The Hydraulic Design of Bridges for River Crossings—A Case History

ALVIN G. ANDERSON, University of Minnesota, St. Anthony Falls Hydraulic Laboratory

Following collapse of one of the I-29 bridges on April 1, 1962, a model study was performed to define hydraulic conditions that led to collapse and to test proposed protective works. The collapse was initiated by the undermining by erosion of one of the piers of the upstream bridge. A consequence of the erosion pattern was the subsidence of a portion of the left bank, which was left without support when the bed downstream of the bridges eroded to a considerable depth.

The erosion pattern was the result of at least four factors: (1) a flood discharge 1.64 times as large as the maximum flood prior to the design of the bridge was conducive to high velocities through the bridge section; (2) cohesiveness of the riverbed effectively prevented the formation of bed load because when the cohesive bond was broken the bed material went into suspension; (3) the low stage in the Missouri River was favorable to a still higher increase in velocity; and (4) the bridge geometry provided constriction and obstruction to flow and piers were skewed to direction of flow.

The proposal for stabilization of the left bank and prevention from further subsidence incorporated revetment protection for the bridge piers, which had been reconstructed with extensive foundation piles. Hydraulic model tests established the flow and erosion patterns resulting from the revised revetment and showed that the stabilization proposal would not be detrimental to the flow pattern through the bridge structure.

•THE I-29 highway leading from Iowa into South Dakota crosses the Big Sioux River about a mile and a half upstream of its confluence with the Missouri River near Sioux City, Iowa. At this point the river makes an approximate right-angle bend. Just upstream of the bend, two parallel bridges were constructed which crossed the river at an angle to the main flow in the channel. They were supported on webbed piers spaced 120 ft apart. The piers were numbered from the west (South Dakota) side beginning with the abutment so that the first pier from the west is designated as pier 2. Piers 3 and 4 for each bridge were founded on the river bottom proper. At normal flow the river was about 250 ft wide and with the mean bed level at about elevation 1071 ft. The footings of the piers at elevation 1066 ft were supported by piles 25 to 34 ft long.

On April 1, 1962, during a flood, the upstream bridge of the pier collapsed. The initial failure occurred about 3:30 a.m. and from the evidence available it appears to have been the result of the undermining of pier 3 (Fig. 1). This in turn resulted in the rupture of the adjacent spans with the consequent failure of pier 4 about 12 hr later. An outstanding hydraulic characteristic of the failure aside from the undermining of pier 3 was the development of a very deep and extensive scour hole downstream of the structure. In this area the bed scoured to a depth of approximately 40 ft, which was considerably deeper than the section at the bridge centerline.

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Figure 1. Upstream I-29 bridge after collapse of pier 3 and before collapse of pier 4; river flow is from left to right. (Photo courtesy South Dakota Department of Highways)

Figure 2. Subsidence of left bank following collapse of upstream I-29 bridge.

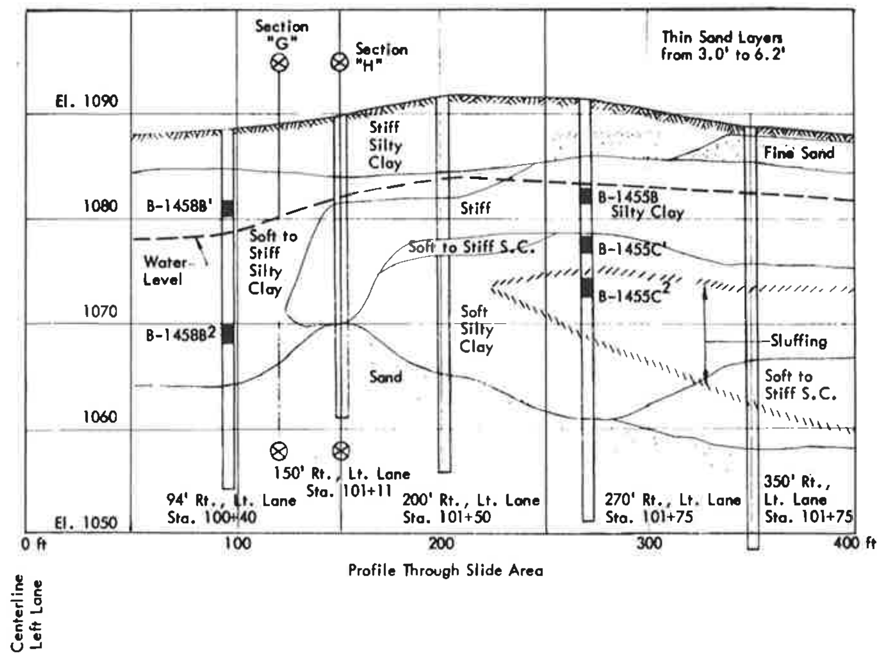


Figure 3. Soil profile through slide area south of bridge failure.

Immediately following the collapse of the bridge, pertinent data and related descriptions were collected by various agencies, including the Highway Departments of Iowa and South Dakota, the Bureau of Public Roads, and the Corps of Engineers. Much of the following information is based on reports and data relating to the hydraulics of the Big Sioux River collected by these agencies.

A secondary consequence of the conditions that caused the collapse of the bridge was the subsidence of the left bank just downstream of the highway spans and opposite the large scour hole that developed downstream of the structure. Figure 2 shows the area in which the subsidence occurred and the character of the settlement near the pier of the downstream bridge. Figure 3 shows the results of borings made in the left bank near where the subsidence occurred. The material labeled "soft silty clay" underlying the surface layer is significant. It might be concluded that the subsidence resulted from the removal of the lateral support of the left bank by the large hole eroded downstream of the bridge at the time of the collapse. The overburden then tended to squeeze the soft clay out into the river with the consequent subsidence of the bank itself. Some concern was felt at the time of reconstruction of the upstream bridge that the area of subsidence might be enlarged and endanger the nearby piers of the downstream bridge.

Hydraulically speaking, the essential characteristics of the replacement bridge were the increased span and the cylindrical piers that were used to support the bridge. The central span of the new bridge was such that the piers were essentially out of the channel itself, so that for ordinary flows there was relatively little obstruction to the passage of the water. Also, the flow past the cylindrical piers was independent of their orientation, and consequently the resistance or other effect of the piers on the flow pattern was the same for any direction of flow. To support the piers of the new bridge and to strengthen the piers of the remaining bridge, H piles were driven to a very considerable depth (to elevation 960-970 ft, respectively), certainly beyond any depth to which the river might reasonably be expected to scour. Upon completion of the bridge reconstruction and rehabilitation, the physical situation became one in which the two bridges located just upstream of a bend in the river consisted of the upstream bridge supported on cylindrical piers 225 ft apart and a downstream bridge on webbed piers having a central span of 120 ft so that the two central piers were located in the channel. In addition, the piers of the downstream bridge were aligned normally to the centerline of the bridge and consequently were skewed by about 25 degrees to the direction of flow in the main channel. A third characteristic of the situation was the subsidence of the left bank immediately downstream of the downstream bridge.

HYDRAULIC FACTORS INFLUENCING UNDERMINING OF PIER

The collapse of the upstream I-29 bridge was apparently the outgrowth of a number of causative factors which combined to create conditions antagonistic to the continued existence of the bridges. If any one of these factors had been inoperative there is a possibility that the collapse would not have occurred, but with all of them influencing the hydraulics of the flow through the bridge openings the destruction of the bridge was the result. The failure was due to the undermining of one of the piers by erosion of the streambed. The depth and extent of erosion depended upon the various factors in different ways.

Erosion or local scour is the consequence of a dynamic force created by flowing water upon a movable or erodible boundary or bed. Continuity states that the rate of erosion is equal to the difference between the rate of local transport and the rate of supply to the area. Symbolically this can be written as

$$\frac{d\,f(B)}{dt} = g(B) - g(s) \tag{1}$$

where f(B) represents the local bed elevation, g(B) is the rate of transport from the scoured area, and g(s) is the rate of supply into the scoured area. This expression can be used to describe many situations, including local scour. It states that when the rate of supply is equal to the rate of transport the bed is stable. The rate of supply depends on upstream conditions and may be equal to zero as is the case immediately downstream of a reservoir. The transport rate from the area of scour depends on the local boundary shear, which in turn depends on the velocity near the bed and the flow pattern or boundary geometry. The transport rate also depends on the character and resistivity of the sediment composing the bed. The bed sediment, whether it is cohesive or noncohesive, can be characterized by a critical boundary shear. If this critical shear is exceeded the sediment will move. At the same time the boundary shear acting on the bed will tend to decrease as the depth of scour increases because of changes in the flow pattern. The relative scour, and hence the undermining of the pier, then depends primarily upon the local velocity, the properties of the bed sediment, and the geometry of the system.

Discharge

The first unusual factor involved in the collapse was the flood discharge, which was larger than any previously recorded for the Big Sioux River. The maximum discharge at Akron, Iowa, the USGS Gaging Station, was 54,200 cfs. The previous peak discharge was 49,500 cfs in 1960. Previously the maximum discharge, which occurred in 1952, was only 33,000 cfs so that the flood of 1962 was 1.64 times as large as the maximum recorded discharge prior to 1960 and at the time of the original design. The frequency distribution of annual floods based on the records prior to the design of the bridge indicated that the 50-yr flood was about 42,000 cfs. This was approximately the discharge for which the bridge was designed. The frequency distribution based upon all of the records up to and including the 1962 flood is plotted in Figure 4, showing a distinct upward curvature as the maximum floods are plotted in relation to the previous record. These results suggest that the 1960 and 1962 peak discharges represent floods of a much longer return period than might be expected from the present record. It is also conceivable although extremely doubtful that some pronounced change may have taken place in the Big Sioux watershed so that the magnitude of the annual floods is now considerably greater than in the past.

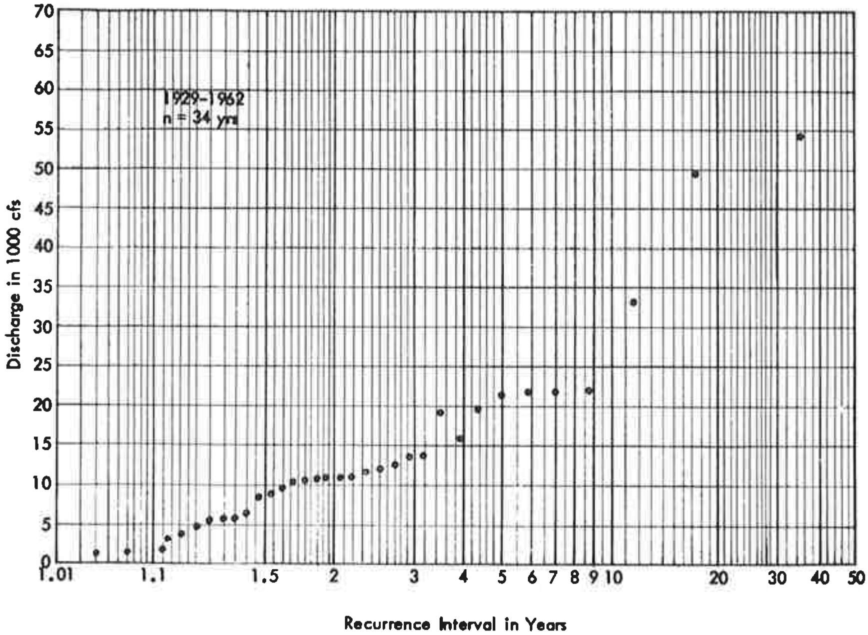
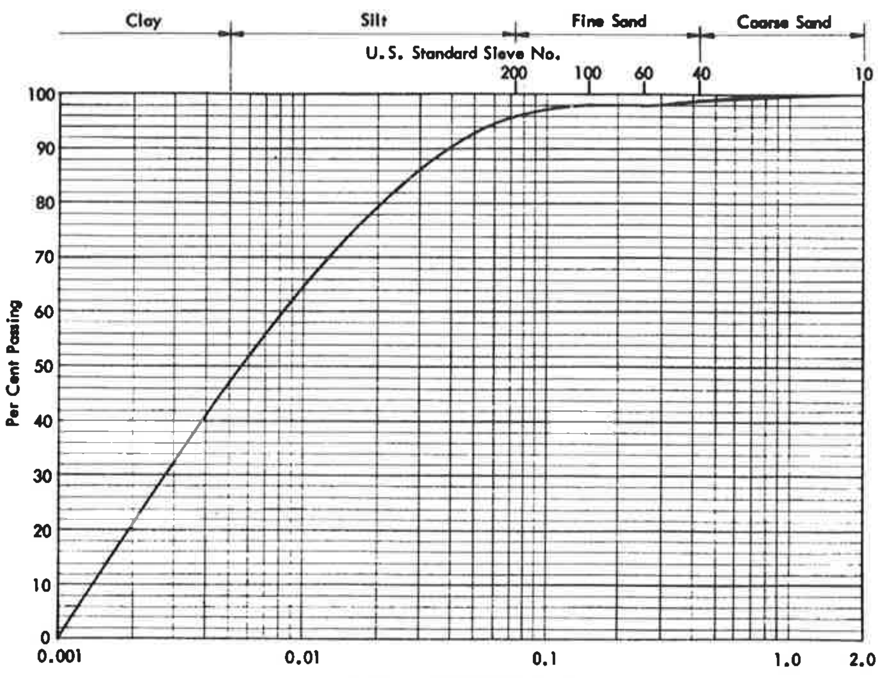


Figure 4. Flood-frequency curve for Big Sioux River at Akron, Iowa, 1929-1962.



Mat'l. Pass. No. 40			Class Name	P.R.A. Class	Color
L.L.L.	L.P.L.	P.I.	Clay	A-7-6(19)	Grayish Brown
57	27	30			

Figure 5. Size analysis of composite sample of Big Sioux bed sediment.

The role played by the flood discharge is that of increasing the velocity in the river as a whole and particularly in the constricted section caused by the bridges. The increase in mean velocity of the stream can be seen from a flow equation such as the Manning formula

$$V = \frac{1.49}{n} R^{2/3} s^{1/2} \qquad (2)$$

where V is the mean velocity, n is the roughness coefficient, R is the hydraulic radius, and s is the slope. Even if the slope remains constant the velocity will increase because of the increased hydraulic radius resulting from the enlarged cross-sectional area of flow. In addition a further increase in velocity approximately proportional to the discharge will result if the flow section is constricted, as at the bridge cross-sections. It follows then that the local velocity tending to cause scour around the bridge piers increases with the magnitude of the flood discharge.

Composition of Bed Sediment

The second factor involved in the local scour is the character of the bed sediment. In the Big Sioux River, it consists of material approximately 50 percent clay and 50 percent silt. The distribution curve of this material as obtained from a composite sample is shown in Figure 5. That the bed sediment of this composition is in a natural stream suggests very strongly that it possesses a high degree of cohesiveness and that the clay acts as a binder for the coarser particles. This implies that the resistivity of the bed to erosion depends primarily on the strength of the cohesive bond rather than the properties of the individual particles. While for noncohesive sediment the critical boundary shear at which movement begins depends primarily on the properties of the individual particles (size, shape, and density); for cohesive sediment the influence of size and other properties on the critical shear is masked by the greater bond strength. As a consequence greater force is necessary to erode initially the particles of a cohesive bed than for equivalent noncohesive beds, but once the bond has been broken the subsequent behavior of the bed particles in the flow depends on the particle properties only.

When shear forces are large enough to erode cohesive material, they are also large enough to carry the individual particles immediately into suspension and transport them through the section as part of the water complex. This suggests that for cohesive sediments there is relatively little material transported near the bed as bed load and that the rate of supply—g(s) in Eq. 1—into the cross-section where the bridges are located approaches zero. This means that the rate of scour depends only on the rate at which sediment can be transported from the scour hole and that the erosion will stop and the bed will become stable when the rate of transport out of the scour hole approaches zero. This will happen when the gradually decreasing boundary shear has been reduced to the critical shear for the bed material in the scour hole. It is clear then that size and depth of the hole will depend on the strength of the cohesive bond. One expects therefore that if the sediment of the riverbed had been noncohesive so that bed transport could have occurred, the local depth of scour at the obstruction would have been somewhat less than that actually generated.

Control of Stages on Missouri River

Before completion of the control structures on the Missouri River, the stages of the Missouri were the consequence of the normal runoff from the watershed basin. These stages more or less coincided with the flood stages of the tributary streams and a relatively high stage of the Missouri created backwater stages on the tributary streams. Thus the depths in the tributaries were greater but the slopes were much smaller so that high stages on the main stem reduced the flood velocities and hence the erosion potential. With the completion of the reservoirs on the Missouri, the stage at any point, and particularly those points downstream of the system of reservoirs, was de-

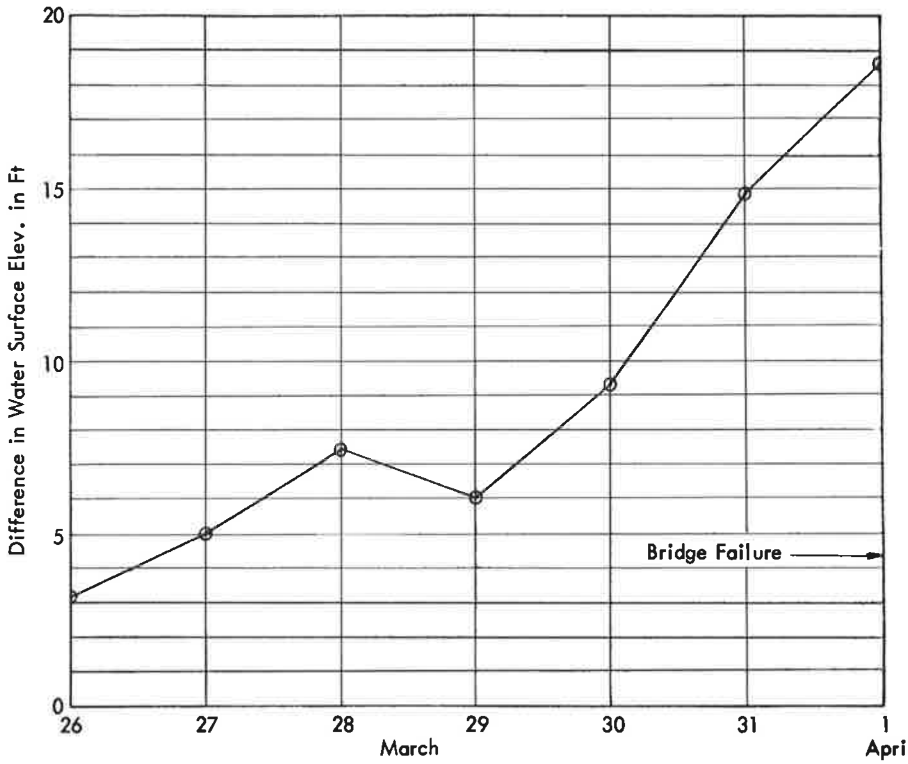


Figure 6. Difference in water surface elevations between Highway 77 bridge at Big Sioux River and combination bridge of Missouri River at Sioux City for week preceding bridge failure.

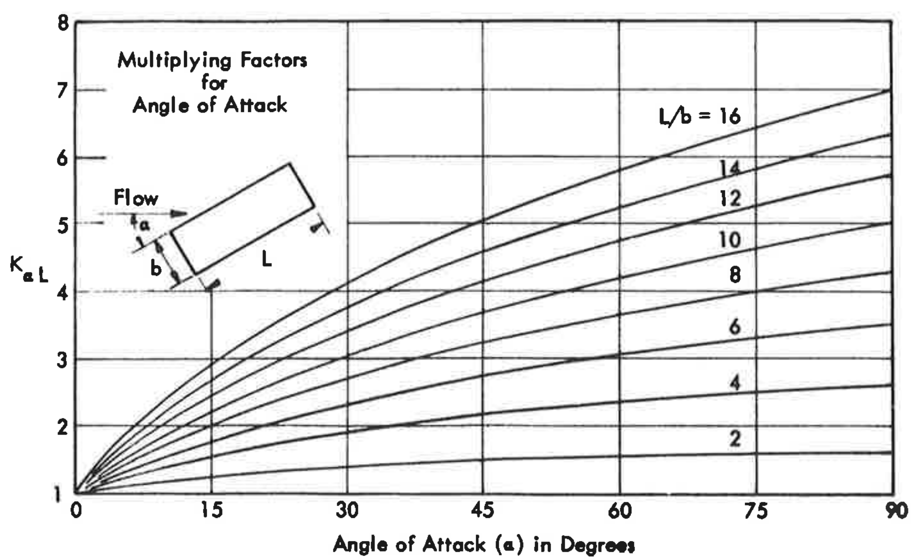


Figure 7. Design factors for piers not aligned with flow.

pendent on the artificial release of water from the upstream reservoirs and thus was relatively independent of the stages of the smaller tributaries during the flood season. In the case of the Big Sioux River during the 1962 flood at Sioux City the difference in elevation between gaging stations on the Big Sioux and on the Missouri increased from about 3.2 ft on March 26, 1962, to 18.6 ft on April 1, 1962 (Fig. 6). In this particular instance the stage of the Missouri remained relatively constant while that of the Big Sioux during the course of the flood increased very appreciably.

The consequence of the increased gradient or surface slope is a higher velocity than would be the case with a smaller slope. Visual observations of the flow in the Big Sioux shortly after the collapse of the upstream bridge indicated a velocity in the neighborhood of 13 ft/sec and, although the flood exceeded the design flood by a considerable amount, the stage at the bridge was still below that for which the bridge was designed. The high velocity resulting from the low stage of the Missouri tended to increase the shearing or erosive forces on the bed of the Big Sioux beyond that normally expected and hence subjected the bridge area to larger scouring forces.

Geometry of Piers

A fourth factor involved was the geometry of the piers supporting the bridges of the Big Sioux crossing. These piers consisted of two main columns with an intervening web to form a unified structure. Two of the piers of each bridge were founded in the river and constituted an appreciable obstruction and constriction to the flow. In addition the piers were constructed at right angles to the bridge centerline which was considerably skewed to the direction of flow. The angle of attack of the bridge piers to the mean flow was approximately 25 degrees.

Research on the erosion around bridge piers (1) has shown that the angle at which a pier is aligned to the flow is very significant in the extent of erosion that takes place around it. The two factors involved are the angle of attack and the length-breadth ratio of the pier. These results are shown in Figure 7 (1). Thus for an L/b ratio of 10 and an angle of attack of 30 deg, one could expect about three times as much scour as would occur if the pier had been aligned to the flow. For a cylinder in which the L/b ratio is unity, the multiplying factor is also unity since all orientations are equally favorable. Figure 7 clearly shows the importance of alignment of the piers in causing potential scour. It has been stated that the skewed alignment of the piers of the bridge was in anticipation of remedial work to be done on the Big Sioux River in which the reach through the bridges would be straightened by changing the location of the confluence of the Big Sioux with the Missouri River. If this change had been made, the degree of skewness or the angle of attack of the flow would have been considerably reduced.

The several factors affecting the stability of the bridge which have been enumerated are not, in general, subject to modification except perhaps the alignment and spacing of the piers. Aside from these factors it appears that an unfortunate combination of conditions led to the erosion. The value of analysis and of the availability of sufficient data lies in making provisions in the design for such contingencies.

The upstream bridge was reconstructed with cylindrical piers spaced 225 ft apart so that the flow obstruction was greatly reduced. The downstream bridge was strengthened by underpinning. For both bridges additional H piles were driven to provide ample support for the piers from further scour. In addition riprap and timber piles were placed on the left bank to prevent further subsidence of the bank and to protect the piers of the downstream bridge.

THE MODEL AND ITS VERIFICATION

To test the adequacy of these works in relation to their influence on the flow pattern, a model study was conducted at the St. Anthony Falls Hydraulic Laboratory. The studies were performed in an undistorted 1:75 scale movable bed model. The reach covered by the model consisted of a length of approximately one-half mile upstream and downstream of the bridges, and incorporated the bend just downstream of the bridges. Figure 8 is a general view of a portion of the model showing the bend. The banks of the model stream with the exception of the region near the bridges were made

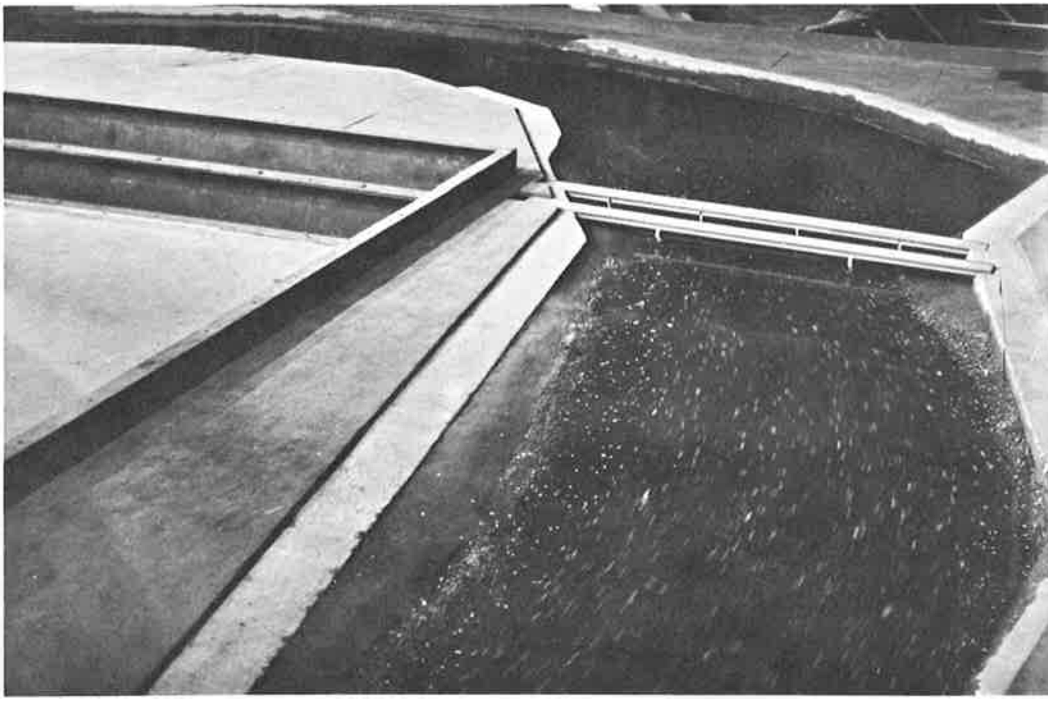


Figure 8. Overall view of Big Sioux model showing location of reconstructed bridges and the Big Sioux Bend downstream.

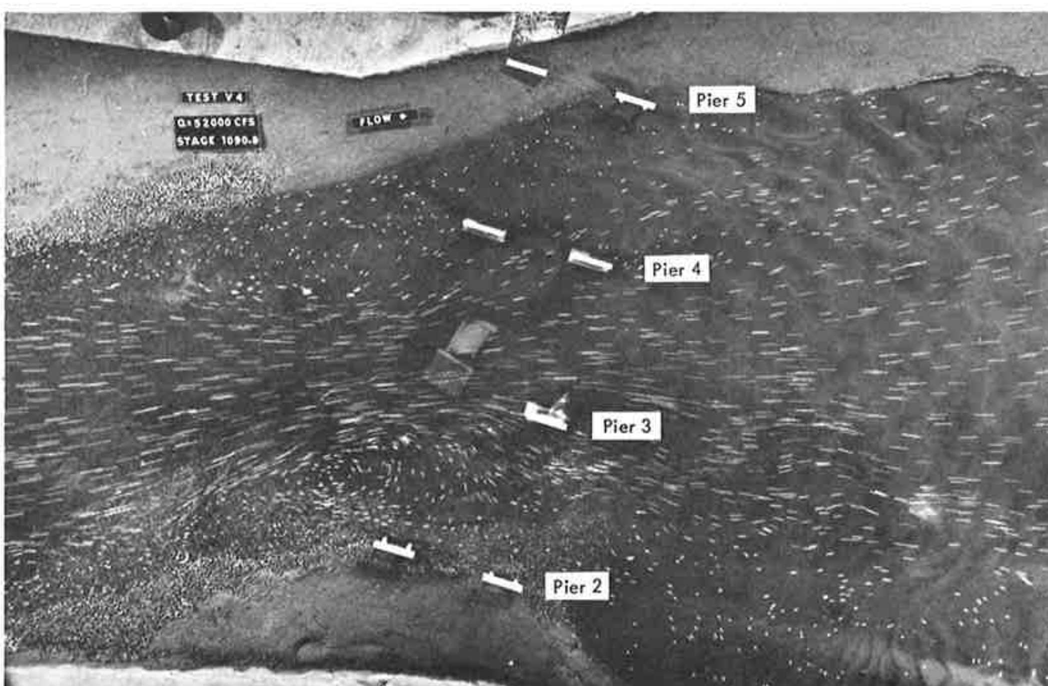


Figure 9. Verification test of hydraulic model showing collapse of pier 3 due to undermining; flow pattern delineated by confetti on water surface.

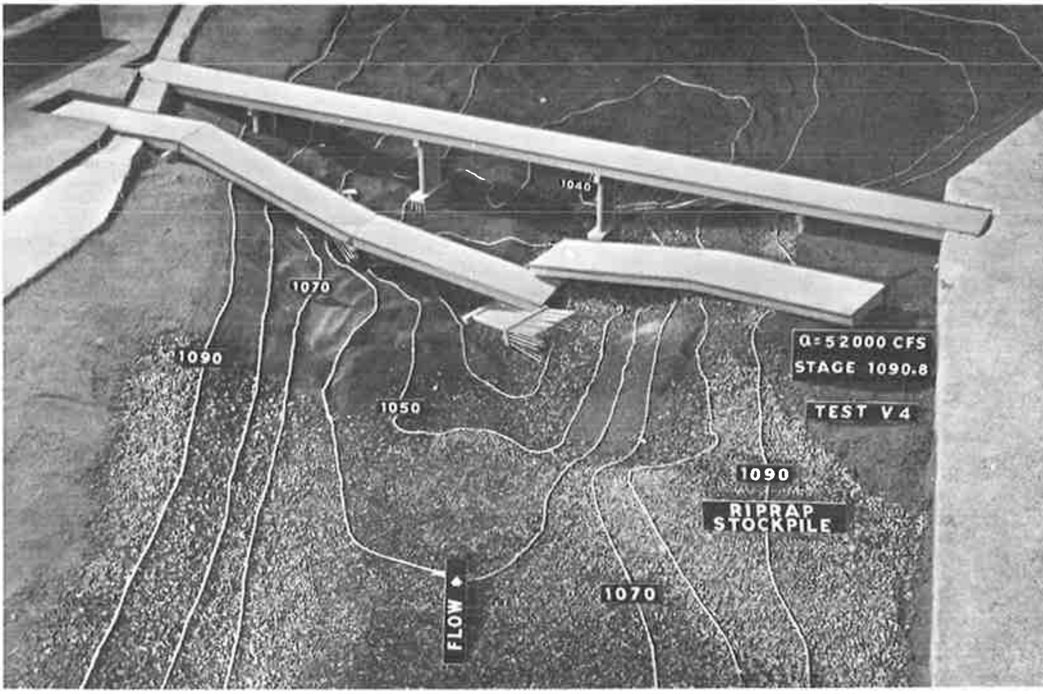


Figure 10. Erosion pattern following verification tests.

rigid so as to define the channel. The bed of the model consisted of a fine sand approximately 0.2 mm in size (initially, no attempt was made to simulate the cohesiveness of the upstream bed). The bridge piers were constructed to scale with the piles under the piers also made to scale.

In situations involving local scour the observed scour geometry is the result of the action of two types of forces. The flow of the water is a gravitational phenomenon while the removal of sediment from the scour hole depends on the size of the sediment and on the bed shear forces generated by the flow. These are complex functions of the viscous properties of the fluid. Generally it is not possible to satisfy exactly these two conditions at the same time in a small-scale model. In practice the model is designed and operated according to the gravitational requirements (Froude law) with the size of the model sediment adjusted so that qualitatively similar scour patterns will be developed. Because of the uncertainty introduced by the combination of two or more of these criteria recourse is had to the so-called verification process, wherein the model is operated as dictated by the gravitational requirements and then successively adjusted until it duplicates some significant observed event in nature. When this verification has been achieved so that qualitatively, and sometimes quantitatively, similar scour patterns are developed, it is assumed that the model will also simulate the scour pattern generated by a modification of the flow geometry.

The initial verification tests in the model were designed to determine its ability to simulate the conditions in the prototype that existed prior to the failure in order to duplicate the failure as it occurred in nature. The initial tests were made with the original piers with the model subjected to a flow corresponding to the 1962 flood. Initially, however, the model pier was not undermined as it should have been. The reason was that the bed of the model consisted of a noncohesive movable sand which as it was transported down the channel served to replace that which was removed around the pier so that the depth of scour was diminished and the system reached an equilibrium at a stage such that failure would not occur. It was therefore necessary to simulate the cohesiveness of the upstream bed material so that it would not contribute to the bed transport. Since the cohesive sediment in the prototype would have gone into suspension upon the rupture of the cohesive bond, it would have played no part in the stability of the structure.

This result could be duplicated by preventing the erosion of the material upstream of the bridge. This was done by placing a thin layer of gravel on the bed upstream of the structure. This fine gravel was of such a size that it would not be moved during the flood. No gravel was placed in the region around the piers so that local erosion could occur and the bed regime could then correspond to that which was found in nature. By doing this, it was possible to duplicate the failure of the upstream bridge through the collapse of pier 3 and the creation of a large and deep scour hole downstream of the bridges. The flow pattern and the submerged pier for the verification test are shown in Figure 9, and the erosion pattern resulting from the flow is shown in Figure 10. After the pier collapsed the bridge spans were put in place in order to develop the final scour pattern as it occurred in nature (Fig. 10). The simulation of the conditions for the failure of the upstream bridge represented the verification of the model, and it was assumed that new patterns observed in the model would correspond to those in the prototype for the same conditions.

TESTS OF PROPOSAL FOR STABILIZATION OF LEFT BANK

On completion of the verification tests and before modification of the model was undertaken to incorporate the initial corrective measures, additional study by the Iowa State Highway Commission of the stability of the left bank resulted in a revised proposal. (It should be borne in mind that two problems existed at the bridge site: one was the safety of the bridges from undermining and the other was the stabilization of the left bank—the solution of the second problem insofar as riprap and other revetment

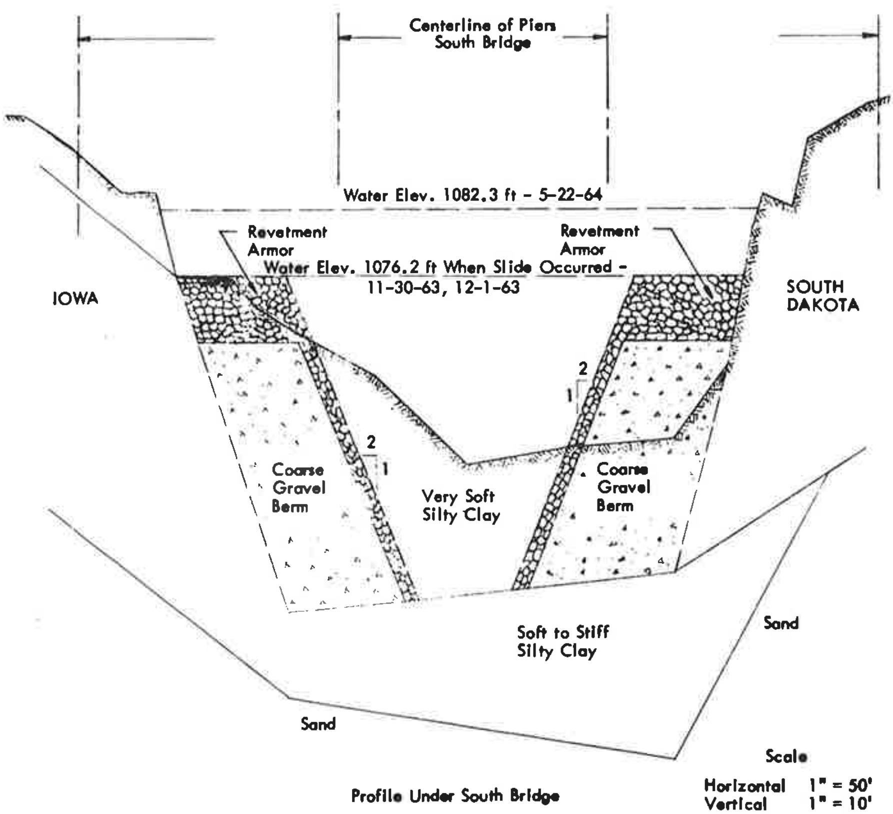


Figure 11. Stabilization proposal of Big Sioux channel for I-29 bridges.

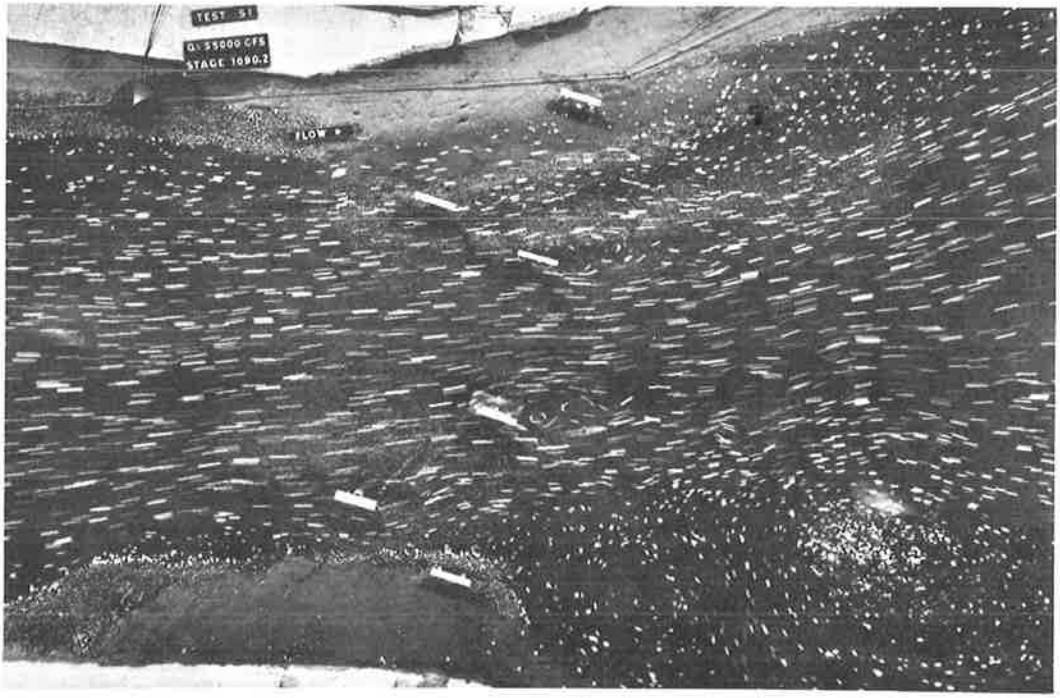


Figure 12. Flow pattern through bridge section with reconstructed bridges and with proposed berms and revetment in place.

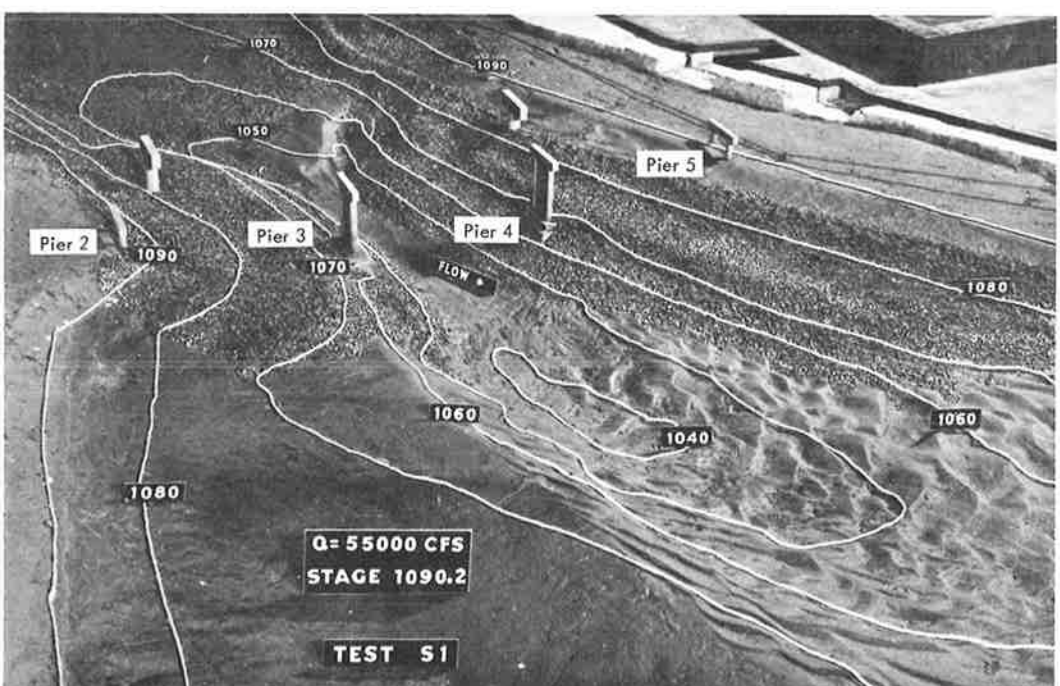


Figure 13. Erosion pattern resulting from flow through reconstructed bridges and protective works.

Figure 14. Flow pattern through bridge section with reconstructed bridges and proposed berms and revetment for maximum discharge simulating very low stage in Missouri River.

Figure 15. Erosion pattern resulting from maximum flow and low tailwater.

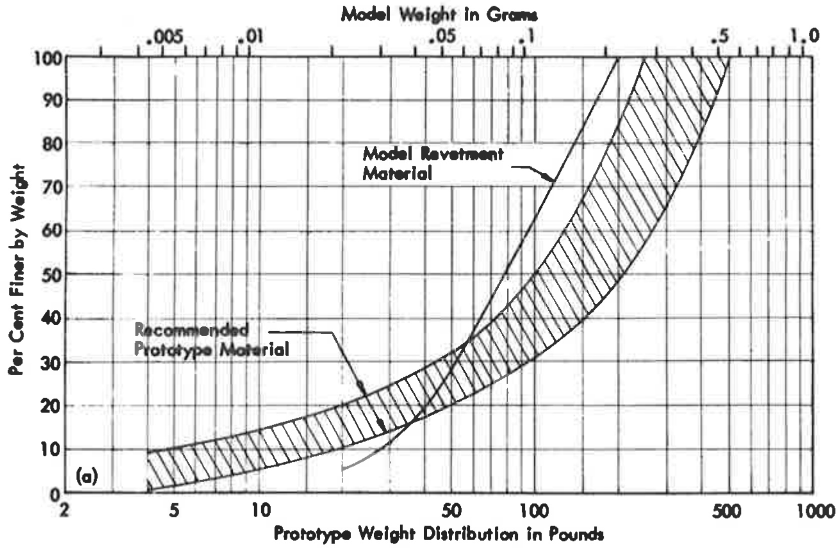


Figure 16. Revetment material weight distribution.

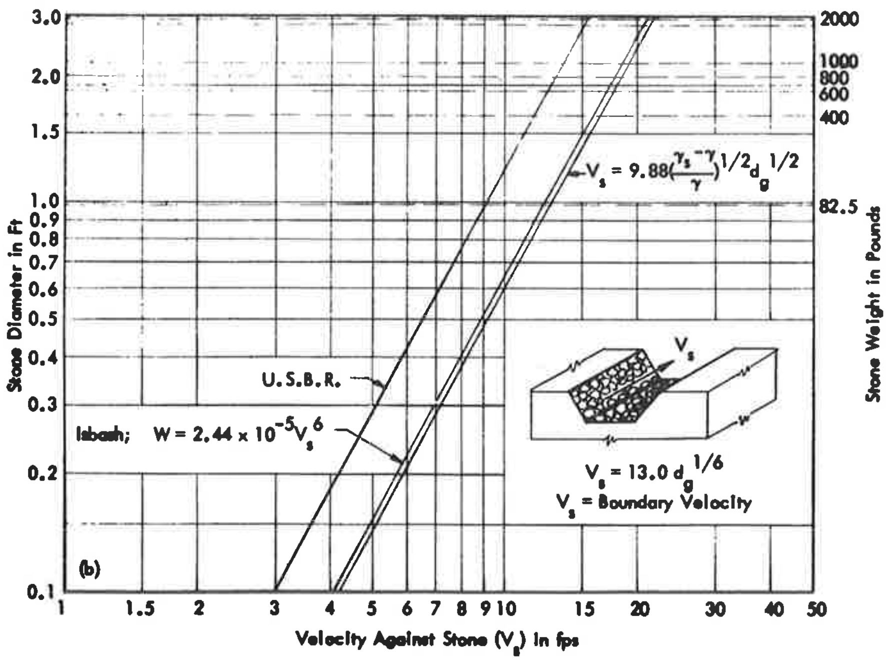


Figure 17. Critical velocities against stones.

was to be used might also constitute a solution for the first.) Stability studies indicated that the material required to stabilize the bank would have to be much more extensive than that previously proposed. Based upon this study the highway department proposed the plan shown in Figure 11. It consisted of rather extensive, coarse gravel berms protected by riprap revetment. These berms and revetment were to be placed on both sides of the river at the bridge cross-sections and extend downstream on the left bank to encompass completely the area of subsidence. It was apparent that the berms and revetment would appreciably encroach upon the waterway area, although the depth to which it was proposed to carry the protective works was considerably below the original stream bed elevation.

If the necessity of the bank stabilization is accepted, the hydraulic problem then becomes one of determining the influence of the proposed protective measures on the flow and the effect of the flow upon the berms. Since the proposed berms and revetment also encompassed the two central piers of the downstream bridge, the problem of undermining the pier was no longer relevant as long as the proposed riprap could not be moved by the flood flows.

The model was modified to incorporate the proposed berms and revetment, the new piers for the upstream bridge, and the additional pile underpinning of the downstream bridge. Tests were made at a discharge of 55,000 cfs and were adjusted so that the stage at the bridges would be the same as that measured during the 1962 flood. The flow pattern that developed for these conditions is shown in Figure 12. The streaks on the surface are the images of confetti particles recorded over the exposure period. The elapsed time is recorded by the motion of a white streak on the disk revolving at a rate of one revolution per second. The elapsed time in this case was about $\frac{1}{5}$ sec. The photograph also clearly delineates the wake downstream of the skewed piers of the downstream bridge and the large clockwise whirl that developed at the outside of the bend downstream of the bridges.

One measure of the influence of the proposed corrective works is the increase or decrease in upstream stage as compared to conditions without the changes. One of the questions that was raised was the possibility of increased flooding upstream. Observations in the model showed that the stage was actually lowered after the stabilizing berms had been installed; the reason for this is shown in Figure 13, which shows the erosion pattern caused by the flow illustrated in Figure 12. The central portion of the channel that is unprotected by riprap has been eroded to enlarge the flow section, but the riprap protecting the banks and the piers has not been disturbed. The depth of scour downstream of the piers should be noted. Scour of this magnitude and location was observed in the prototype. In this case, however, the excessive scour in this area appears to be due to the action of a relatively high velocity jet through the bridge piers impinging upon the unprotected bed downstream. In this study it was assumed that the riprap to be used in the prototype would be of such a size that it would not be moved by the flow. The size required cannot be established by the model experiments but must be determined by other means. Given that the revetment would not be moved, the model revetment was chosen so it also would not be moved by the model flow. As a consequence of this, the flow pattern generated in the model would be similar to that generated in the prototype, and so also the scour pattern developed in the model will be at least qualitatively similar to that which would be developed in the prototype.

These results were the outcome of experiments in which the stage at the bridges was maintained at a level corresponding to the stage measured in the prototype. It appeared desirable to observe the effect on the bridge protection of a still lower stage as would be the case if the stage of the Missouri were artificially lowered. Such conditions are shown in Figures 14 and 15. Figure 14 shows the flow pattern for a discharge of 55,000 cfs when the stage at the bridge has been reduced to elevation 1085.5 ft. The separation zones are more distinct and the strength of the downstream whirl has increased. The jet effect caused by the piers of the downstream bridge is clearly shown as well as the high velocities initiating vortices at the inside bank of the bend. The erosion pattern created by this flow is shown in Figure 15. For these extreme conditions some of the riprap has raveled into the channel between the piers and in a downstream direction. The jet (Fig. 14) has eroded an excessively deep hole downstream of the piers. In spite of this, however, the piers themselves appear to be unaffected by the flow.

The effectiveness of the riprap revetment in protecting the piers depends on its ability to resist the dynamic forces generated by the flow. Experiment and experience have established a relationship between the velocity near the boundary and the size of the riprap necessary to resist movement. Figure 16 shows the size distribution curve of the model sediment transferred to prototype sizes. This material was stable in the model. Using the graph in Figure 17 as a guide and considering the practical limits of quarry production, a recommended riprap size distribution is also shown in Figure 16 for comparison with the model riprap.

The hydraulic model studies of the flow pattern with the proposed gravel berms and riprap revetment indicate that the protection provided will be adequate for flood discharges considerably larger than those recently experienced if the field construction is similar to that tested in the model. The studies also serve to demonstrate the value of hydraulic model studies combined with analysis and field data as an adjunct to the design of bridges over waterways and other hydraulic structures.

ACKNOWLEDGMENTS

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