necessarily mean that the flood of given magnitude will occur exactly once in the interval. The recurrence interval indicates only an average condition. The reliability for determining the recurrence interval depends on the accuracy of data and the length of record.

For determining the flood frequency in the Wabash River, the records available at New Harmony, Indiana, and Mt. Carmel, Illinois, cover respectively 9 years (October 1938 to September 1947) and 20 years (October 1927 to September 1947). The annual maximum mean daily discharges of records were listed in increasing order of magnitude. The annual maximum discharge record at Mt. Carmel covering
a period (October 1938 to September 1947) corresponding to that at New Harmony was also listed in increasing order of magnitude. By plotting these data on a special probability paper developed by Powell (Ref. 3) for the Gumbel method of frequency analysis, three straight lines were drawn respectively for plotted data of the periods: 1938-1947 for Mt. Carmel, 1938-1947 for New Harmony, and 1927-1947 for Mt. Carmel. The plots indicate that for discharges greater than 86,000 cfs., a flood of a given magnitude would have a shorter recurrence period at New Harmony than at Mt. Carmel. In other words, for the same recurrence interval, the discharge at New Harmony would be greater than that at Mt. Carmel.

By comparing the short record at New Harmony and the long record at Mt. Carmel and by increasing discharges of high recurrence intervals at Mt. Carmel in proportion to the increase indicated by the records of same period at New Harmony and Mt. Carmel, it was possible to derive a frequency curve of longer period at New Harmony. This curve together with the Wabash River rating curve and the New Harmony stage versus Bridge 3 discharge curve were used to derive a relation between discharge at Bridge 3 and recurrence interval as shown in Figure A-14. With reference to the rating curve for Bridge 3, a relation between the recurrence interval and the upstream water surface elevation at Bridge 3 can also be plotted as shown in Figure A-15.
APPENDIX 4 REFERENCES

(1) U. S. Waterways Experiment Station, "Method of Operation of the Birds Point - New Madrid Floodway, Missouri; Model Investigation," Technical Memorandum No. 2-300, August 1949.


APPENDIX 5  FIGURES
Fig. 1 Location of bridge 3 and map of vicinity

Fig. 2 Contours of the Blue Hole

Fig. 3 Volume of material removed from the Blue Hole below el 360 vs time
Photo 1
Taken from north side of west abutment. Note high degree of turbulence and super-elevation in near bay. Depth of water under bridge is about 14.5 ft. Q = 14000 cfs approx. Upstream water surface elevation, 377.65.

Photo 2
View of flow around north side of east approach fill taken from Bridge 3. Note the curvilinear flow. Q = 14000 cfs approx. Upstream water surface elevation, 377.65.

Photo 3
North bank of the Blue Hole taken from east side of hole 75 ft south of roadway. Note quantity of rubble along edge of hole.

Photo 4
View of area of deposition at southwest end of hole. Note size of rubble that was transported from bridge to present location.

Photo 5
View of terminus of deposition of well graded sand scoured from hole. This deposit extends 200 ft from the southwest edge of hole. Note the abrupt change to the tilled land in the foreground.

Photo 6
View of upstream slope paving of west approach fill. Good hydraulic properties of prism facilitate approach flow normal to the center line of bridge opening.

Fig. 4 Photographs of the problem area
Fig. 5 Hydrographs for the 1943 flood

Fig. 6 Rating curves at bridge 3 and for the total flow

- Flood plains and main channel, total discharge
- Bridge 3 discharge
- Bridge 4 discharge
Fig. 7 Water surface elevations at New Harmony Bridge vs Bridge 3

Fig. 8 Rating curves for Bridge 3 with respect to upstream and downstream stages

Fig. 9 Per cent of material removed vs discharge for the original model
Area to be filled and topped with level surface

Cross section of bridge opening at south 45

Section along bridge opening

Fig. 11 Recommended cut-off wall

Fig. 10 Photographs for calibration tests
Photo 1 View of erosion due to wind-blown wave action on the north bank of the Blue Hole east of the bridge. Note that erosion is occurring where the bank is not protected by rubble.

Photo 2 View of the north bank of the Blue Hole near the westerly approach to Bridge 3.

Fig. 12 Photographs showing erosion caused by wind-generated waves.

Fig. 13 Recommended dikes.
Fig. A1 Data for Revision A
**Fig. A2** Photographs for Revision F

**Fig. A3** Data for Revision G
Photo 1 View of dike arrangement of Revision G.

Photo 2 Overhead view of model. $Q_p = 12,000$ cfs

Photo 3 Overhead view of result of silt test. $Q_p = 7,950$ cfs Time for test: 120 minutes

Photo 4 Overhead view of result of silt test. $Q_p = 17,160$ cfs Time for test: 120 minutes

Fig. A4 Photographs for Revision G
Photo 1 Overhead view of model. An excellent example of curvilinear approach flow becoming transformed to parallel flow through bridge opening. \( Q_p = 8000 \text{ cfs} \)

Photo 2 Overhead view of model. Note surface undulations along westerly dike caused by overtopping of dike. \( Q_p = 12000 \text{ cfs} \)

Photo 3 Overhead view of result of silt test. \( Q_p = 8000 \text{ cfs} \) Time for test: 120 minutes

Photo 4 Overhead view of result of silt test. \( Q_p = 17000 \text{ cfs} \) Time for test: 120 minutes

Fig. A6 Photographs for Revision J
Plan

no cut off wall

slope paving

Section A-A

Rating curves

Original
Revision M

Material moved, %

Original
Revision M

Fig. A7 Data for Revision M
Photo 1 Overhead view of model. \( Q_p = 8000 \text{ cfs} \)

Photo 2 Overhead view of model. Note the more nearly parallel approach to the bridge opening. \( Q_p = 12000 \text{ cfs} \)

Photo 3 Overhead view of result of silt test. \( Q_p = 8000 \text{ cfs} \) Time for test: 120 minutes

Photo 4 Overhead view of result of silt test. Note badly scoured condition and extent of deposition outside of hole. \( Q_p = 17000 \text{ cfs} \) Time for test: 120 minutes

Fig. A8 Photographs for Revision M
Fig. A9 Data for Revision N
Photo 1 Overhead view of model. $Q_p = 8000$ cfs
Note confetti stopped by the westerly dike.

Photo 2 Overhead view of model. $Q_p = 12000$ cfs
Note rotating cell on the westerly side of hole.

Photo 3 Overhead view of result of silt test.
$Q_p = 8000$ cfs  Time for test: 120 minutes

Photo 4 Overhead view of result of silt test.
$Q_p = 17000$ cfs  Time for test: 120 minutes

Fig. A10 Photographs for Revision N
Fig. All Data for Revision J’
Photo 1 Overhead view of model. $Q_p = 8000$ cfs
Note the quantity of confetti stopped by the westerly dike.

Photo 2 Overhead view of model. $Q_p = 15000$ cfs
Note surface turbulence caused by overtopping of dikes.

Fig. A12 Photographs for Revision $J'$

Fig. A13 Comparison of scour curves in various revisions with the original condition

Fig. A14 Flood frequency curve at Bridge 3 - discharge vs recurrence interval

Fig. A15 Flood frequency curve at Bridge 3 - stage vs recurrence interval