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Report on

STUDIES OF THE STABILIZATION OF THE BIG SIOUX RIVER AT THE INTERSTATE 29 BRIDGE CROSSING

by

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INTRODUCTION

This report describes the results of model studies of the Interstate-29 bridge crossing of the Big Sioux River near Sioux City, Iowa. These studies were concerned with the testing of the provisions for protection of the bridges from scour during future floods and the stabilization of the banks from further subsidence. Interstate-29 crosses the Big Sioux River about 1.5 miles above the confluence of the Big Sioux with the Missouri River. A portion of the Big Sioux River and the I-29 bridge crossing is shown in Photo 1, an aerial view of the site. The photograph shows the bend of the river in the neighborhood of the bridge and the alignment of the bridge with respect to the current of the river.

As a consequence of a flood on April 1, 1962, one of the piers of the upstream bridge was undermined by scour and the bridge collapsed. A view of the fallen bridge is shown in Photo 2. Of particular interest, from the hydraulic point of view, was the development of a relatively large scour hole downstream of the two bridges. The precise reason for the occurrence of this scour hole in this area is not clear, although the wake downstream of the piers of the downstream bridge, the higher velocity between the piers of the downstream bridge, and the presence of the collapsed upstream bridge in the channel undoubtedly were factors in causing the excessive scour. Following the flood and during the construction work involved in the replacement of the collapsed bridge and the additional underpinning of the remaining bridge, it was observed that a sizable portion of the left bank downstream of the remaining bridge subsided, as shown in Photo 3. The extent of the subsidence and its approach to the piers of the remaining bridge raised the fear that this pier might be damaged. The model studies were undertaken to investigate and suggest solutions for two conditions: (1) to provide protection for the piers of the reconstructed bridges from damage or undermining in the event of future floods, and (2) to stabilize the banks in order to prevent further subsidence that could endanger the piers.

This report will include: (1) a description of the model and its verification, (2) the results of a study of the flow characteristics in the region of the piers before and after the corrective works had been installed, and (3) a study of the influence of groins along the right bank of the river upstream of the bridge on the flow pattern of the water passing through the bridge section.

DESIGN AND DESCRIPTION OF HYDRAULIC MODEL

The model was constructed to a linear scale ratio of 1:75 and incorporated a section of the Big Sioux River from a point approximately 1/2 mile upstream of the bridges to a point approximately 1/2 mile downstream of the bridges. This length of river was included in order to provide the proper entrance conditions as well as to incorporate the pronounced bend of the river immediately downstream of the two bridges. The model was a so-called movable bed model with the side walls molded in concrete to conform to the general configuration of the river. The bed consisted of sand obtained from the Missouri River with a mean size of 0.18 mm. This size sand was chosen, based upon past experience, as a sediment which would move and permit scour to develop for such discharges as would exist in the model. Chart 1 shows the general layout of the hydraulic model including the centerline of the two bridges and the location of the piers for each.

The water supply came directly by gravity from the laboratory supply flume through a 12-inch pipeline and was controlled by a hydraulically operated 12-inch valve. The discharge was measured by a calibrated orifice in the supply line. At the downstream end of the model was a sediment trap and tailgate. The sediment trap was for the purpose of collecting sediment that had been transported out of the model during the course of the experiments. By means of the tailgate, the water surface elevation in the neighborhood of the bridge piers could be controlled or manipulated to represent any prescribed stage of the Missouri River. Prior to any experiment the sand bed could be and was molded to correspond with the bed elevations as measured in the field.

The model was designed to be operated in accordance with the Froude criterion for similarity since such open channel flows or rivers function as a consequence of the force of gravity. Since water is the fluid medium in both instances, the following ratios relate occurrences in the model to those in the prototype, that is, the river itself. These ratios are given in terms of the arbitrarily chosen length-scale ratio.

SIMILARITY RELATIONSHIPS IN MODEL TESTS

Model Property		Ratio
Lengths	ft	$L_r = L_r = 1:75$
Areas	sq ft	$A_r = L_r^2 = 1:5625$
Velocities	fps	$V_r = L_r^{1/2} = 1:8.66$
Discharges	cfs	$Q_r = L_r^{5/2} = 1:48,712$
Tractive Forces	psf	$\tau_r = L_r = 1:75$

Because the transport and scour of sedimentary materials is dependent upon the characteristics of the prototype sediment and because the laws of similarity for sediment transport involve other forces in addition to the force of gravity, the similarity relationships are considerably more complicated and not well understood. For this reason as well as the necessity of adjusting the model to obtain similarity of flow pattern in all its important features, the model must be verified by comparing its operation and results with known conditions and flows in the prototype. Since the collapse of the upstream bridge as the result of scour around one of the piers, which occurred during the extreme flood of April, 1962, was the reason for these model studies, this was obviously the event to be compared in the model and the prototype in the verification process. Another reason for adjusting the model to correspond to this event was that a considerable amount of field survey data was available as a consequence of the catastrophe, and in addition certain other observations were made by observers who were present shortly after the collapse. Therefore, in preparation for the test series to be undertaken to study the stabilization scheme for the banks of the Big Sioux River, a number of model tests were made for the purpose of establishing the correct hydraulic geometry and the mode of operation in the model. The criteria for these tests were the comparison of model data with various data collected at the site during and after the flood. These data included water surface and bed profiles, bed cross-sections, hydrologic records, and photographs taken during and after the collapse of the north bridge. Some photos and other data for pre-flood conditions were available for use in establishing the original model geometry.

In a preliminary test the banks of the river were positioned approximately in accordance with pre-flood photos and available data. The bed was molded to Elev. 1071 and the piers and associated piling for the original bridges were put in place in the model. The model was run continuously for 48 hours with periodic observation of developments. The stage at the bridges was maintained at Elev. 1090.8, as observed in the prototype. The test was marked by severe bank erosion and almost negligible bed degradation. It was concluded from this test that the upstream reach of the model would have to be stabilized in order to preserve the correct approach geometry and to eliminate the bed load which was tending to reduce the depth of scour in the test section. A second test was made in which a layer of fine gravel was placed on the upstream bed and banks of the model down to a section located 10 ft upstream of the crossing. The riprap stockpile as found in the prototype was placed on the right bank about 200 ft upstream of the bridge so that it would ravel into the stream as the bank erosion proceeded. The right bank in the vicinity of the bridges was also armored with fine gravel in an attempt to duplicate the armor placed on this bank after the flood of 1960. A flow of 55,000 cfs was established in the model and maintained for two hours. The left bank (in the vicinity of the bridges) had scoured to a maximum of about 13 ft at the bridge piers. It was apparent that under these conditions the bed would not erode enough to undermine a pier. After some consideration of the field conditions (geometry and nature of the bed material) it was concluded that in nature there would be no bed load entering the bridge section. The bed material in the river is approximately 50 per cent clay and 50 per cent silt which form a cohesive bed. Erosion then depended upon the bond between the particles which was much larger than the critical tractive force of the individual particles. When erosion occurred these particles were immediately carried away in suspension and did not contribute to the rate of supply into the area undergoing scour.

In a river with a non-cohesive bed a considerable quantity of sediment is transported as bed load. This bed load has a pronounced effect on the scour pattern which will form in the vicinity of an obstruction. Thus the scour hole will deepen only until equilibrium is reached between the transportation into and out of the section. If no sediment is being transported to supply the area undergoing scour, the depth of scour will then depend upon the size of the sediment making up the bed in this vicinity and the velocity pattern inducing the scour. In order to simulate the condition in the prototype of no bed load transport into the region of the bridges, the upstream portion of the model was stabilized by extending the fine gravel down to a point only 2 ft upstream of the site. This represented the condition in the prototype where any erosion that took place upstream of the bridge became suspended load and was transported in suspension through the critical section.

Following these preliminary adjustments of the model, additional experiments were made to verify that the model performed as the prototype.

Verification Test V-1

The first verification test was undertaken with the bed upstream of the bridge stabilized and the banks in the vicinity of the bridge positioned in accordance with post-flood data. The bed was molded to Elev. 1071 ft. When the flow was established (55,000 cfs) it was noticed that the angle of skew of the flow to the piers was not quite as great as observers at the site had reported. To correct this situation the left stabilized bank upstream of the bridges was extended out into the flow during the run until the angle of skew of the flow had been adjusted. After 3 hours of operation, pier 3 of the downstream bridge collapsed as shown in Photo 4. The fallen pier and the erosion pattern around pier 3 of the upstream bridge is shown in Photo 5. The erosion near pier 3 of the upstream bridge was also quite severe so that it appears to be a matter of chance as to which of the piers would fail first. Charts 2 and 3 document this erosion test.

Chart 2 shows the plan and profile along the longitudinal centerline while Chart 3 shows cross-sections taken at the upstream and the downstream bridges. The bed in this region was somewhat littered with fine gravel (contamination in the Missouri's sand plus ravelled materials from the stabilized regions upstream of the bridges) which, no doubt, impeded the bed erosion. In Chart 3 the comparison between the cross-sections generated in the model and that in the prototype shows that the right bank suffered considerably more erosion in the model than in the prototype. At pier 3 the model erosion is nearly 20 ft greater. At the center of the river the opposite is true, the model erosion being 10 to 15 ft less. The cross-section along the centerline of the upstream bridge shown in Chart 3 also indicates that the erosion in the model was considerably less than that in the prototype in this region. If the additional restriction caused by the fallen superstructure had been present in the model, the model and prototype erosion would probably have been more comparable.

Verification Test V-2

The model was remolded and the riprap stockpile on the upstream right bank was installed. Photo 6 shows the flow pattern in the model 2-1/2 hours after the start of the test. The riprap stockpile deflected the flow toward pier 3 of the upstream bridge, which collapsed shortly before the photograph was taken. The general flow pattern, separation zone, deflection from the riprap stockpile, and the direction of the high velocity stream seem to be in good agreement qualitatively with the prototype flow pattern as obtained from field photographs. Photo 7 shows the fallen pier after the flow had been stopped. The bridge decking consisting of spans 1, 2, and 3 and the fallen piers 2 and 3 were introduced into the model and positioned on the bed as indicated by field observations. The flow of 55,000 cfs was reestablished and maintained for about 4-1/2 hours. At this point, pier 4 and span 4 were atrificially collapsed to represent the event in the prototype. The flow pattern for these conditions is shown in Photo 8, after which the test was continued for another 2 hours and Photo 9 was taken to show the resulting scour pattern.

Chart 4 is a plot of the erosion contours and longitudinal profile at the completion of the experiment. The cross-sections measured at the upstream and downstream bridges are shown in Chart 5. In Chart 4 the scour pattern through the bridges shows that the erosion degraded the bed to Elev. 1040. The cross-sections presented in Chart 5 show that the two sets of model data, that is, with piers alone and with superstructure, bracket the prototype data quite well, and that the best agreement was obtained at the upstream bridge. The agreement in regard to shape of the eroded cross-section is rather good. It appeared that if the stabilized banks in the model were adjusted slightly the agreement would be even better.

Verification Test V-3

Before further tests were made in the undistorted model, a test was made to determine the effect of a distorted model on the erosion pattern and the rate of erosion. This was done by increasing the discharge but maintaining the same stage at the bridges by adjusting the tailgate. A flow of 65,000 cfs was established and the tailwater lowered to obtain the lowest possible stage at the bridges. This was found by measurement to be Elev. 1092.4 ft. For this condition, erosion at the site of the bridges was extremely severe. In about 10 minutes pier 3 of the upstream bridge fell. After 2 hours pier 3 of the downstream bridge also collapsed. The results of these tests are shown in Charts 6 and 7. Chart 6 shows the erosion contours and the longitudinal profile along the centerline and shows that pier 3 of the upstream bridge was completely undermined and that pier 3 of the downstream bridge was undermined sufficiently to be unstable. The corresponding cross-sections for the condition described in this test are shown in Chart 7. These indicate that the piers would be quickly undermined. Based upon this test, it was concluded that distortion of the model would not be necessary in order to obtain a good comparison between the model and the prototype. The experiment did indicate, however, the character of the erosion that could occur for this extreme discharge and high slope compared to the flow conditions for the 1962 flood of 52,000 cfs and water surface Elev. 1090.8 at the bridge crossing.

Verification Test V-4

Test V-4 is similar to verification test V-2. The stabilized right bank in the vicinity of the bridges was adjusted slightly and the model remolded. Photo 10 shows the bridge crossing and the bridges prior to the test. Additional information from the field indicated that the maximum discharge was 52,000 cfs so that test V-4 was carried out using this slightly lower maximum discharge. The flow pattern through the bridge piers after pier 3 had fallen, in about 1-1/2 hours from the beginning of the test, is shown in Photo 11. The erosion pattern that corresponds to this operation is shown in Photo 12. Following the collapse of pier 3 the superstructure of the bridge was placed on the bed to correspond to the arrangement in the prototype and the experiment continued for a total time of 10 hours. Photo 13 shows the flow pattern around the superstructure submerged in the flow. The corresponding scour pattern after the test had been stopped is shown in Photos 14 and 15. Photo 14 shows the dry bed with the superstructure placed as during the test. Photo 15 shows the bed itself after the superstructure had been removed in order to show more details of the erosion pattern. A plan view of the erosion pattern and contours, and a longitudinal profile along the centerline for this test after 10 hours of operation with the piers and the superstructure submerged in the flow, are shown on Chart 8. It is apparent from the longitudinal profile that the upstream pier 3 had been completely undermined, but that pier 3 of the downstream bridge was still relatively stable even though the footing had been exposed. The cross-sections for these conditions shown in Chart 9 more clearly elucidate the situation and the degree of similarity between the model and the prototype. The cross-sections taken along the downstream bridge show good agreement with the prototype measurements. This is also true for the cross-sections at the upstream bridge when the cross-section obtained in the model for both the piers and the superstructure in the channel is compared with that obtained in the prototype.

Conclusions - Verification Tests

Upon the completion of verification test V-4, it was concluded that the model when operated in accordance with this test corresponded reasonably well with the prototype under similar conditions. On this basis it was felt that the model could be used to simulate other conditions in the prototype such as the study of the bank protection works and the provisions for the control of bank subsidence.

The verification tests were also valuable in demonstrating in general the manner by which the collapse of the prototype bridge occurred and the circumstances surrounding this failure. It may be concluded that the collapse of the bridge was partially due to the constriction created by the bridge piers and the consequent increased velocity in this region. However, this condition was considerably enhanced by the nature of the bed material that formed the bed of the river. The fact that this bed material was cohesive in character and extremely fine in size caused any eroded material to go into suspension. If the bed had been of non-cohesive sand which could be transported as bed load, this would have had the tendency to decrease the maximum depth of scour, possibly by a sufficient amount to prevent the bridge from collapsing.

TESTS OF PROTECTIVE MEASURES

These experiments were concerned with the hydraulic effect of the proposed revetment on the water surface profiles and erosion patterns in the neighborhood of the bridges.

The flood in April of 1962 caused a considerable amount of scour between and downstream of the bridge piers. Subsequently, a portion of the east bank downstream of the downstream bridge subsided and exposed the downstream pier to possible damage. The subsidence was presumably due to the removal of lateral support when the river bed was eroded to a considerable depth. After much study, it was proposed that this portion of the east bank be stabilized by replacing, at least partially, the eroded bed material with a gravel blanket and rock revetment. At the same time, the gravel blanket and revetment was to be extended upstream to incorporate the piers on the east bank. Similar rock revetment and gravel blanket were proposed for the west bank to protect these piers and to prevent possible subsidence in this area. There was some concern that the placement of revetment in the neighborhood of the bridges would restrict the flow to such an extent as to enhance the possibility of flood damage upstream. The purpose of the experiments described in this section was to examine the effect of this revetment on the flow profiles through the bridge section and the extent of any erosion that might be the consequence of the placement of revetment in this area.

Stability of Bank Revetment

For the purpose of these tests, the model was remolded in accordance with the present prototype situation. The east bank upstream of the bridge was excavated to widen the upstream channel. The piers of the original upstream bridge were removed and replaced with cylindrical piers of considerably greater span, and piers 3 and 4 of the downstream bridge were underpinned with piles of greater length. The pile and rock-filled dike was installed along the east bank under and downstream of the bridge, and the new revetment was placed on the opposite banks as proposed. Chart 10 shows all of the modifications of the model and represents the geometry of the model as tested. The sand bed between the sloping bank armor was molded to Elev. 1058. Photo 16 shows the model with all of these modifications prior to the test, while

Photo 17 is another picture of the same conditions with the bridge decks in place. The material used in the model to simulate the revetment rock is shown as a size distribution curve in Chart 11. This curve gives the size of the rock to both the model and the prototype scale. The rock revetment was chosen somewhat arbitrarily based upon past experience with such models and was designed to be resistant to motion for the normal flood flow conditions.

In order to study the development of the erosion pattern and the water profiles resulting from the placement of the revetment, the experiments were started at a low discharge which was then successively increased until the maximum discharge of 55,000 cfs was reached. The tailgate was set so that the stage at the bridge would be 1090.8 ft when the discharge was 55,000 cfs. It was then held fixed at this value for all of the smaller discharges. The tailgate represented the fixed stage of the Missouri River.

The flow pattern in the model in the neighborhood of the bridges for these discharges is shown in Photos 18 through 23. In these tests the flow pattern was delineated by confetti on the water surface. The length of the streaks in relation to the exposure time represents the relative velocities in different parts of the model. The exposure time is reckoned from the angles swept by the white line on a rotating disk placed on the model in the field of the photograph. The disk rotates at 60 rpm, or one revolution each second. Of particular interest in this series of photographs is the developing flow pattern and the increasing velocity. In addition, particularly for the smaller discharges, the revetment under the water can be seen. The photographs also show the development of the wake downstream of the two central piers of the downstream bridge. The strength of this wake, of course, increases with increasing velocity that, in turn, increases with the discharge. Photo 23 shows the flow pattern for the flood discharge of 55,000 cfs when the stage at the bridge crossings was 1090.2 ft instead of the expected 1090.8 ft. This difference is due partly to the increase in velocity through the constricted bridge section, partly to the change in the upstream cross-section, and partly to possible errors in setting the tailgate.

The surface velocities as observed in the model by means of the floating confetti were evaluated on the basis of the exposure time and the streak length and are plotted in Charts 12 and 13. Chart 12 shows that the velocities generated in the river for discharges up to 30,000 cfs were relatively mild. The increasing velocity with increasing discharge is clearly shown. Chart 13 shows the patterns for the higher discharges up to 55,000 cfs. Here again as the discharge increases, the velocity increases, particularly that in the center of the channel in the neighborhood of the intermediate piers of the downstream bridge. Chart 14 shows the erosion pattern developed in the course of the experiment as the discharge increased from 10,000 to 55,000 cfs. The test for each discharge was continued long enough so that no further sediment was being transported by that discharge. Consequently, Chart 14 shows the scour pattern that would have developed for a discharge of 55,000 cfs. The contours show that the deepest erosion occurred downstream of the bridges and reached Elev. 1040 ft.

Photos 24 and 25 illustrate this erosion pattern somewhat more clearly. Photo 24 is a view from the downstream right side looking up towards the bridge piers, and Photo 25 is a view from above looking vertically downward on the erosion pattern. These two photographs show that the revetment was undisturbed and that the erosion occurred on the unprotected bed between the two protected banks. With this arrangement all of the piers were adequately protected by the revetment. These two photos are to be compared with Photos 16 and 17 to show the development of the erosion pattern on the unprotected bed between the piers.

In Chart 15 the final cross-section along the centerlines of the upstream and downstream bridges has been plotted along with the original bed as molded in the model. These show, as is also shown in Photos 24 and 25, that the sediment composing the bed between the two areas of bank revetment has been eroded to a point below the elevation of the bottom of the revetment. This suggests that with further erosion in this area, the rock revetment would begin to ravel. This should supply rock to the eroded area and thus tend to inhibit further erosion. The data shown in Charts 15 and 16 represent the conditions that might be expected as a consequence of a flood of 55,000 cfs through the revised cross-section at and upstream of the bridge crossing. In the course of reaching this condition, the discharge was successively increased from 10,000 to 55,000 cfs, with each discharge maintained until the bed had stabilized. Measured cross-sections for the larger discharges have been plotted in Chart 17, which shows the successive changes in the depth of erosion downstream of the bridges. The lower cross-sectional profile was taken just downstream of the downstream bridge so that it is similar to the upper profile shown in Chart 15. The upper two profiles show the developing scour pattern at two cross-sections somewhat further downstream from the bridge at a point where only the left side of the channel is protected.

Effect of Revetment Upon Upstream Water Surface Elevations

In addition to the erosion pattern and the stability of the revetment, the experiments were concerned with the effect of the revetment upon the water surface elevations upstream of the bridges in relation to possible flood hazard as a consequence of the proposed revetment. As previously stated, the tailgate had been set at the elevation determined in the verification tests so as to provide a water surface at Elev. 1090.8 ft at the bridge crossing. With this tailgate position and a discharge of 55,000 cfs, the water surface elevation at the bridge and at points upstream and downstream of the bridges was measured in order that the water surface elevations at these points with revetment could be compared with those measured in the verification tests without revetment. The water surface profiles for both conditions are plotted in Chart 16. It appears that the upstream water surface elevation is actually lower as a consequence of the upstream excavation, the removal of two piers, and the revetment construction than it was prior to these changes. Apparently, the reduction in resistance due to the upstream excavation more than offsets the increase in resistance due to the rough revetment surface.

Effect of Increased Slope on Erosion Pattern

The previous set of experiments showed what is likely to happen in regard to erosion in the neighborhood of the crossing when the revetment consists of rocks large enough to resist any movement. For these conditions, with a discharge of 55,000 cfs through the proposed section, the bed was eroded approximately to the toe of the revetment, or about 15 ft. At this point there may be incipient ravelling of the revetment. The revetment itself has not been disturbed and the bed under the bridge was still composed of sand. A hole which started to develop downstream of the bridges may be the result of the expanding high velocity jet from the constricted bridge crossing. The results of these experiments depended upon the maintenance of the water surface elevation at the tailgate at a level corresponding to that which occurred during the flood. This water surface elevation was presumably the consequence of the then existing water surface elevation in the Missouri River at its confluence with the Big Sioux River.

It was felt that an additional experiment would be desirable to observe the erosion pattern that would result for more extreme conditions, such as a still lower water surface elevation in the Missouri River than that which existed at the time of the flood. Inasmuch as the revetment was still intact after the previous experiments, additional tests, S2, S3, and S4, were performed by simply lowering the tailgate to represent lower Missouri River water surface elevations. The tailgate was adjusted so that the stage at the bridge crossing was successively 1088.7 ft (Test S2), 1087.4 (Test S3), and 1085.5 ft (Test S4). The respective flow patterns and erosion patterns were observed. The consequence of lowering the stage was a correspondingly increased velocity through the bridge cross-section. The successive cross-sectioned profiles are shown in Chart 18. Also plotted on this graph is the profile for the "standard" condition of 55,000 cfs and water surface at Elev. 1090.2 ft. It is of significance that in the cross-section some distance downstream of the bridge the revetment was not disturbed even though the greatest depth of scour was considerably below the elevation of the toe of the revetment. In the section just downstream and parallel to the bridges, where the revetment was placed on both sides of the channel, the increased velocities scoured the bed between the bank revetment to such an extent that the toe of the revetment was ravelled and deposited on the bottom of the channel. The corresponding longitudinal profiles are shown in Chart 19. A relatively great depth of scour occurred downstream of the bridge sections where there is no rock revetment. This was presumably due to the high velocity jet through the revetted section attacking the unprotected bed downstream.

Charts 20 and 21 show the velocities in the neighborhood of the bridge crossings as developed during these experiments. The location of the high velocities and the effect of the presently existing bridge piers of the downstream bridge on the flow pattern are apparent in Chart 20. The subsurface velocities as measured with a propeller meter at the bridge cross-section for these different stages are shown in Chart 21. It shows that as the stage is successively lowered, the maximum velocity is correspondingly increased.

All of these results are shown pictorially in Photos 26 through 31. Photo 26 is a view of the water surface flow pattern and shows the wake behind pier 3 of the downstream bridge and the vortices generated near the right bank on the outside of the curve. Photo 27 is the corresponding erosion pattern for a discharge of 55,000 cfs when the stage at the bridge has been reduced to 1088.7 ft. It shows the depth of scour in the bridge section and provides evidence of the ravelling of the revetment and its deposition on the intervening bed. It should also be noted that the revetment downstream of pier 3 of the downstream bridge began to erode as a consequence of the turbulence in the wake of this pier. This is evidenced by the elevation of 1054 ft in this region. Photos 28 and 29 show the erosion pattern as a result of a flow of 55,000 cfs with the stage further reduced to elevation 1087.4 ft. In order to determine more precisely if the rock revetment was disturbed, some of the particles composing the revetment were colored and laid in strips across the sections and on the revetted banks. These strips are shown in Photo 28. Photo 29 shows the same bed at the end of the test when the bed had again become stable. A comparison of Photos 28 and 29 shows that, for this condition, the revetment is relatively undisturbed except immediately downstream of pier No. 3 of the downstream bridge, where the colored material had been completely removed, and to a lesser extent on the toe of the revetment downstream of the bridges on the left bank, where it appears that some of the rock particles have been transported. It will also be noticed that the bed between the revetment has now been covered with rock particles and that apparently some of the revetment on the left bank downstream of the bridge has also been ravelled and deposited on the bed. Aside from this there appeared to be relatively little decrease in bed elevation due to this flow.

Photo 30 shows the surface flow pattern for a discharge of 55,000 cfs with the stage still further lowered to elevation 1085.5 ft. The high velocities through the constricted section and the development of vortices on both sides of the channel downstream of the constricted section is apparent. The maximum velocity through the bridge piers was of the order of 16 ft per second The erosion pattern (Photo 31) resulting from this flow has been contoured to show the character of the scour hole downstream of the bridge piers. The photograph shows that even for these extreme conditions the revetment has been relatively little disturbed and that further erosion of the bed between the piers appears to be inhibited by the deposition of revetment material on the bed.

Conclusions - Tests of Protective Measures

These experiments showed rather clearly that the presence of the revetment at the bridge section was not detrimental to the flow but that rather it protected the bed particularly around the piers from erosion by discharges of the same order of magnitude as those experienced during the flood of 1962. Furthermore, the study appeared to show that the piers would be safe and the banks stabilized even for conditions more extreme than those obtained during the flood if the revetment is of such a size that the rock particles themselves will not be removed by the flow. Chart 11 shows the size of the rock revetment as used in these experiments as plotted to the prototype scale. This rock was stable in the presence of the flow. If the prototype revetment rock is equal to or larger than that used in the model, it may be expected that it also will be stable during a similar flood in the prototype.

EFFECT OF GROINS ON FLOW PATTERN IN BRIDGE SECTION

This section will outline the results of tests made on various arrangements of permeable dikes in the bend upstream of the bridges to delineate their influence on the alignment of the flow in the river as it approaches the bridge sections. The purpose of the permeable dikes is twofold, 1) to rectify the flow alignment so that it would approach the bridge section more nearly at a right angle, and 2) to prevent further retrogression of that portion of the right bank lying along the outside of the curve.

The overall plan of this portion of the river showing the bridge crossing and the river alignment upstream and downstream of the bridge is given in Chart 22. This chart shows the location of the region for the proposed permeable dikes upstream of the bridges. It also shows the abrupt curve in river alignment downstream of the bridge cross-sections. The material used for the dikes was expanded metal sheet having a porosity of about 50 per cent. Two sheets were used for each dike. These were held together in such a manner as to decrease the porosity to approximately 20 per cent. The combined sheets, held upright by one or more support rods, were then forced into the bed to form the dike. In the test to be described, the discharge was 55,000 cfs and the tailgate was adjusted so that the stage at the bridge would be maintained at Elev. 1090.2 ft. The model had been run for a long enough period of time so that the erodible bed downstream of the bridges was essentially stable.

The data collected in those experiments was in the form of comparable photographs of the water surface. Confetti was used so that the flow pattern would be clearly delineated by the paths taken by the confetti particles. Two overlapping photographs taken from directly overhead were matched so that the immediate area of interest could be enlarged and covered photographically. The effectiveness of various permeable dike arrangements was judged on the basis of these photographs.

Dike Test D1

Test D1 was performed with the model in the original condition, that is, without any dikes, so as to provide a basis for comparison to determine the effectiveness of various dike configurations. The flow pattern for this condition is shown in Photo 32. In this composite photograph it will be noted that the separation at the outside of the curve on the right bank is relatively minor but that the separation zone at the inside of the bend in the neighborhood of the channel excavation on the left bank is very pronounced and extends over the entire region of the excavated area. The angle of attack of the flow on the bridge piers of the downstream bridge is about 23 degrees and the highest velocities in this region are about 12 fps. At Station 14+00 upstream of the bridge, shown at the upstream extremity of the photograph, the water elevation was 1091.5 ft. These data are shown in Chart 23. From the photo it appears that if the left bank excavation were to be extended upstream in order to decrease the curvature of the flow in this area, part of the separation zone along the left bank might be eliminated with a consequent lowering of the velocities and a decrease in the angle of attack against the bridge piers. The flow pattern shown in Photo 32 will serve as the basis for comparison of subsequent tests.

Dike Test D2

For the first experiment with permeable dikes, four dikes slanting downstream at about 75 degrees to the direction of flow and of a maximum length of about 145 ft were installed in the model. The spacing of the dikes was about equal to their length so that approximately square cells were formed. This arrangement is shown in Chart 23. The flow pattern generated by this dike geometry is shown in Photo 33. The effectiveness of the dikes in redirecting the flow is clearly demonstrated by comparison with Photo 32. The velocities within the dike cells, as indicated by the short confetti streaks, were relatively small. The left bank separation zone was virtually eliminated. The upstream stage rose to Elev. 1093.3 ft (an increase of 1.8 ft over the original conditions) and the maximum stream velocity increased to over 16 fps. This increase in stage and stream velocity was due to the constrictive effects of the long dikes and the fact that the excavation of the left bank does not extend far enough upstream to be compatible with this dike arrangement. The angle of attack on the piers was decreased to 10 degrees as compared to 23 degrees for the original conditions. Some bed erosion occurred at the end of two of the dikes because of the high velocities in this region. Such a phenomenon could also be expected in the prototype. This dike design succeeded in straightening the flow to a very considerable degree but probably at the price of a considerably increased velocity in the channel.

Dike Test D3

This design employed more dikes of a shorter length, as shown in Chart 24. Six dikes were used and the greatest length of dike was approximately 90 ft. This arrangement also formed cells along the left bank which were approximately square, but the channel as a whole was considerably less constricted. The flow pattern for this test is shown in Photo 34. Immediately apparent was a somewhat increased zone of separation along the left bank and a somewhat greater angle of attack on the piers. The velocity between the dikes was again very considerably reduced and probably had no erosive action on the bank. The angle of attack on the piers was about 18 degrees. The upstream stage was lowered to Elev. 1091.8 ft and the maximum stream velocity was about 11 fps. With this velocity in the model, no erosion of the bed material at the end of the dikes occurred. It appeared that this design was intermediate in effect between the original conditions and thoso of Test D2 and, because of the lowered stream velocity, appeared to be an effective solution.

Dike Test D4

This test was the same as Test D3 except that an approach guide wall 270 ft long has been added upstream of the dikes along the right bank. This geometry is also shown in Chart 24. The guide wall caused little or no change in the overall flow pattern but it did provide a smoother transition between the natural bank and the first permeable dike. This is shown in Photo 35. The guide wall in combination with the shorter dikes of Test D4 provided a very smooth and regular overall flow pattern. Although the velocity in the channel remained about the same as in the previous tests, the angle of attack on the pier was very slightly reduced to about 16 degrees. The photograph shows where the flow pattern has been improved.

Dike Test D5

For this test the dike system consisted of 10 dikes, each about 25 ft long. They were spaced about 55 ft apart. The dike arrangement is shown in Chart 25 superimposed upon the bed geometry. As shown in Photo 36, these dikes had practically no effect on the overall flow pattern, which is very similar to that of the initial undiked channel. Essentially they served only to protect the South Dakota or right bank from erosive attack. Such protection could probably be better obtained by the use of riprap on the present bank.

Dike Test D6

For Test D6, the dike arrangement consisted only of a training dike at the upstream end of the curve on the right bank (Chart 26). The effect of removing the dikes jutting into the flow was a slight spreading of the flow toward the banks. This is shown in Photo 37. The training dike effectively established the separation along the right bank and so reduced the erosive attack along the bank. The separation along the left bank was very similar to that observed in Test D4 with the dikes in place. The angle of attack on the bridge piers was increased somewhat to 20 degrees and the velocities were again between 11 and 12 fps. It appears that the abutment area of the upstream bridge on the right or South Dakota side is subjected to more attack in this arrangement but this could be rectified by a relatively short finger dike placed upstream of the abutment area.

Dike Test D7

For this experiment the approach training dike was increased in length by about 100 ft, adding an additional dike at a slight streamward angle to the downstream end. This geometry is shown in Chart 26. The flow pattern generated by this training dike arrangement is shown in Photo 38. It is apparent that this change in training dike length had a negligible effect in changing the flow pattern over that obtained in Test D6. The flow pattern and other hydraulic conditions are almost identical with those previously obtained.

Conclusions - Permeable Dikes

The physical characteristics of the various dike patterns and the influence of these patterns on the stage upstream of the dike field, the maximum velocity in the channel, and the angle of attack on the bridge piers are tabulated below:

Test No.	No. of Dikes	Greatest Length* Ft.	Total Length Ft.	Upstream Stage Ft.	Maximum Velocity Fps.	Angle of Attack on Bridge Pier Degrees
D1	0	--	--	1091.5	12	23
D2	4	145	485	1093.3	16	10
D3	6	80	460	1091.8	11	18
D4	7	80	680	1091.8	11	16
D5	10	25	250	1091.5	12	20
D6	1	270	270	1091.6	12	20
D7	1	370	370	1091.6	12	19

*Lengths measured from model shoreline. For installation end points should be located and dike extended to shoreline.

The tabulation shows in general that the greater the constriction from the right bank the better is the angle of attack on the piers of the downstream bridge. This improvement takes place at the expense of the water surface elevation upstream of the dike field and the maximum velocity in the channel adjacent to the dike field. It appears that the dike arrangement in Test D2 provides the best alignment for the piers but the main velocity is increased to 16 fps and the upstream stage is about 1.8 ft above that obtained without dikes. With other arrangements providing less constriction to the flow, the rectification of the flow pattern is less extreme in the sense that the angle of attack has not been reduced as much as before, but at the same time the water surface elevation has not been appreciably increased nor has the mean velocity been increased. As a result, the dike geometry shown in Test D3 or Test D4 would improve the flow pattern somewhat and, perhaps more important, protect the right bank from further retrogression in time of floods.

LIST OF PHOTOS

PHOTO 1 Aerial view of the Big Sioux River and Interstate-29 near Sioux City, Iowa showing the bridge crossing where the highway crosses the river into South Dakota. The aerial view shows the general alignment of both the river and the highway in this region.

PHOTO 2 (Serial No. 142-10T) Collapse of the upstream bridge of I-29 crossing of the Big Sioux River. Looking towards Iowa from the abutment of the submerged bridge before the collapse of pier 4. Photo taken by South Dakota Department of Highways.

PHOTO 3 Following the collapse of the upstream bridge the left bank downstream of the downstream bridge began to subside and move toward the river.

PHOTO 4 (Serial No. 142-58) During verification test V-1 and for a discharge of 55,000 cfs, pier 3 of the downstream bridge collapsed in the model. In this test there was no stabilization of the left bank and the riprap stockpile on the right bank upstream of the bridges was ommitted. The flow pattern consequently did not simulate that observed in the prototype. The direction of flow is from right to left.

PHOTO 5 (Serial No. 142-60) Erosion pattern following verification test V-1. This view was taken at the end of the experiment after the water had been drained from the model and shows the collapse of the pier 3 of the downstream bridge. It will be noted that pier 3 of the upstream bridge has also been seriously undermined.

PHOTO 6 (Serial No. 142-61) Big Sioux Verification Test V-2. This test was made after the banks had been adjusted and the stockpile replaced on the right bank. Pier 3 of the upstream bridge has been undermined and fallen in a manner somewhat similar to that observed in the prototype.

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PHOTO 8 (Serial No. 142-65) Big Sioux Verification Test V-2. After pier 3 had fallen into the river the superstructure was also placed in the flow similarly to that observed in the prototype. This photo shows the flow pattern through the bridge section and the collapsed upstream bridge.

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PHOTO 11 (Serial No. 142-72) Big Sioux Verification Test V-4. A view of the flow pattern after the collapse of pier 3 of the upstream bridge.

PHOTO 12 (Serial No. 142-73) Big Sioux Verification Test V-4. The erosion pattern following the collapse of pier 3 of the upstream bridge.

PHOTO 13 (Serial No. 142-75) Big Sioux Verification Test V-4. This is a view of the flow pattern after the superstructure had been placed in the flow to simulate that of the prototype.

PHOTO 14 (Serial No. 142-76) Big Sioux Verification Test V-4. A view of the erosion pattern at the end of the test after the water had been drained from the model to show the contours and the fallen piers and bridge spans.

PHOTO 15 (Serial No. 142-77) Big Sioux Verification Test V-4. A vertical view of the erosion pattern at the completion of the test with bridge spans removed so that the scour pattern can be seen.

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PHOTO 18 (Serial No. 142-82) For a discharge of 10,000 cfs through the constricted bridge section the velocities are relatively low and the flow is relatively smooth. The bank revetment near the bridge piers can be seen through the water.

PHOTO 19 (Serial No. 142-83) The discharge shown here is 20,000 cfs and the stage at the bridge section is 1083.2 ft. The velocities through the constriction have increased somewhat and the wakes downstream of the piers of the downstream bridge are becoming more apparent.

PHOTO 20 (Serial No. 142-84) When the discharge is increased to 30,000 cfs and the stage is increased to 1085 ft, the velocities through the bridge section are correspondingly increased but the flow pattern in general is similar to that observed for the lower discharges.

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PHOTO 24 (Serial No. 142-89) This photo shows the erosion pattern in the neighborhood of the bridge piers developed by a discharge of 55,000 cfs. For these conditions, the revetment is relatively undisturbed and the greatest erosion occurred in the bed downstream of the bridge section.

PHOTO 25 (Serial No. 142-90) This photo is a view from above showing the same erosion pattern for the 55,000 cfs discharge. This view shows more clearly the outline of the erosion contours.

PHOTO 26 (Serial No. 142-91) In this experiment for a discharge of 55,000 cfs the stage was lowered to Elev. 1088.7 ft at the bridges to simulate a low water condition in the Missouri River. The lower water surface elevation results in a higher velocity through the constricted section at the bridge piers and a more pronounced jet into the wider channel downstream of the bridges. The wake downstream of the piers in the flow proper and the vortices generated along either bank are clearly shown.

PHOTO 27 (Serial No. 142-92) A vertical view of the erosion pattern developed by the above flow shows that the revetment is just beginning to ravel from the toe of the bank revetment. The increased depth of scour downstream of the bridge section is becoming more apparent.

PHOTO 28 (Serial No. 142-93) In order to show more clearly whether the rock revetment is being scoured, strips of colored rock particles have been placed on the bed on the bridge centerlines and along the banks. The photo was made before the experiment was performed.

PHOTO 29 (Serial No. 142-95) This photo shows the same bed at the end of the test with a discharge of 55,000 cfs and the stage reduced to Elev. 1087.4 The revetment is relatively undisturbed except immediately downstream of pier 3 of the downstream bridge. Here the revetment has been eroded to a considerable extent by the flow in the wake of the pier. On the left bank downstream of the bridge section, it appeared that some of the rock revetment may have been moved and deposited further downstream on the bed.

PHOTO 30 (Serial No. 142-96) This view shows the water surface pattern for a discharge of 55,000 cfs with the stage at the bridges reduced to 1085.5 ft. The high velocities through the constricted section give rise to a strong jet which penetrates the flow downstream of the bridge and generates strong vortices on both sides of the channel. The disturbance caused by the pier in the flow is also clearly apparent.

PHOTO 31 (Serial No. 142-98) The extremely high velocities through the constricted section and the jet created thereby which attacks the bed downstream of the bridge have caused very considerable erosion in this region. The photo also shows that the area of the bed between the piers has now been completely covered with rock particles ravelled from the bank revetment on both sides of the channel. Some of the rock has also been carried into the scour hole downstream.

PHOTO 32 (Serial No. 142-118) This photo shows the original conditions of the stream channel and the flow pattern upstream of the bridge cross-sections before any protective devices were installed along the right bank. The significant aspect of this flow is the angle of attack on the bridge piers of the downstream bridge and the extent of the zone of separation along the left bank. It appears that in time of flood the right bank may be attacked by the high velocities in this neighborhood.

PHOTO 33 (Serial No. 142-119) For this test four relatively long dikes have been installed along the right bank. The effect of the dikes is to push the flow over towards the left bank and to create relatively large areas of quiet water between the dikes on the right bank. The angle of attack on the bridge piers has been greatly reduced.

PHOTO 34 (Serial No. 142-120) By shortening the dikes and decreasing the spacing, the flow pattern is somewhat improved in that the mean velocity has been reduced. The right shore is still protected from erosion and the angle of attack has been decreased below that for the origina conditions.

PHOTO 35 (Serial No. 142-121) This test is similar to the previous test except that a training dike has been installed at the upstream end of the dike field in order to provide a better transition from the channel to the first dike. The flow pattern is very similar to that observed in Photo 34.

PHOTO 36 (Serial No. 142-122) For this test the length of the dikes has been drastically reduced and the number of dikes has been increased. This system has relatively little effect on the flow pattern but it does provide scour protection along the right bank.

PHOTO 37 (Serial No. 142-123) An attempt was made to direct the flow and protect the right bank by providing a training dike at the upstream end of the curve in place of the system of dikes jutting into the flow. This system is somewhat effective in that it creates a zone of separation along the right bank in which the velocities are greatly reduced. In addition to the zone of separation, there is a slightly reduced angle of attack on the bridge piers.

PHOTO 38 (Serial No. 142-124) In this experiment the training dike has been increased in length by 100 ft. The flow pattern, however, is very similar to that shown in Photo 37 and the added length of dike has relatively little effect in modifying the flow pattern.

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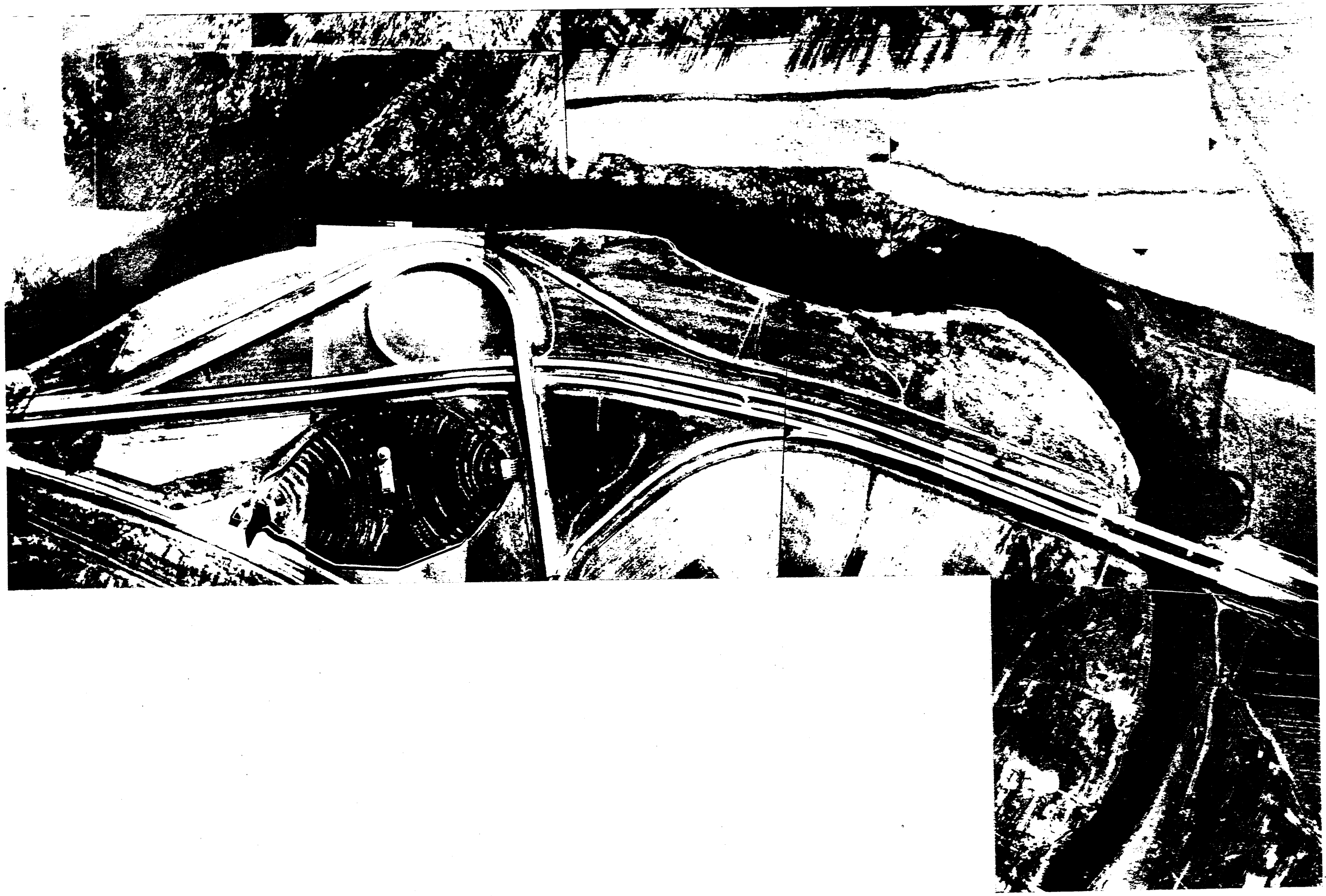

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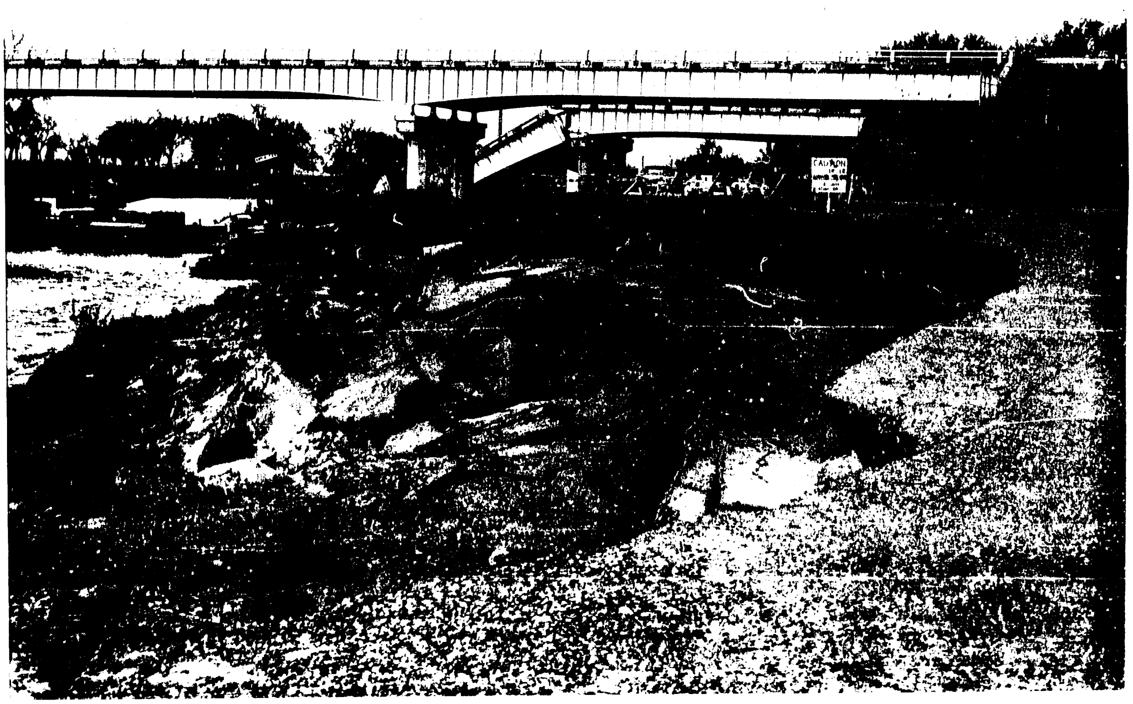

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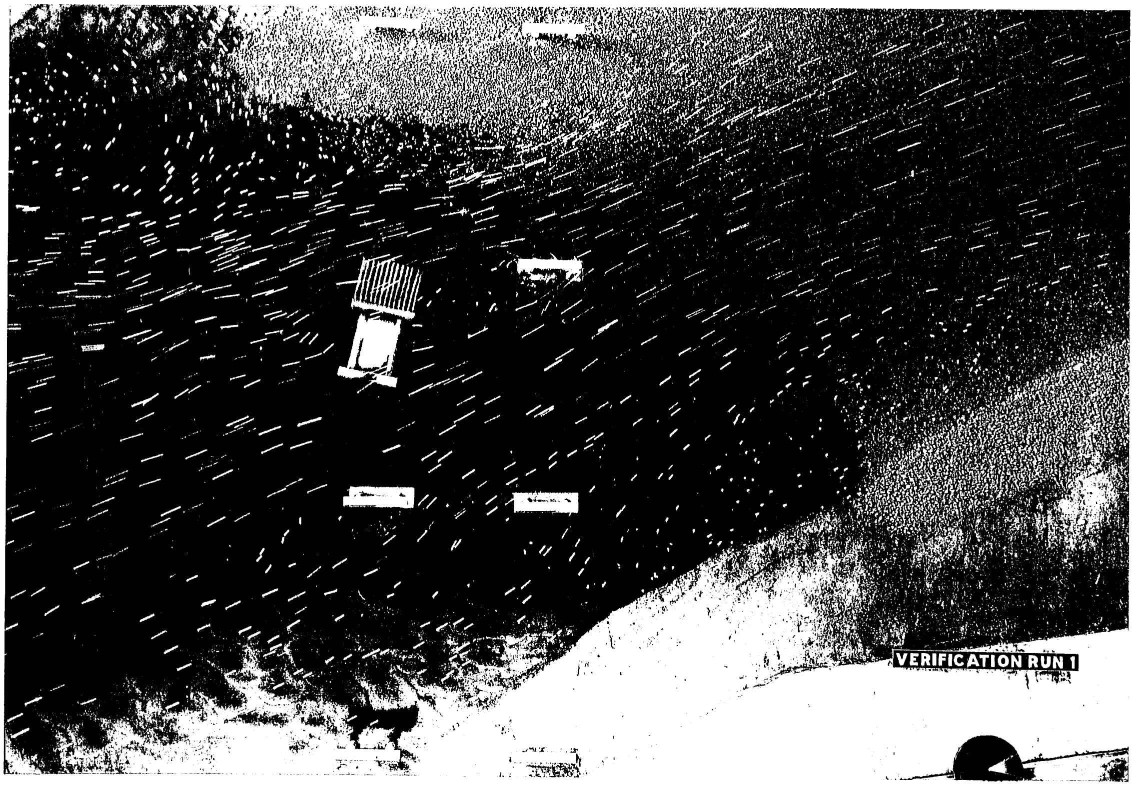
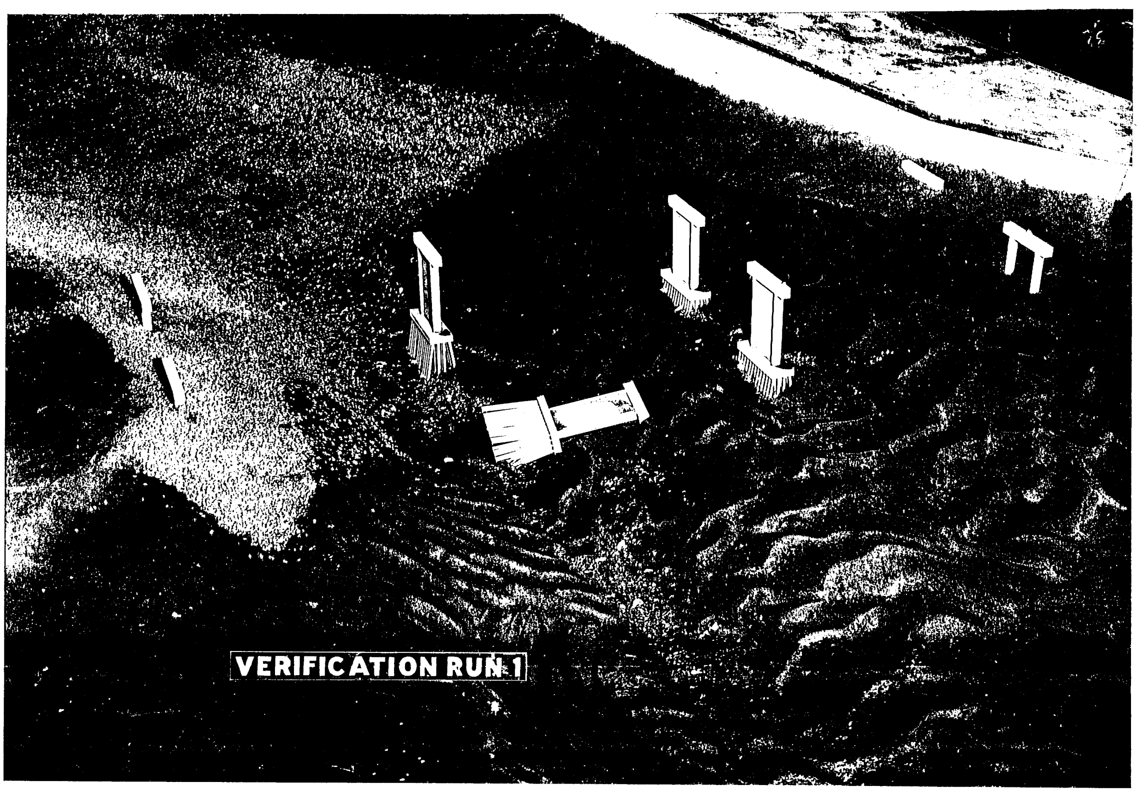


Photo 4



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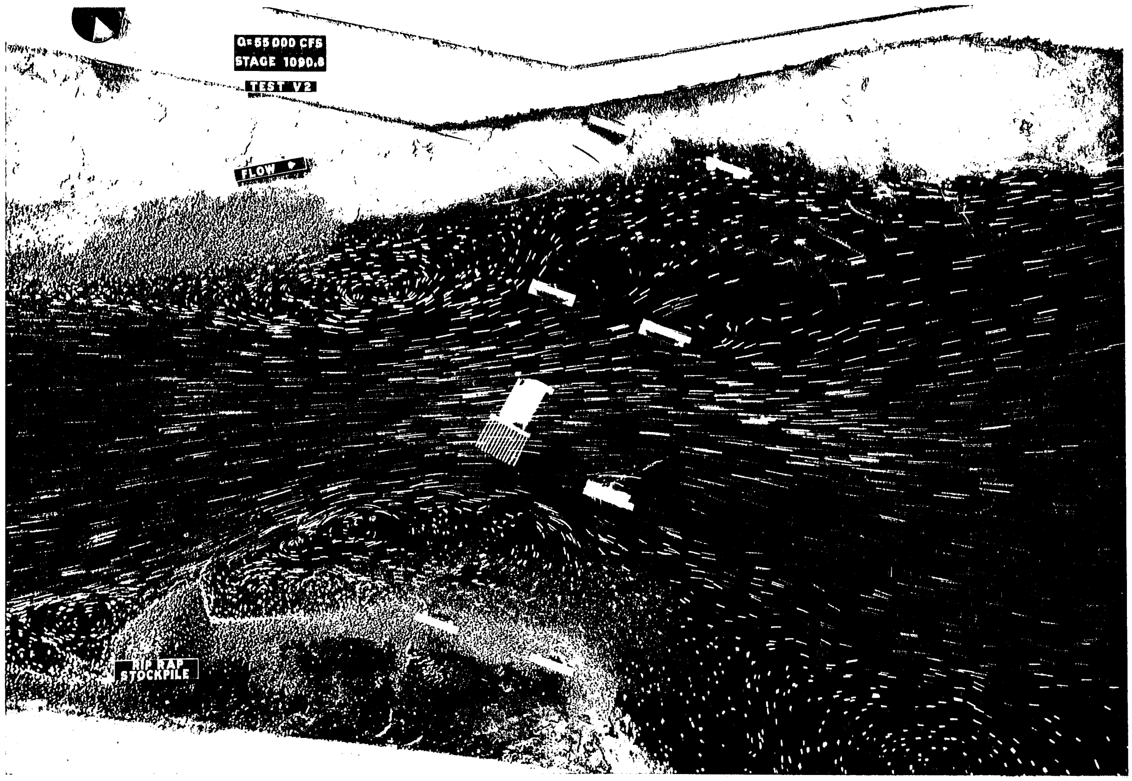
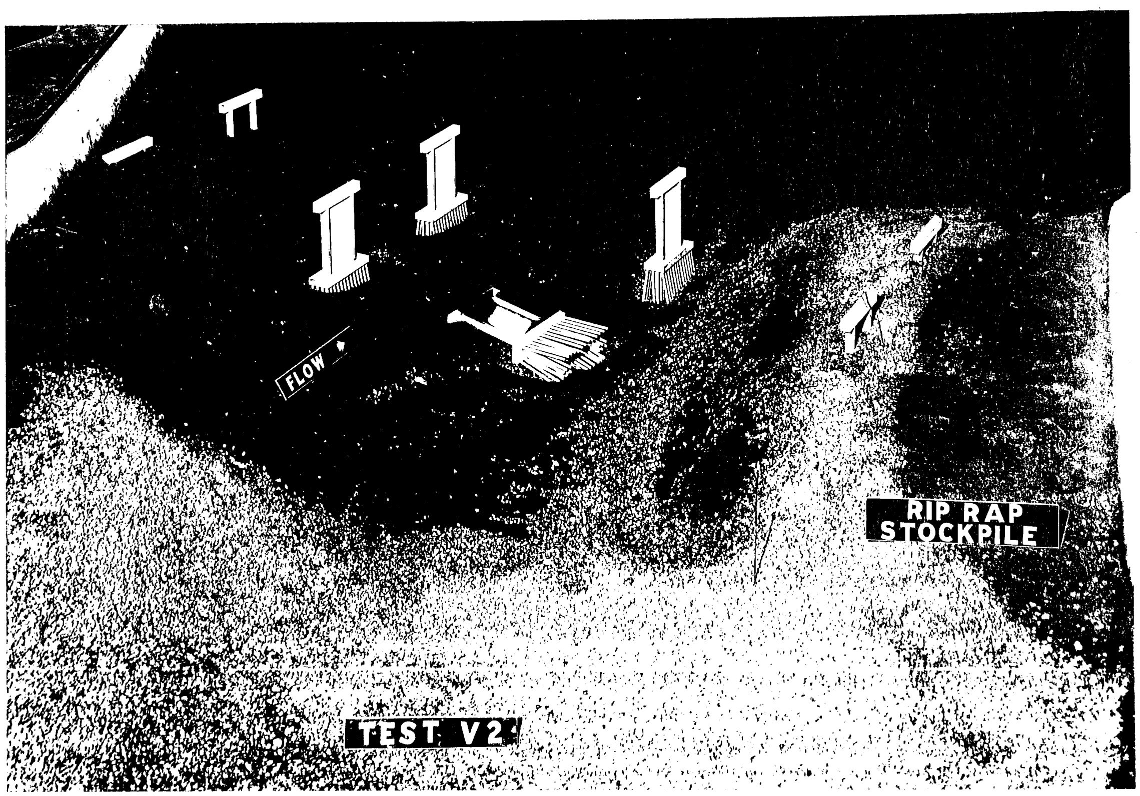


Photo 6



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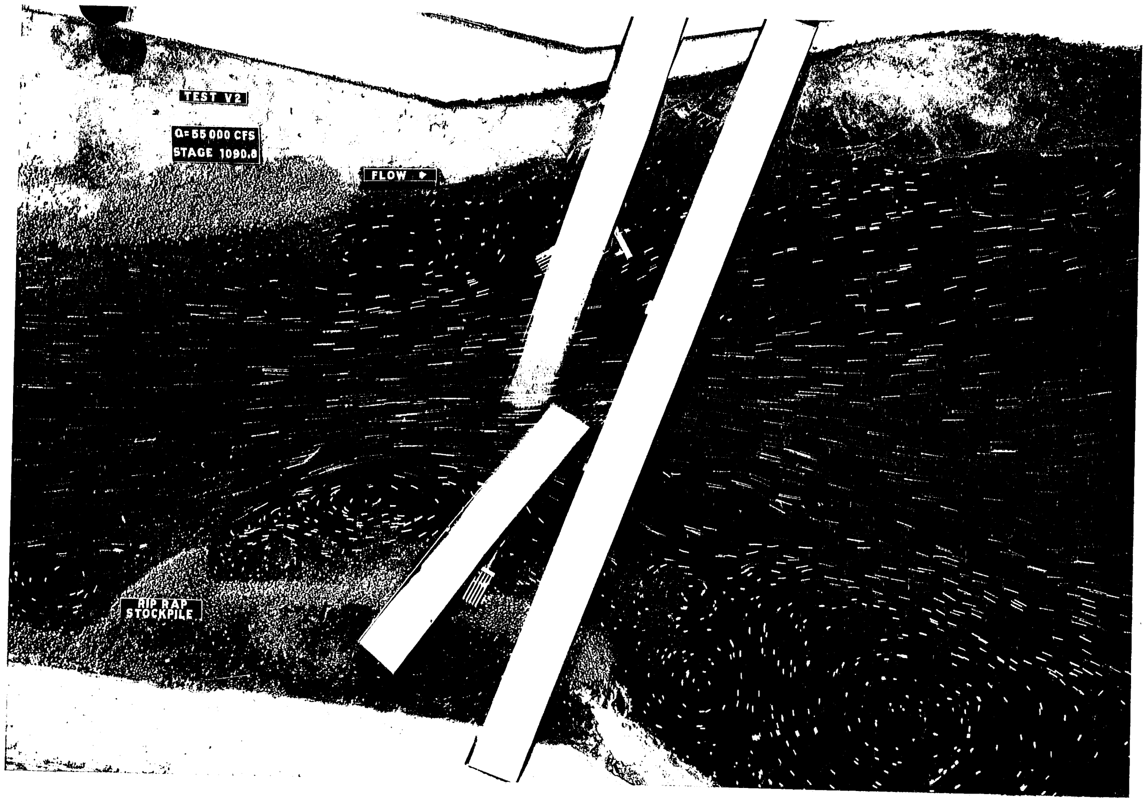

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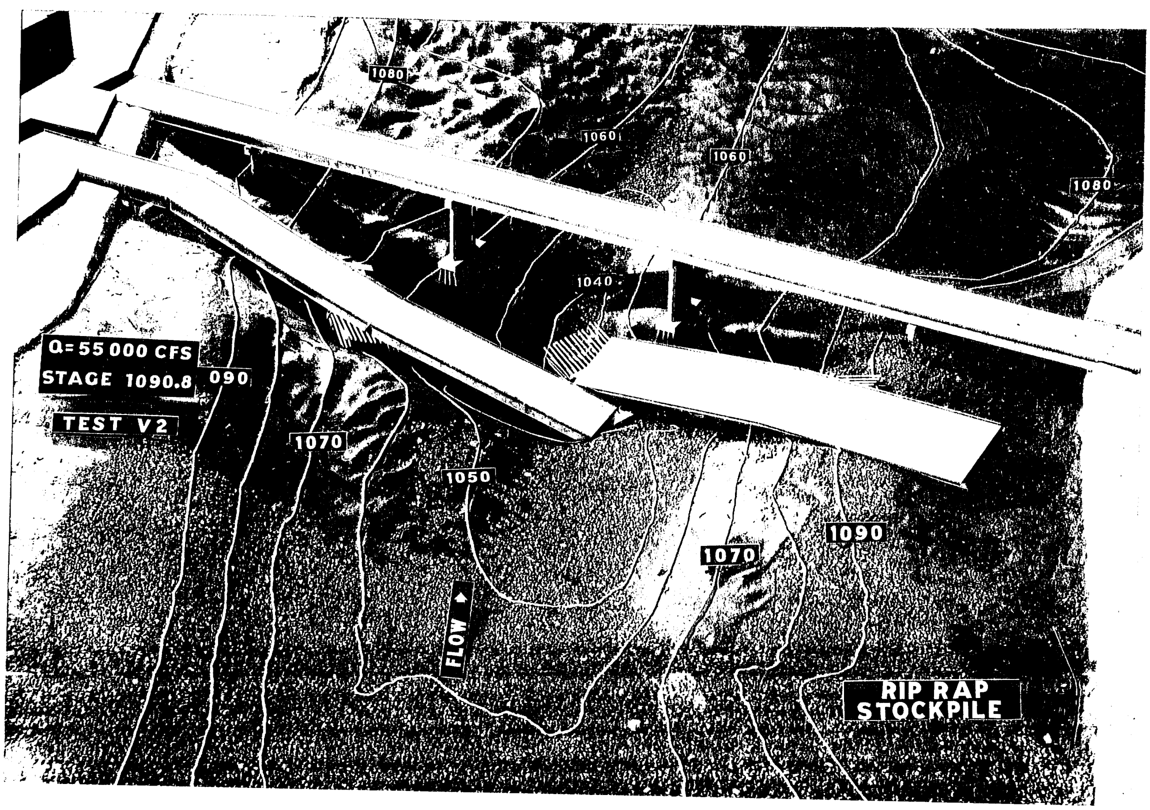

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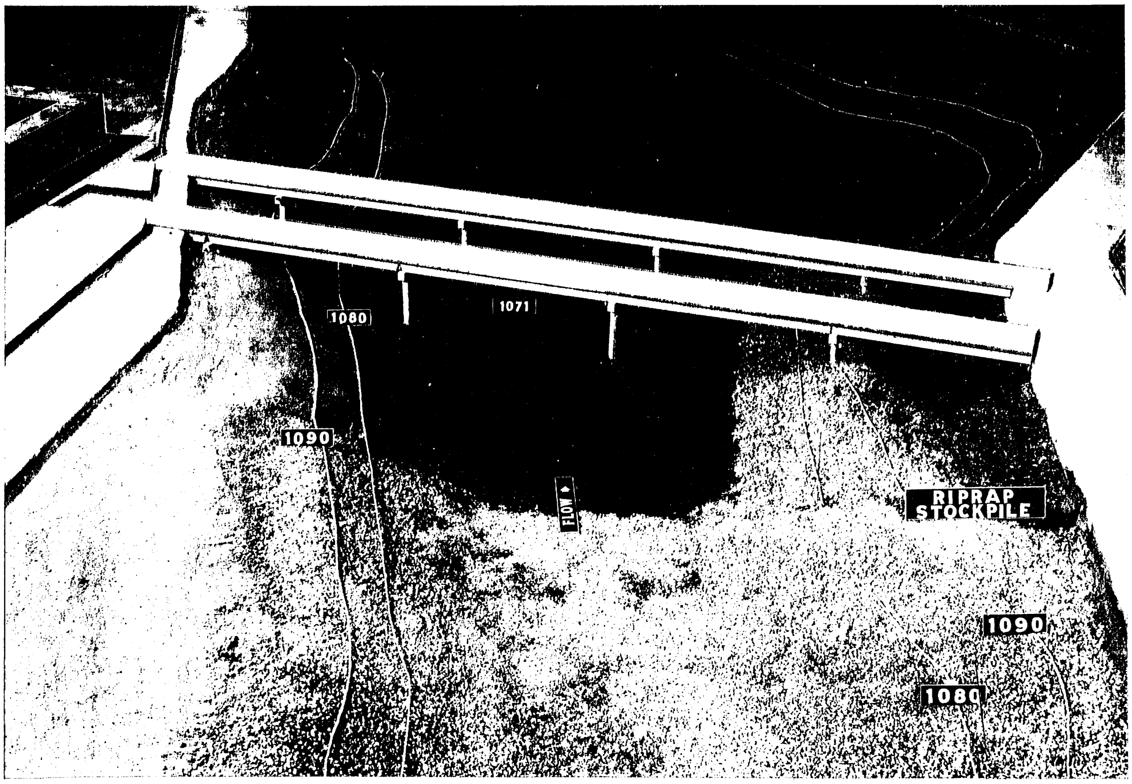
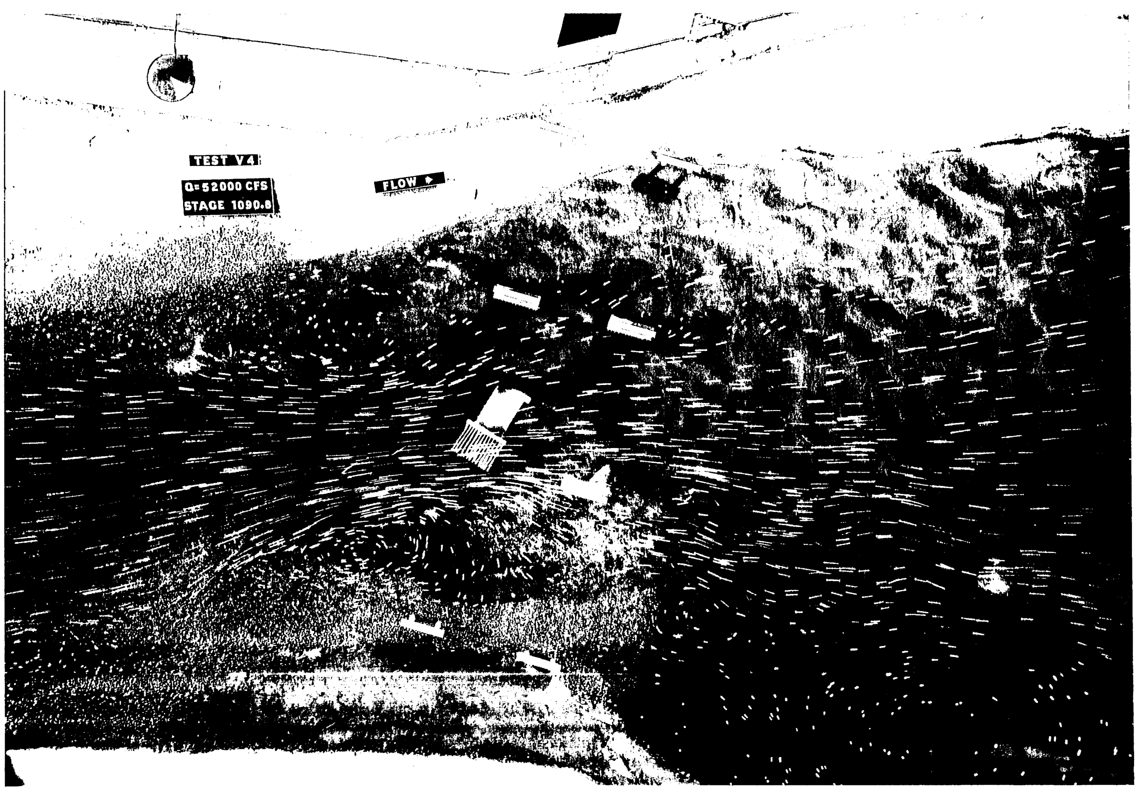


Photo 10



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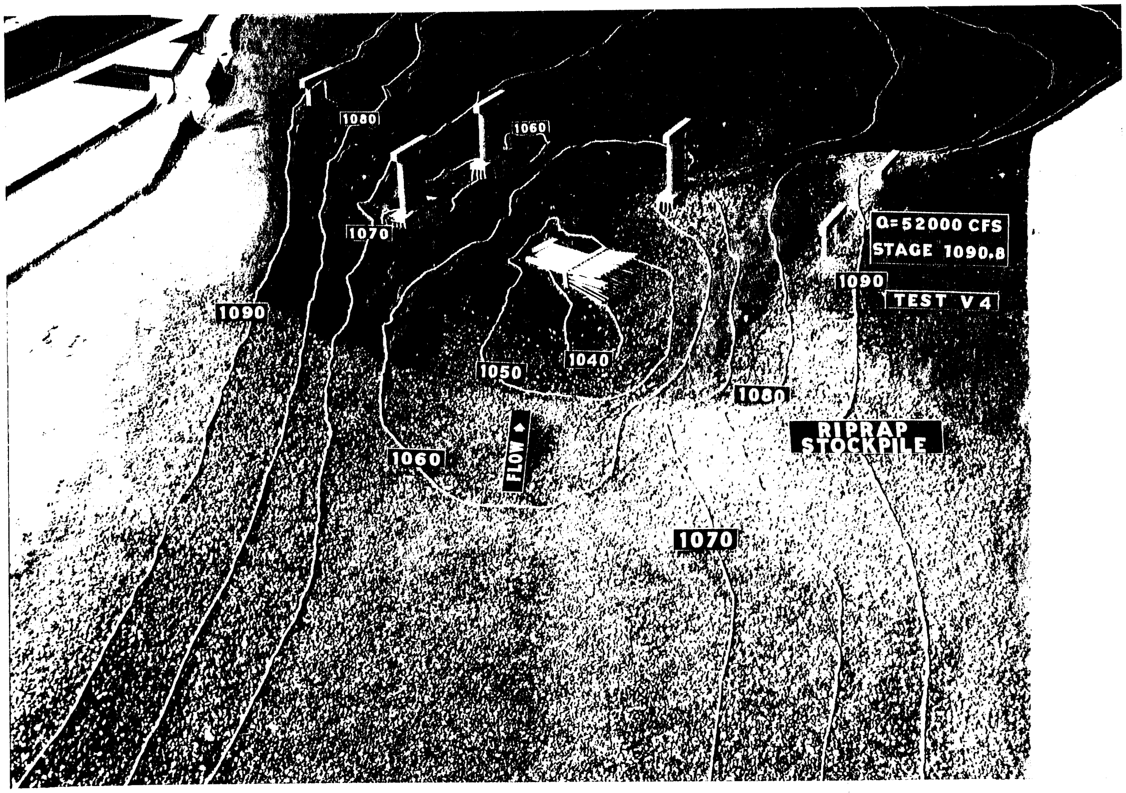
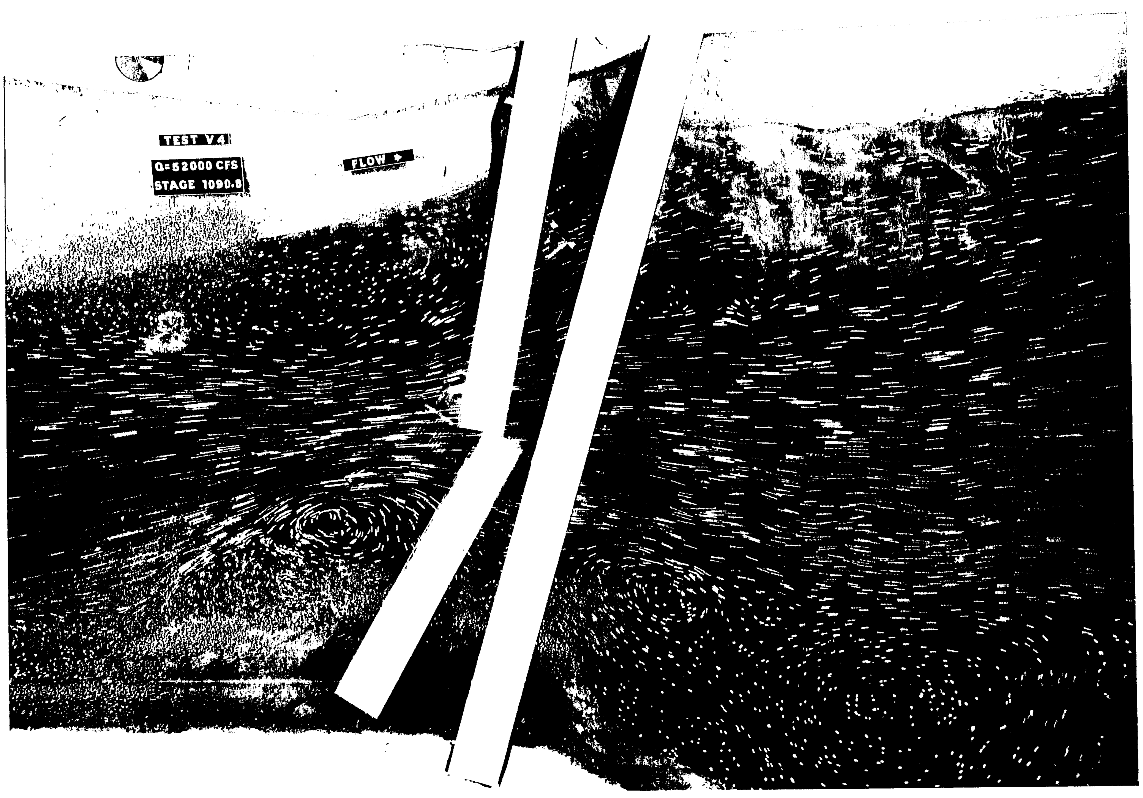


Photo 12



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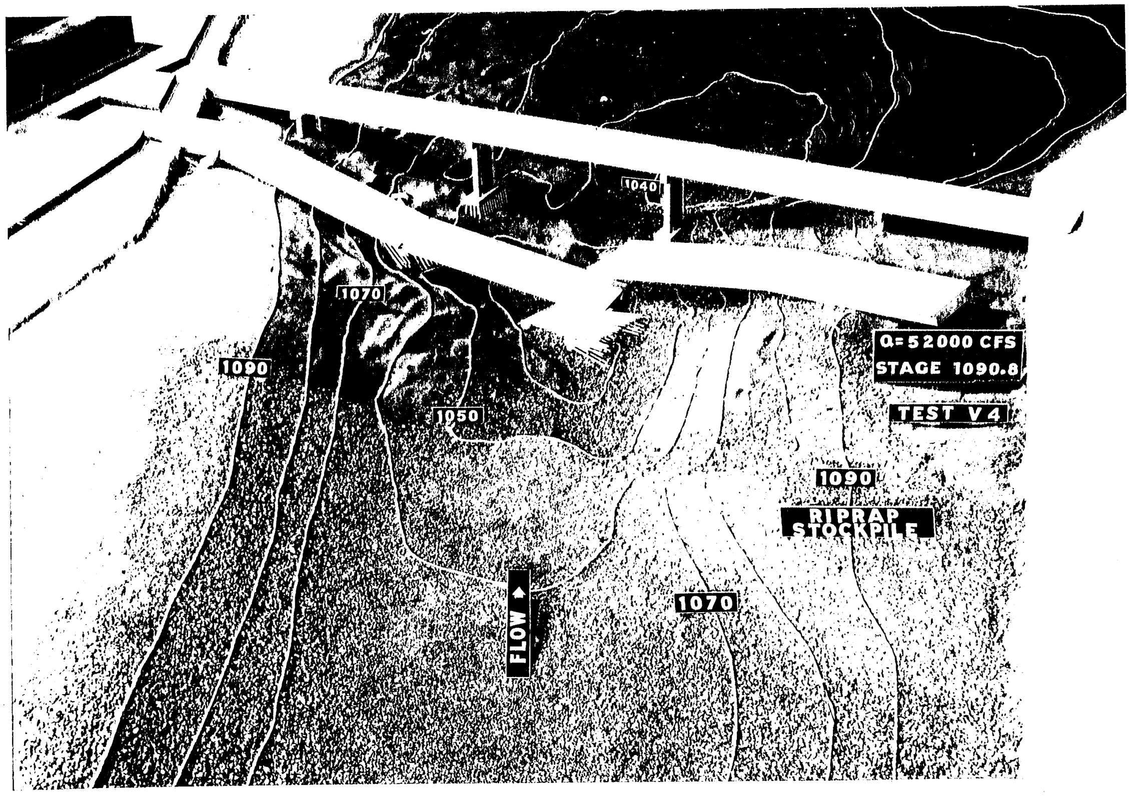

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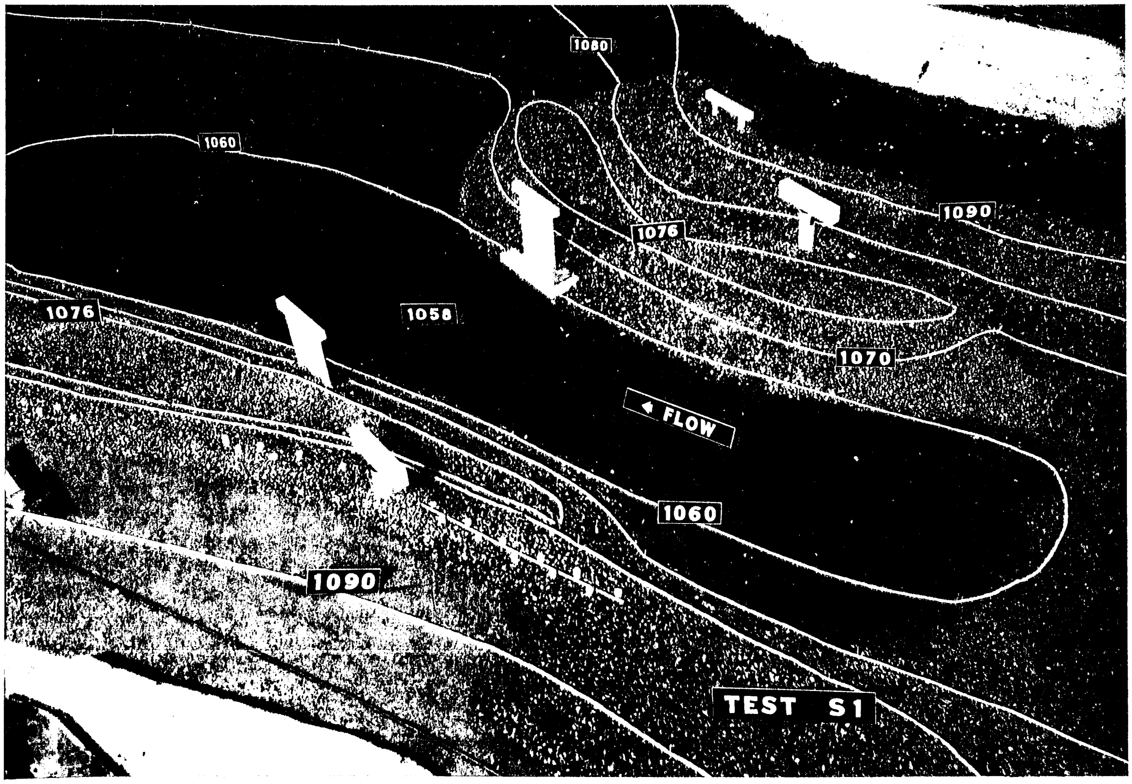

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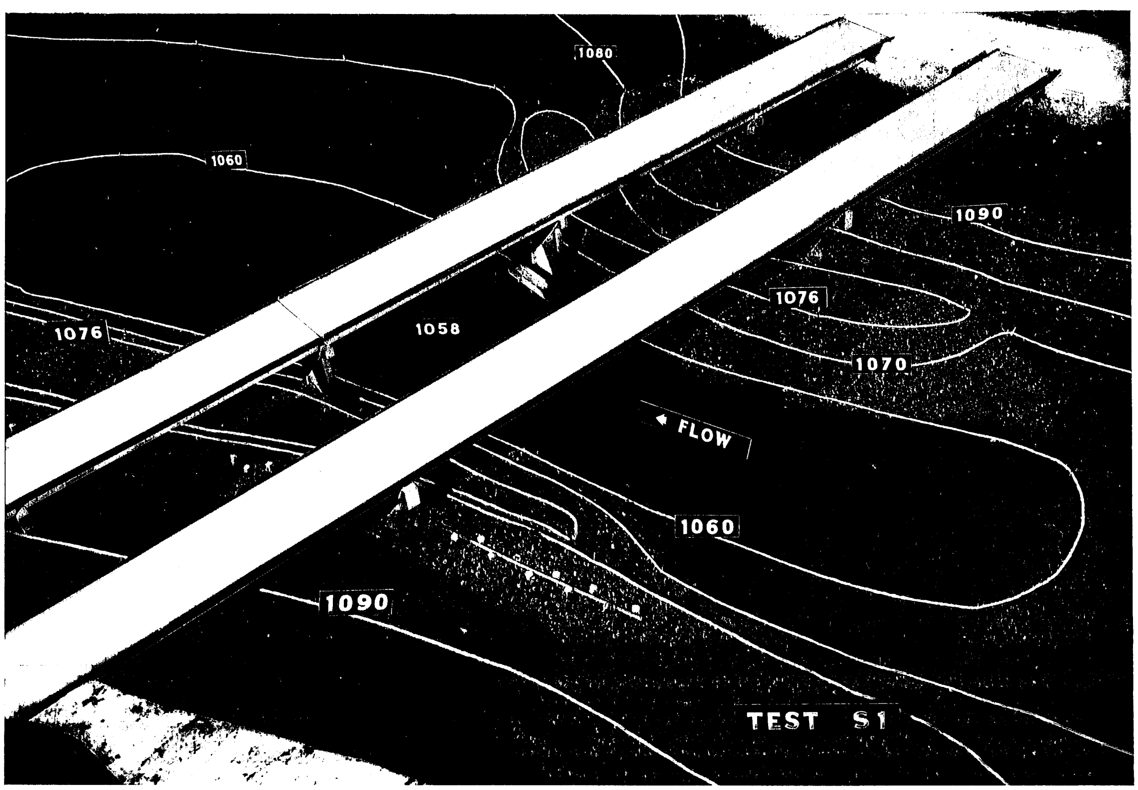

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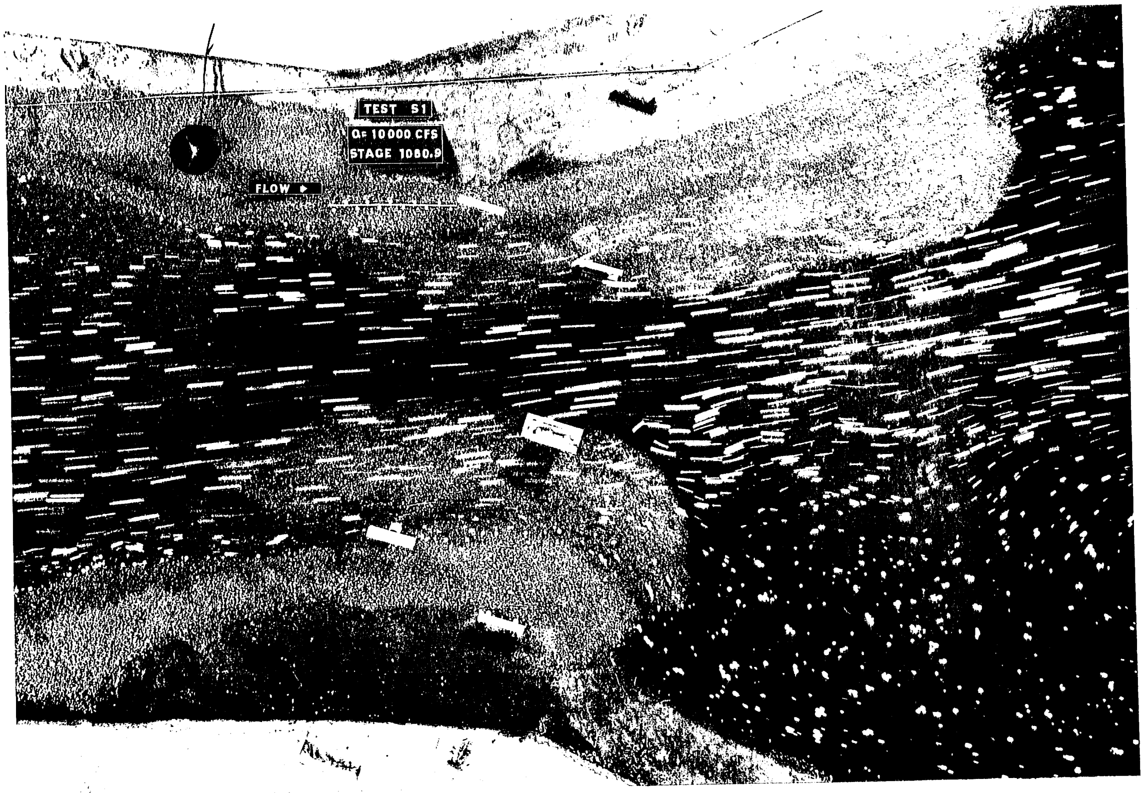

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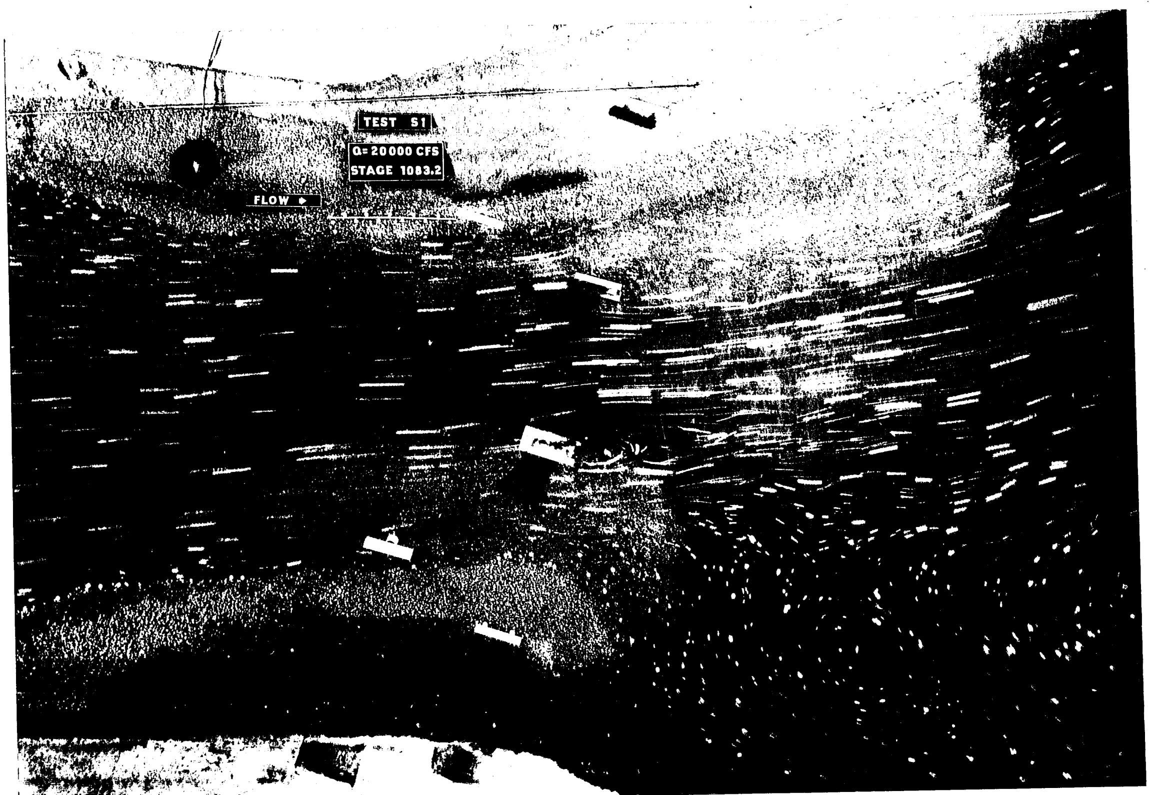

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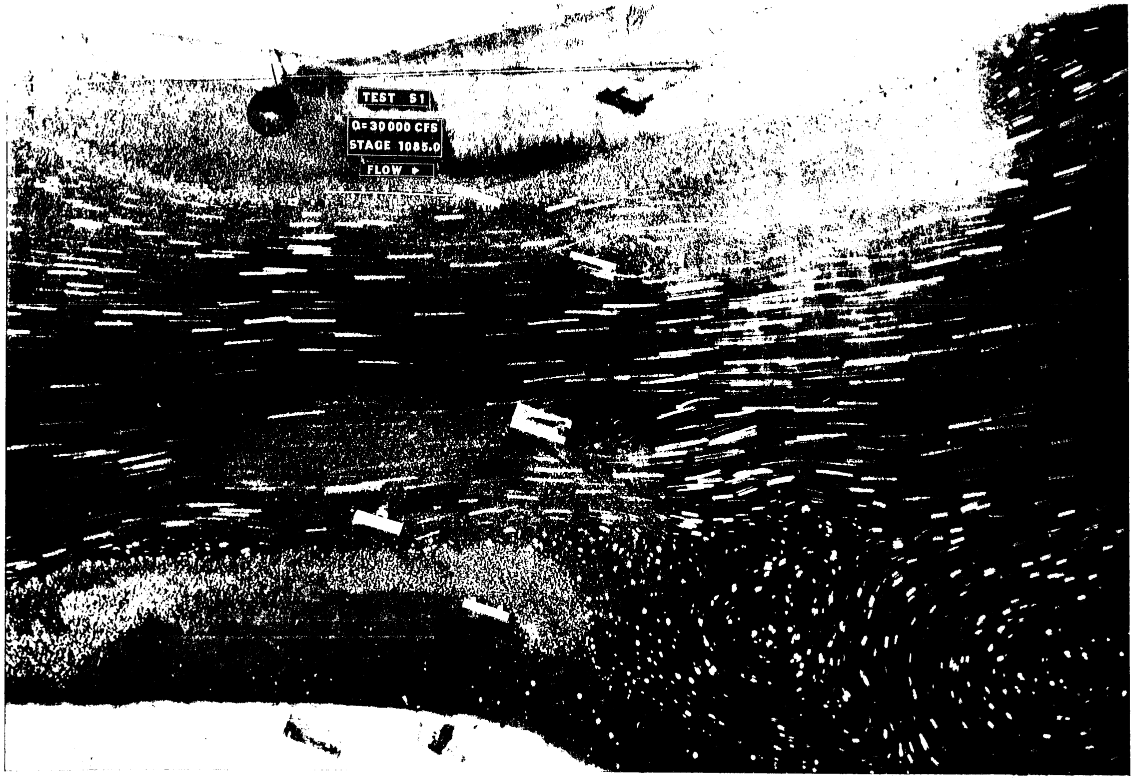

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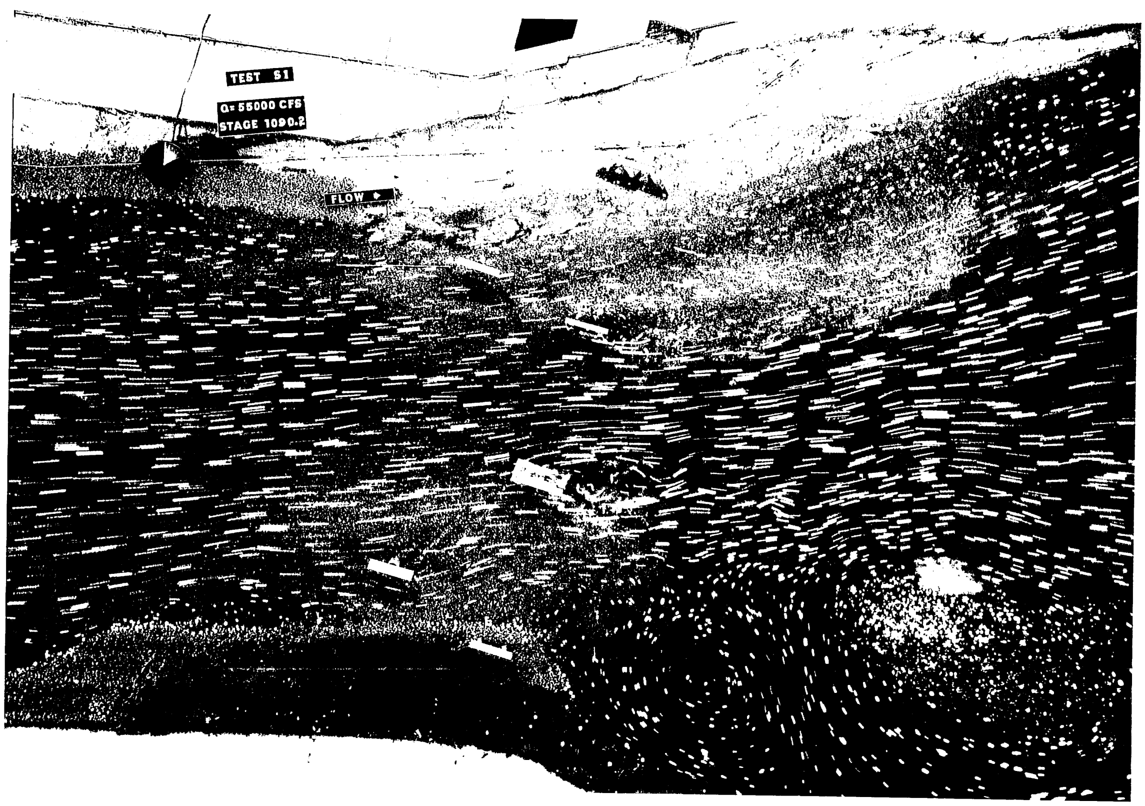

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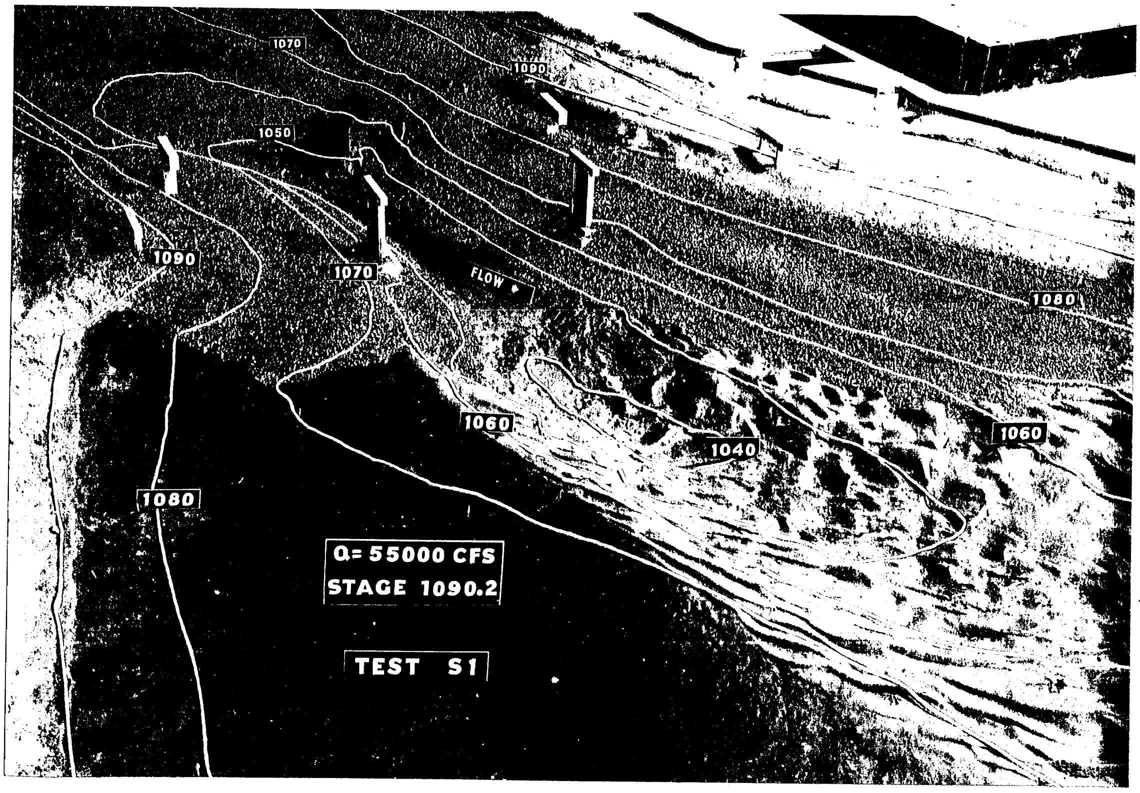


Photo 24

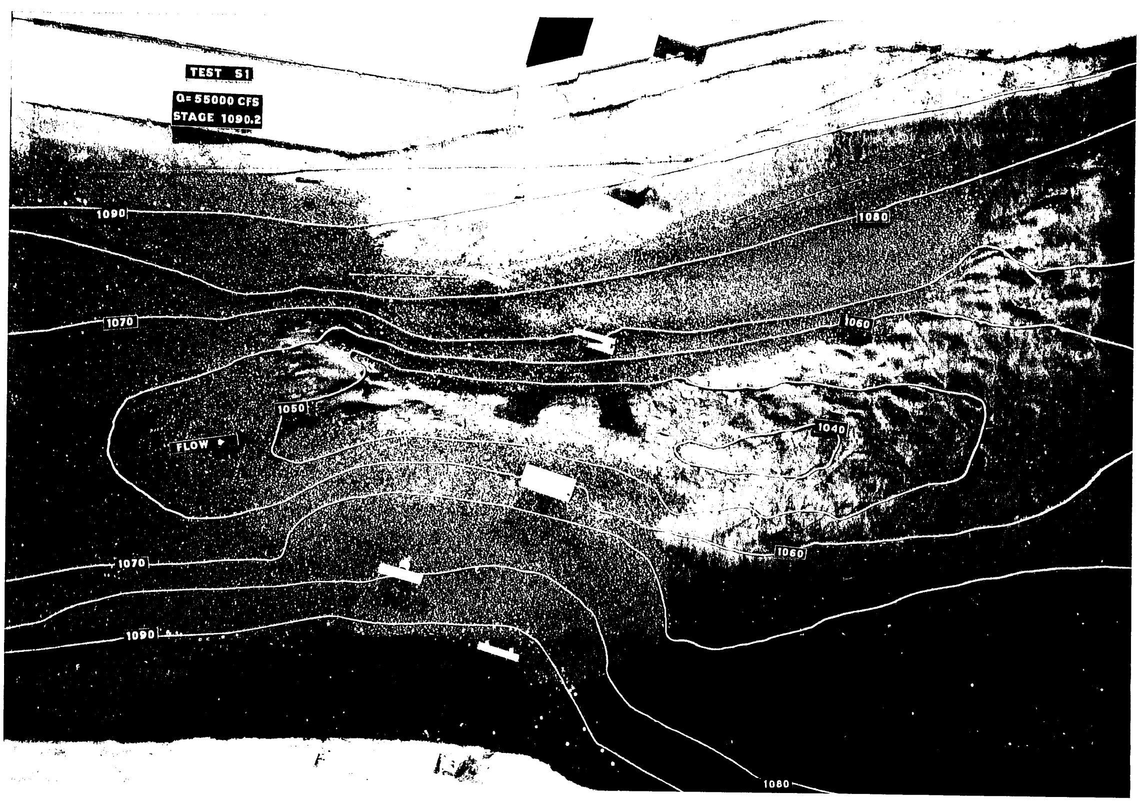


Photo 25

PHOTO 26 (Serial No. 142-91) In this experiment for a discharge of 55,000 cfs the stage was lowered to Elev. 1088.7 ft at the bridges to simulate a low water condition in the Missouri River. The lower water surface elevation results in a higher velocity through the constricted section at the bridge piers and a more pronounced jet into the wider channel downstream of the bridges. The wake downstream of the piers in the flow proper and the vortices generated along either bank are clearly shown.

PHOTO 27 (Serial No. 142-92) A vertical view of the erosion pattern developed by the above flow shows that the revetment is just beginning to ravel from the toe of the bank revetment. The increased depth of scour downstream of the bridge section is becoming more apparent.

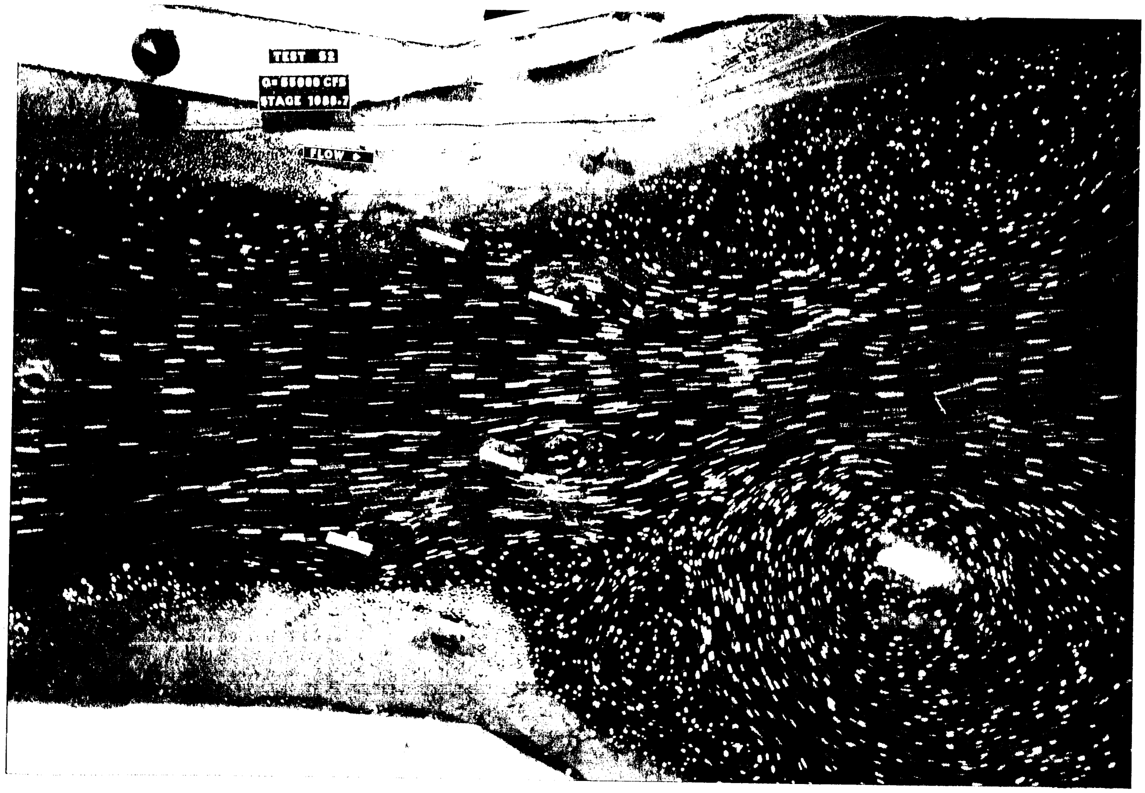


Photo 26

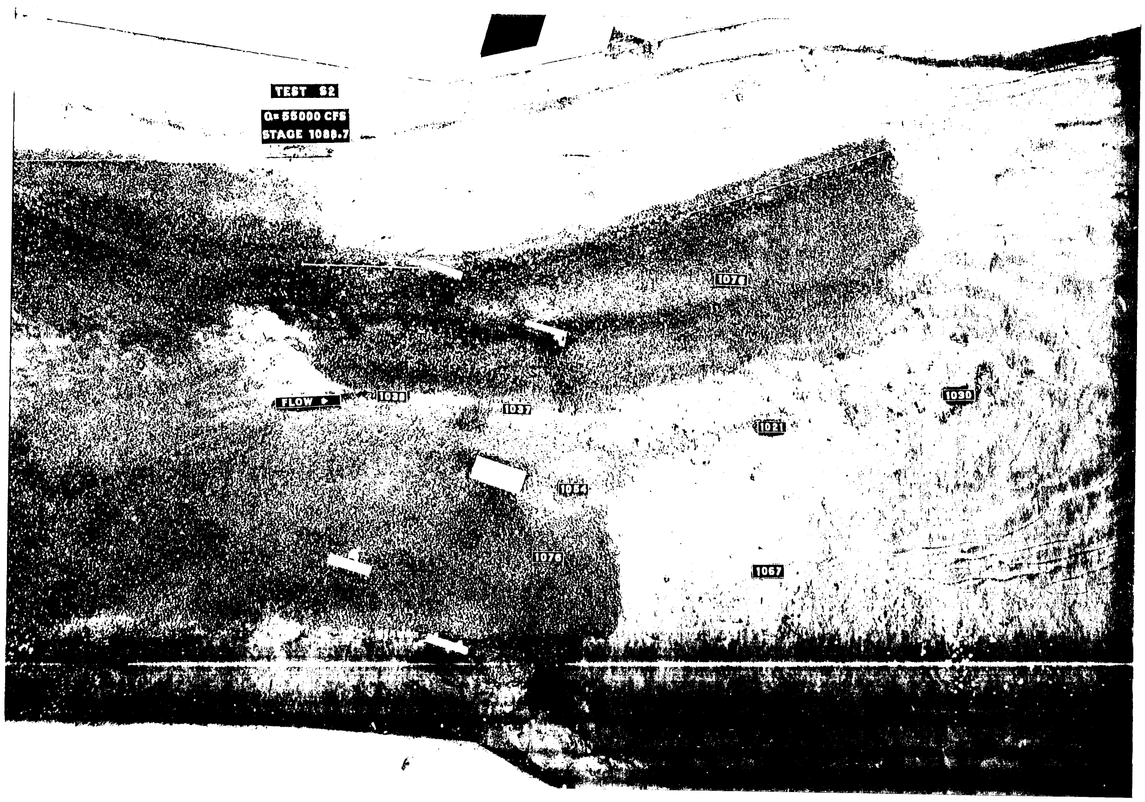


Photo 27

PHOTO 28 (Serial No. 142-93) In order to show more clearly whether the rock revetment is being scoured, strips of colored rock particles have been placed on the bed on the bridge centerline and along the banks. This photo was made befor the experiment was performed.

PHOTO 29 (Serial No. 142-95) This photo shows the same bed at the end of the test with a discharge of 55,000 cfs and the stage reduced to Elev. 1087.4. The revetment is relatively undisturbed except immediately downstream of pier 3 of the downstream bridge. Here the revetment has been eroded to a considerable extent by the flow in the wake of the pier. On the left bank downstream of the bridge section, it appeared that some of the rock revetment may have been moved and deposited further downstream on the bed.

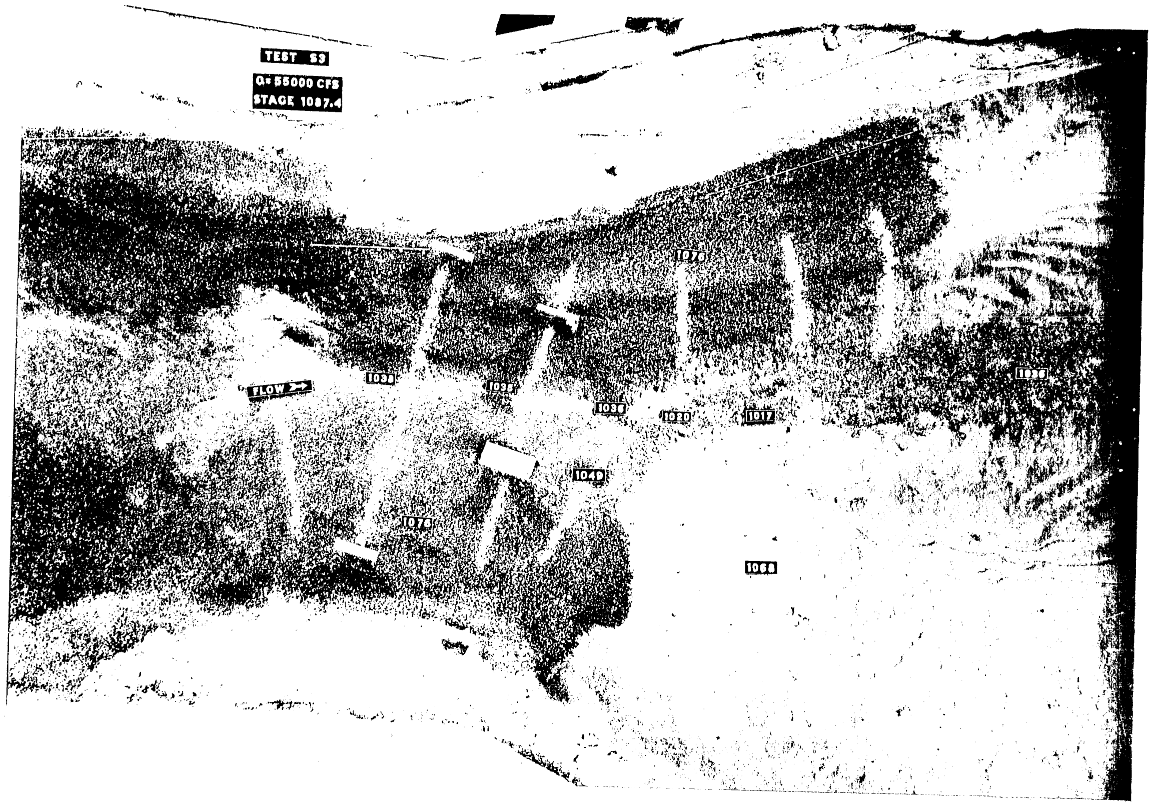


Photo 28

Photo 29

PHOTO 30 (Serial No. 142-96) This view shows the water surface pattern for a discharge of 55,000 cfs with the stage at the bridges reduced to 1085.5 ft. The high velocities through the constricted section give rise to a strong jet which penetrates the flow downstream of the bridge and generates strong vortices on both sides of the channel. The disturbance caused by the pier in the flow is also clearly apparent.

PHOTO 31 (Serial No. 142-98) The extremely high velocities through the constricted section and the jet created thereby which attacks the bed downstream of the bridge have caused very considerable erosion in this region. The photo also shows that the area of the bed between the piers has now been completely covered with rock particles ravelled from the bank revetment on both sides of the channel. Some of the rock has also been carried into the scour hole downstream.

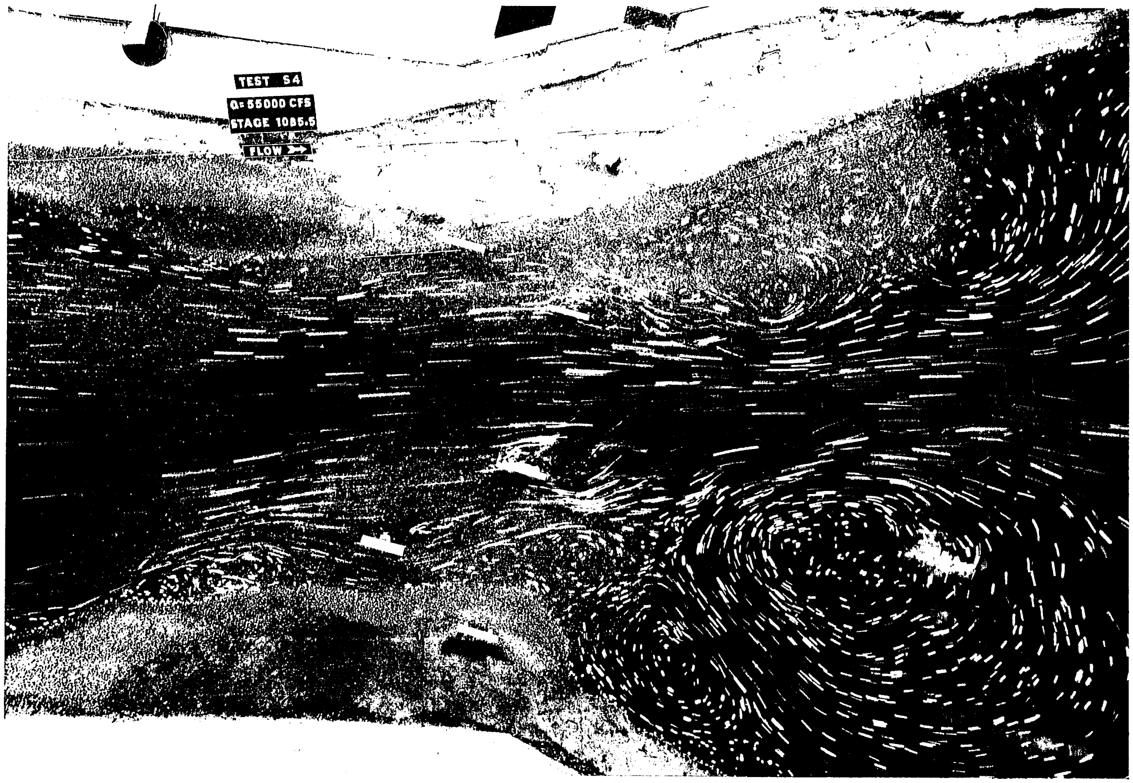


Photo 30

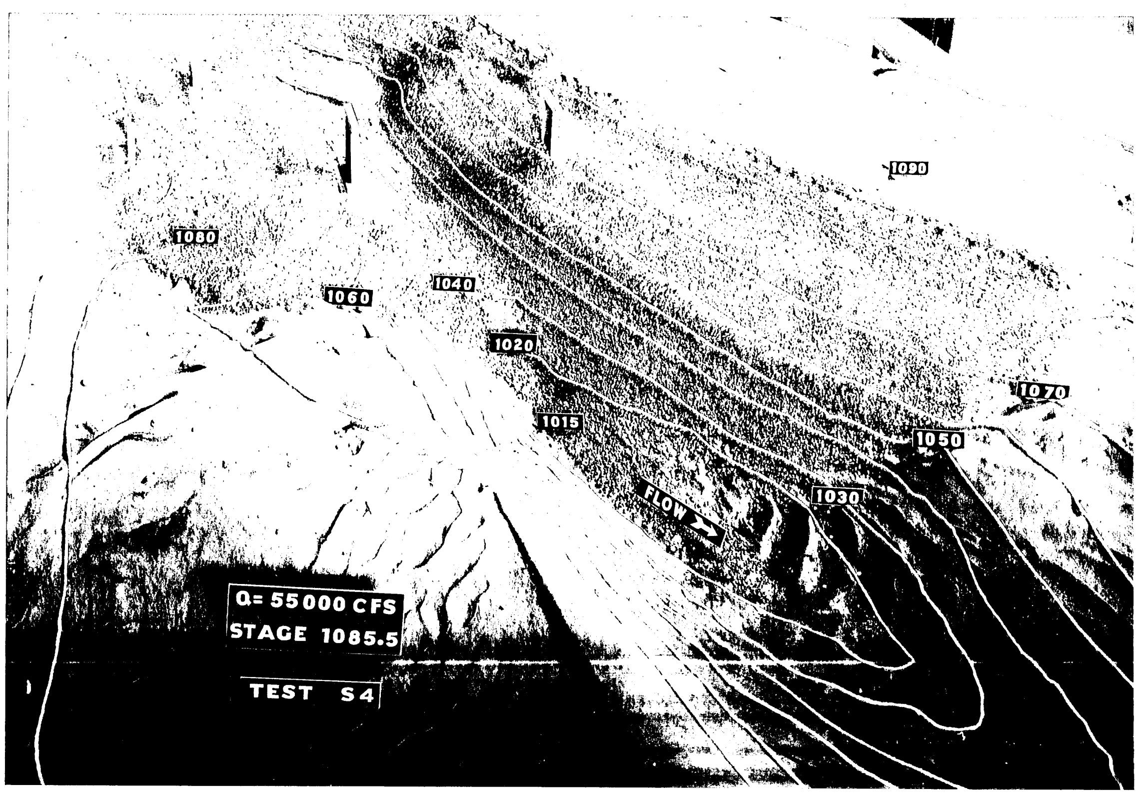


Photo 31

PHOTO 32 (Serial No. 142-118) This photo shows the original conditions of the stream channel and the flow pattern upstream of the bridge cross-sections before any protective devices were installed along the right bank. The significant aspect of this flow is the angle of attack on the bridge piers of the downstream bridge and the extent of the zone of separation along the left bank. It appears that in time of flood, the right bank may be attacked by the high velocities in this neighborhood.

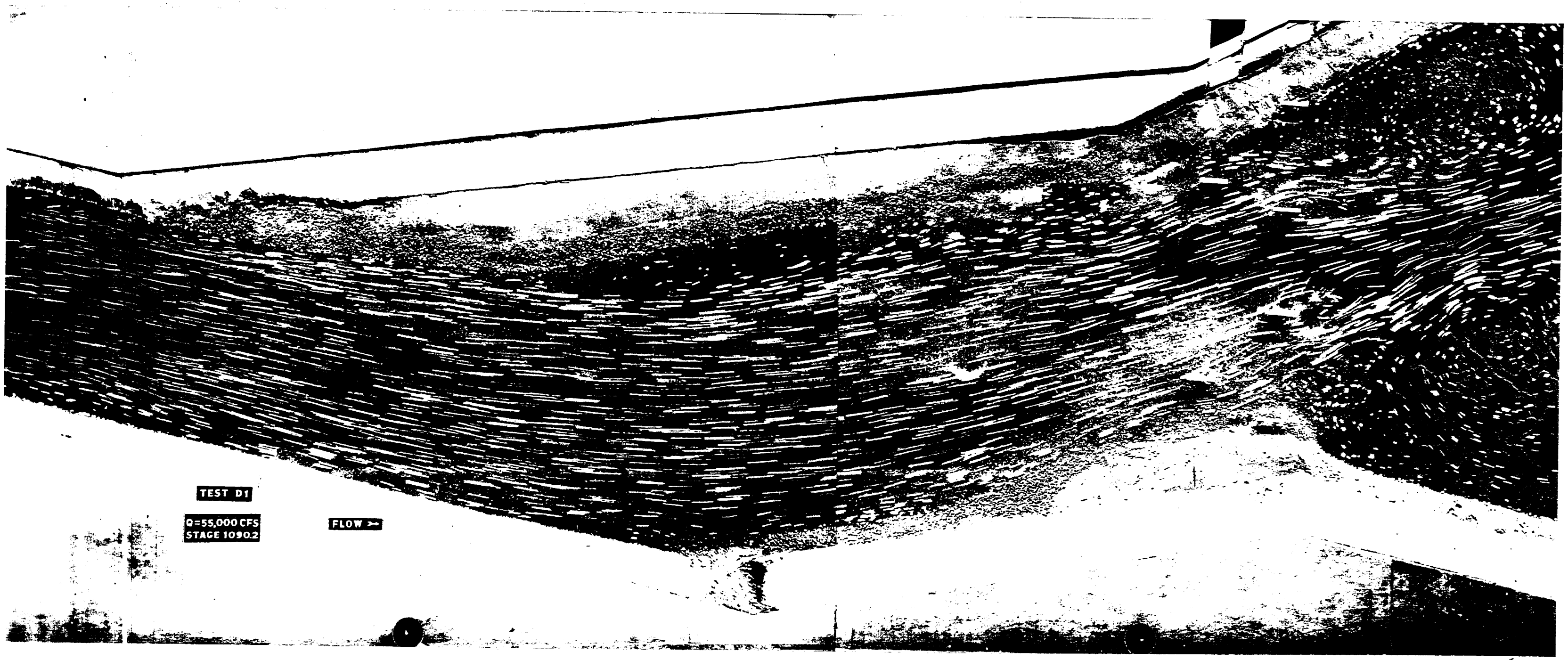


Photo 32

PHOTO 33 (Serial No. 142-119) For this test four relatively long dikes have been installed along the right bank. The effect of the dikes is to push the flow over towards the left bank and to create relatively large areas of quiet water between the dikes on the right bank. The angle of attack on the bridge piers has been greatly reduced.

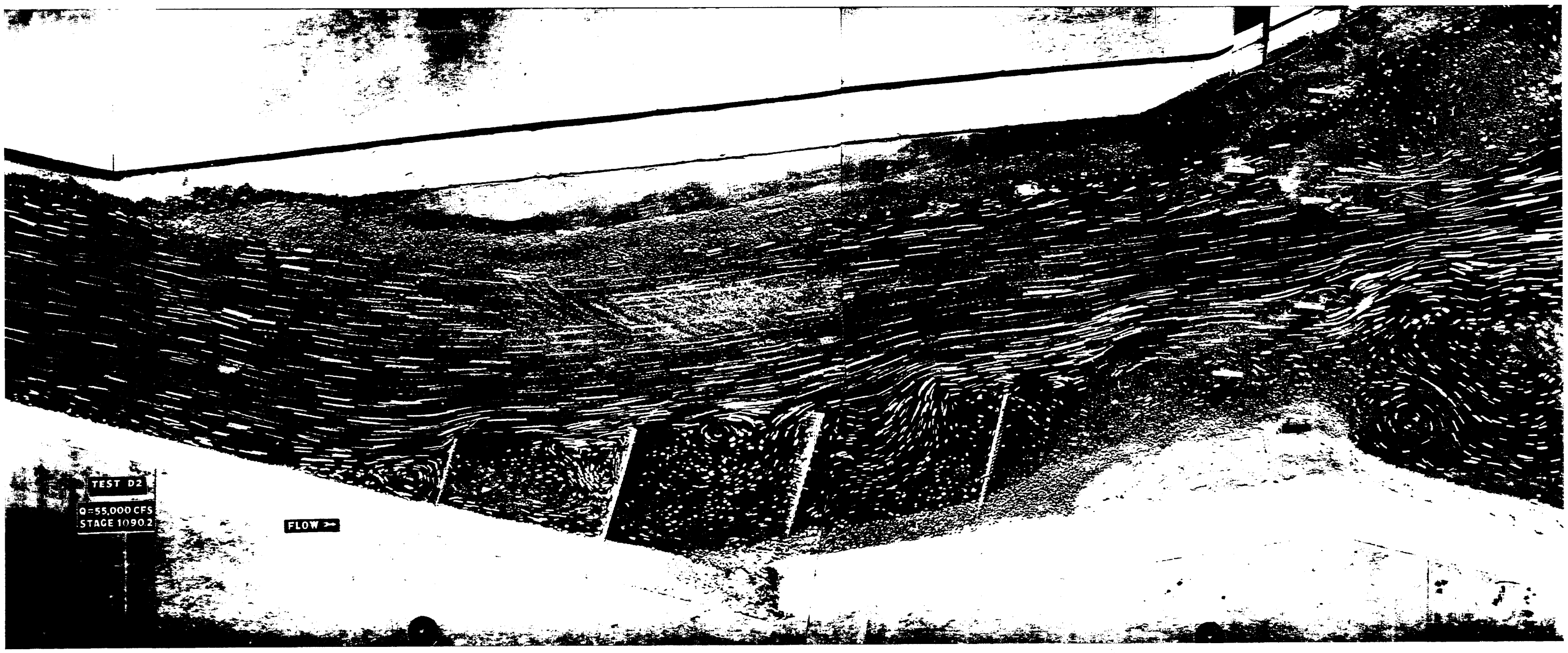


Photo 33

PHOTO 34 (Serial No. 142-120) By shortening the dikes and decreasing the spacing, the flow pattern is somewhat improved in that the mean velocity has been reduced. The right shore is still protected from erosion and the angle of attack has been decreased below that for the original conditions.

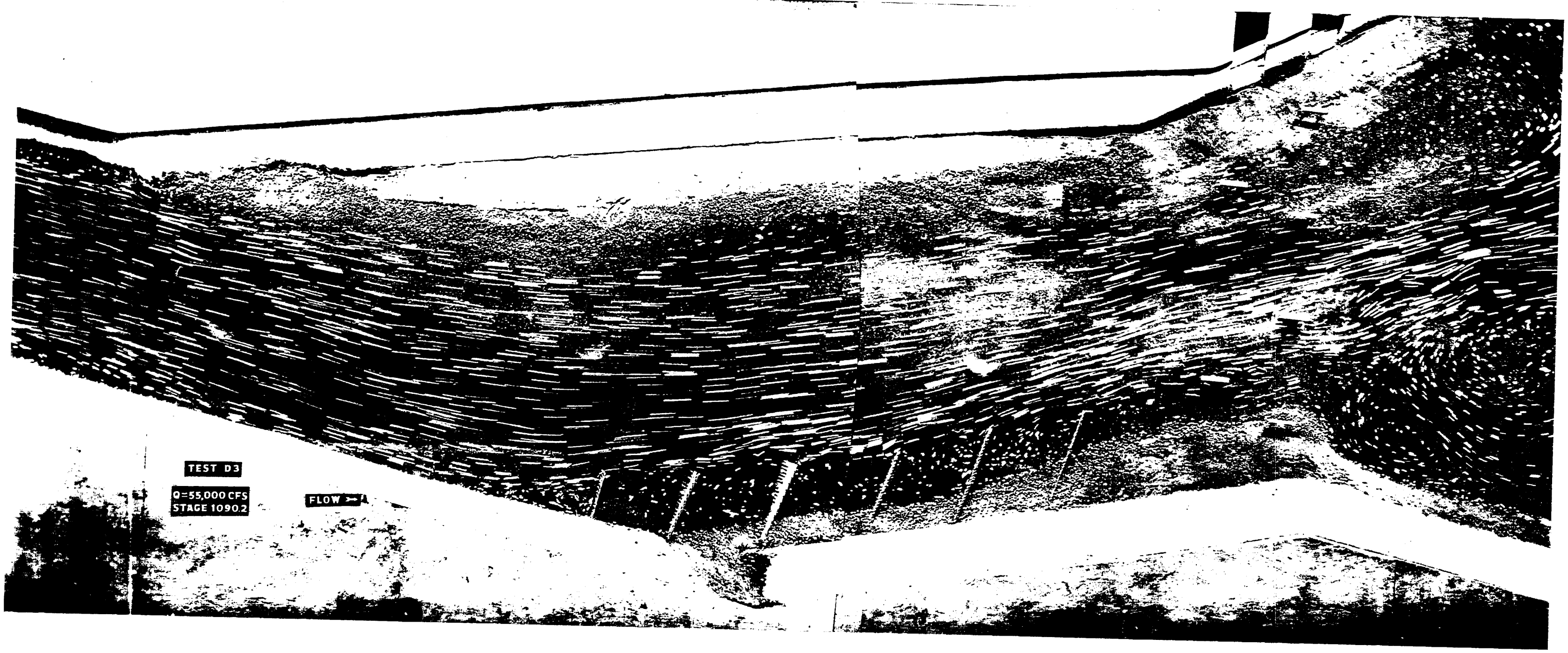


Photo 34

PHOTO 35 (Serial No. 142-121) This test is similar to the previous test except that a training dike has been installed at the upstream end of the dike field in order to provide a better transition from the channel to the first dike. The flow pattern is very similar to that observed in Photo 34.

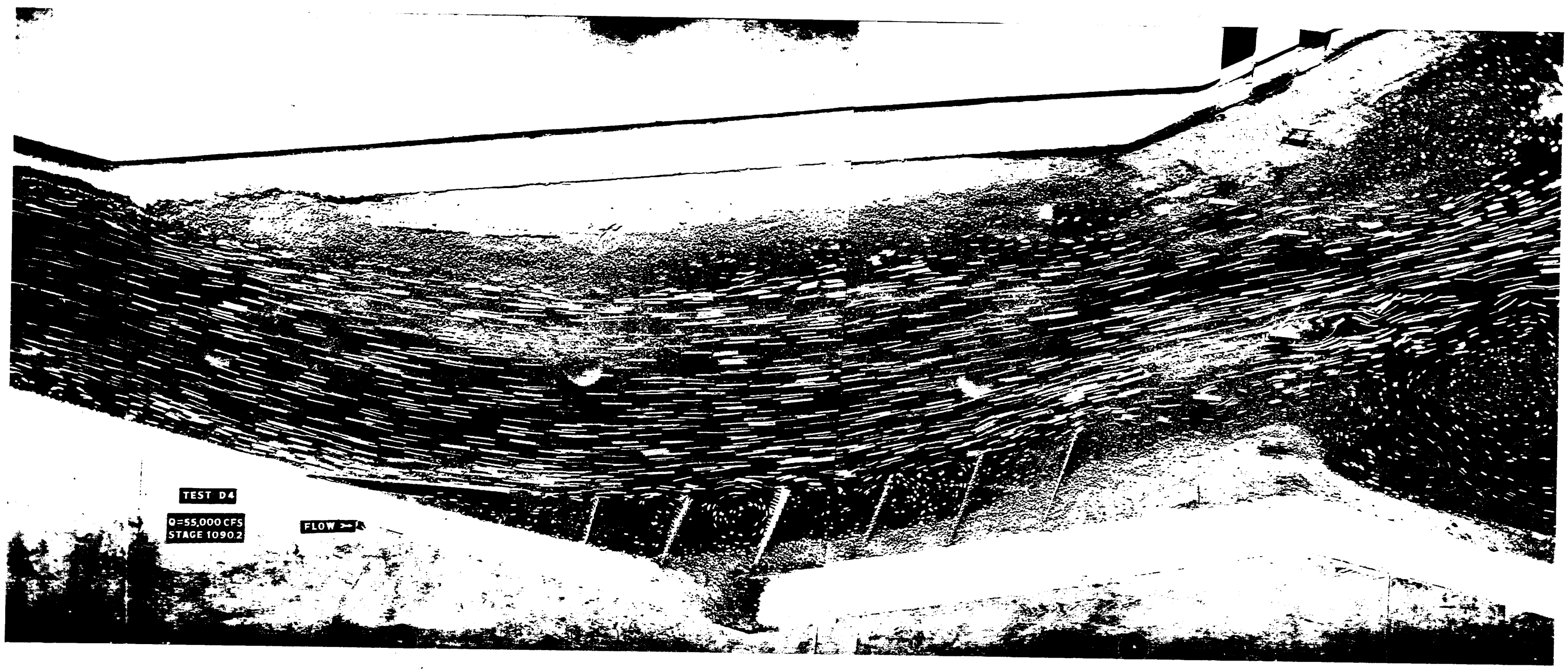


Photo 35

PHOTO 36 (Serial No. 142-122) For this test the length of the dikes has been drastically reduced and the number of dikes has been increased. This system has relatively little effect on the flow pattern but it does provide scour protection along the right bank.

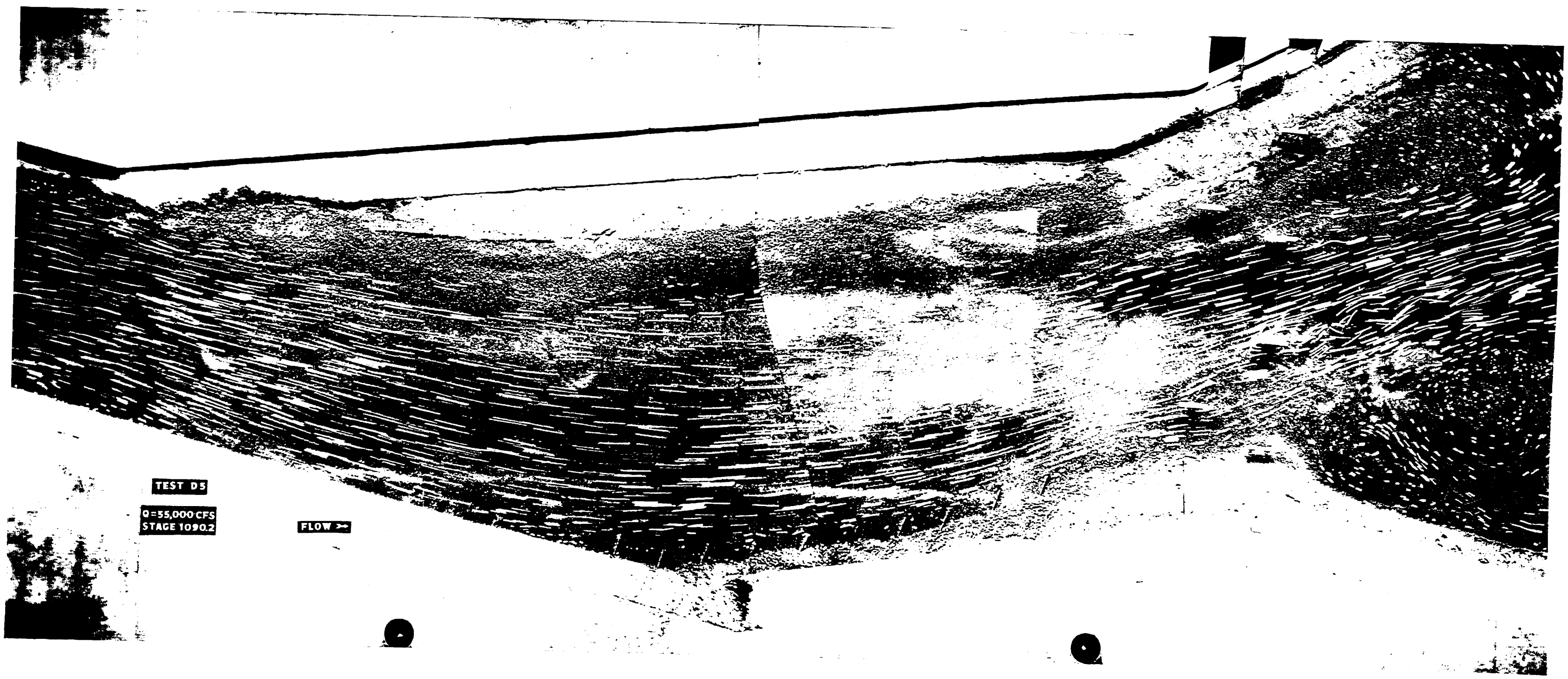


Photo 36

PHOTO 37 (Serial No. 142-123) An attempt was made to direct the flow and protect the right bank by providing a training dike at the upstream end of the curve in place of the system of dikes jutting into the flow. This system is somewhat effective in that it creates a zone of separation along the right bank in which the velocities are greatly reduced. In addition to the zone of separation, there is a slightly reduced angle of attack on the bridge piers.

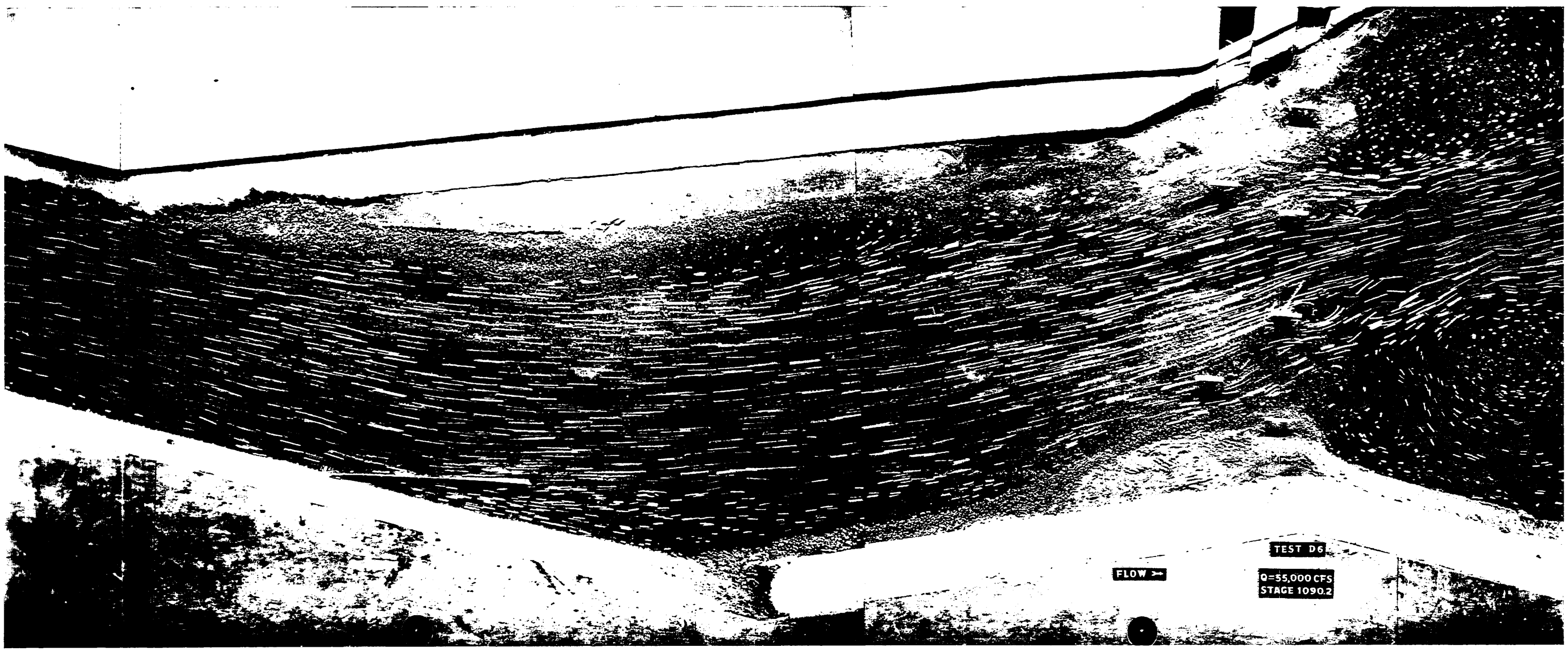


Photo 37

PHOTO 38 (Serial No. 142-124) In this experiment the training dike has been increased in length by 100 ft. The flow pattern, however, is very similar to that shown in Photo 37 and the added length of dike has relatively little effect in modifying the flow pattern.

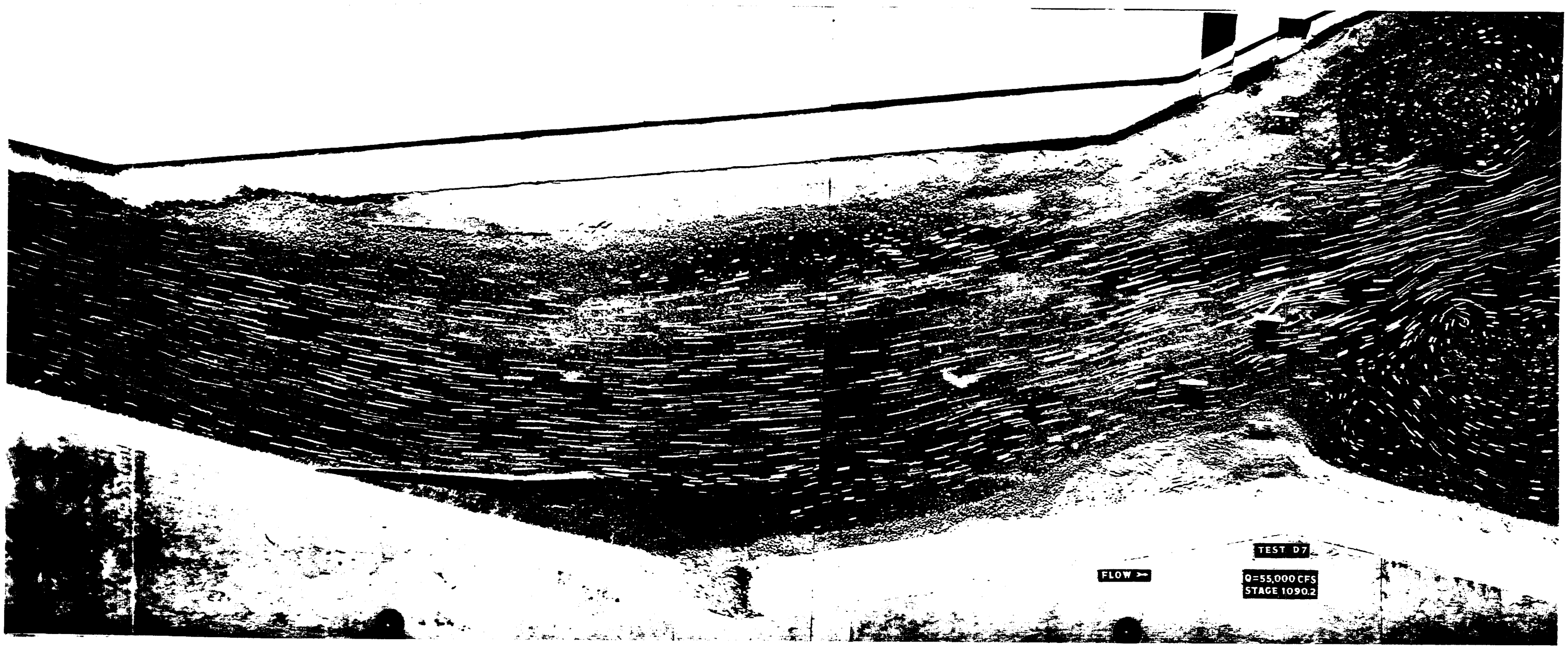


Photo 38

LIST OF CHARTS

CHART 15 (142B442-34) Channel stabilization test, Big Sioux project. Test S1, cross-sections at bridges showing erosion for discharge of 55,000 cfs.

CHART 16 (142B442-35) Channel stabilization test, Big Sioux project. Test S1, longitudinal water surface and bed profiles for various discharges.

CHART 17 (142B442-38) Channel stabilization tests, Big Sioux project. River bed cross-sections showing erosion for various discharges.

CHART 18 (142B442-39) Channel stabilization test, Big Sioux project. Stabilization plan 1. River bed cross-sections eroded by discharge of 55,000 cfs, at various stages.

CHART 19 (142B442-36) Channel stabilization tests, Big Sioux project. Stabilization plan 1. Longitudinal water surface and river bed profiles, eroded by a discharge of 55,000 cfs for various stages.

CHART 20 (142B442-44) Channel stabilization tests, Big Sioux project. Stabilization plan 1. Surface velocity patterns for a discharge of 55,000 cfs and various stages.

CHART 21 (142B442-45) Channel stabilization tests, Big Sioux project. Stabilization plan 1. Velocity distributions at 5 ft below water surface for discharge of 55,000 cfs and various stages.

CHART 22 (14B442-51) Big Sioux Project Model Basin showing plan and alignment of the river upstream and downstream of the bridge crossing.

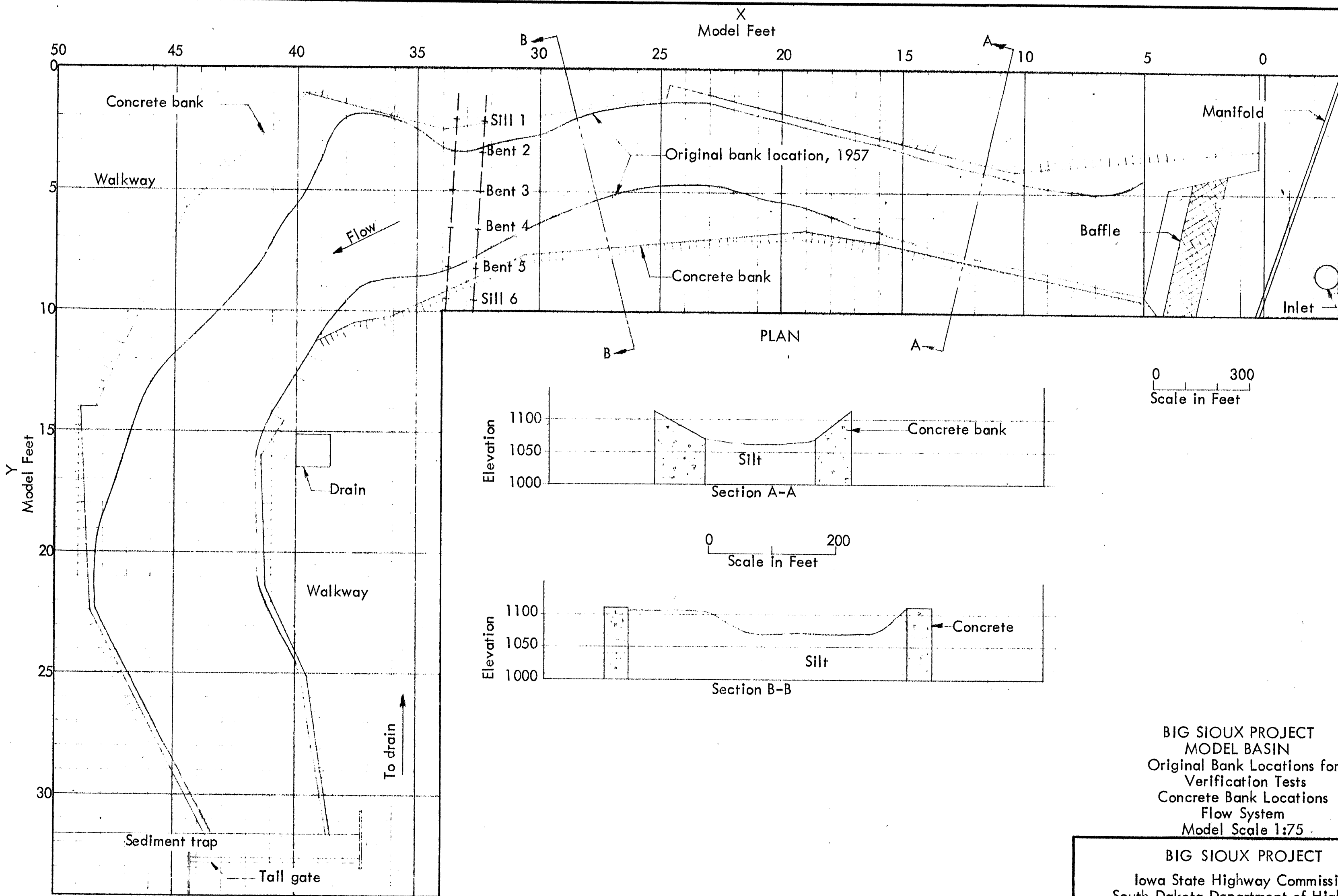
CHART 23 (142B442-47) Big Sioux Project Upstream Permeable Dike Tests. Dike designs for Tests D1 and D2. Q = 55,000 cfs.

CHART 24 (142B442-48) Big Sioux Project Upstream Permeable Dike Tests. Dike designs Tests D3 and D4. Q = 55,000 cfs.

CHART 25 (142B442-49) Big Sioux Project Dike Designs, Test D5. Q = 55,000 cfs.

CHART 26 (142B442-50) Big Sioux Project Upstream Permeable Dike Tests. Dike design Test D6 and D7. Q = 55,000 cfs.

CHART 1



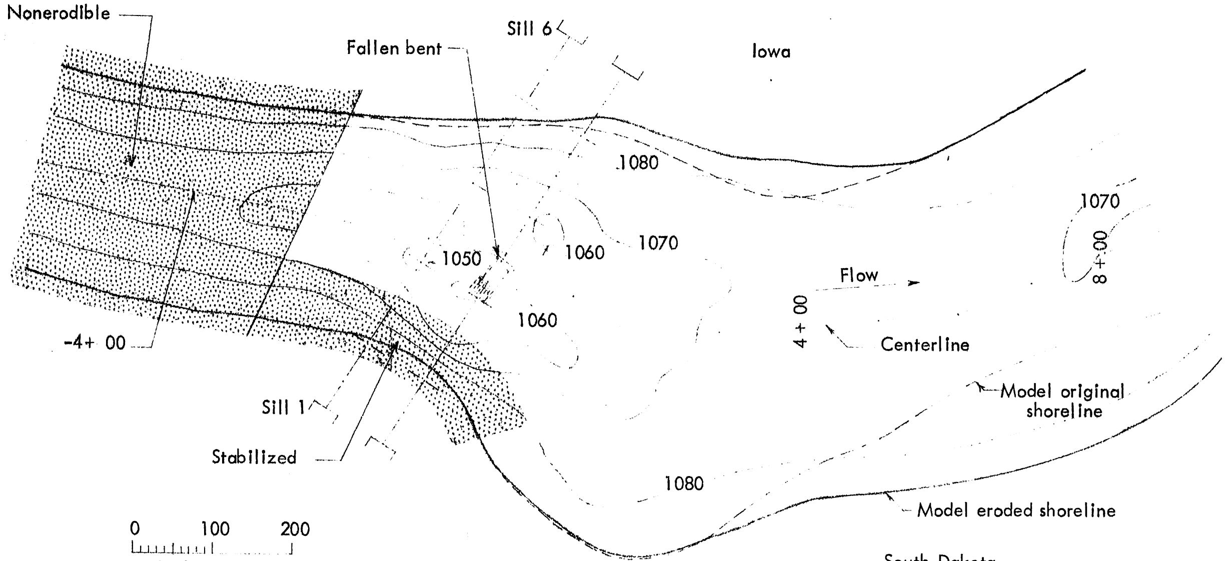
BIG SIOUX PROJECT
MODEL BASIN
Original Bank Locations for
Verification Tests
Concrete Bank Locations
Flow System
Model Scale 1:75

BIG SIOUX PROJECT

Iowa State Highway Commission
South Dakota Department of Highways

SAINT ANTHONY FALLS HYDRAULIC LABORATORY
UNIVERSITY OF MINNESOTA

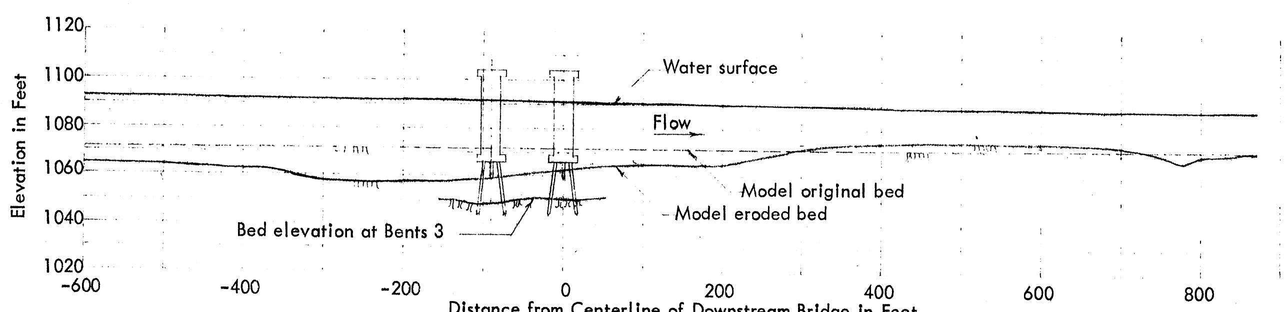
DRAWN PN | CHECKED | APPROVED
SCALE | DATE 8-5-64 | NO. 142B442-21



RIVER BED CONTOURS
PLAN

Legend:
- - - Original conditions
——— Eroded conditions

Note: The stage at the bridges was maintained at elevation 1090.8 ft throughout the test.



WATER SURFACE AND RIVER BED PROFILES
CENTERLINE

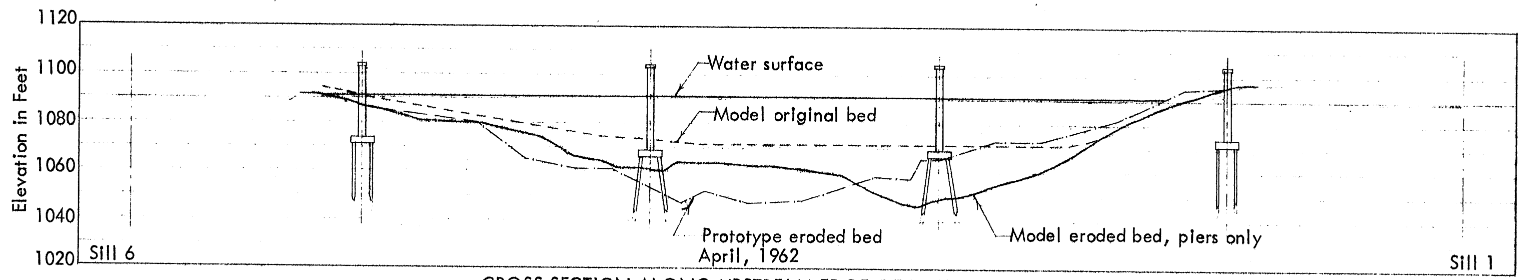
TEST V 1
River Bed Contours
Water Surface and River Bed Profiles
Q = 55,000 cfs
3.5 Hr. Model Test Run
Model Scale 1:75

BIG SIOUX PROJECT

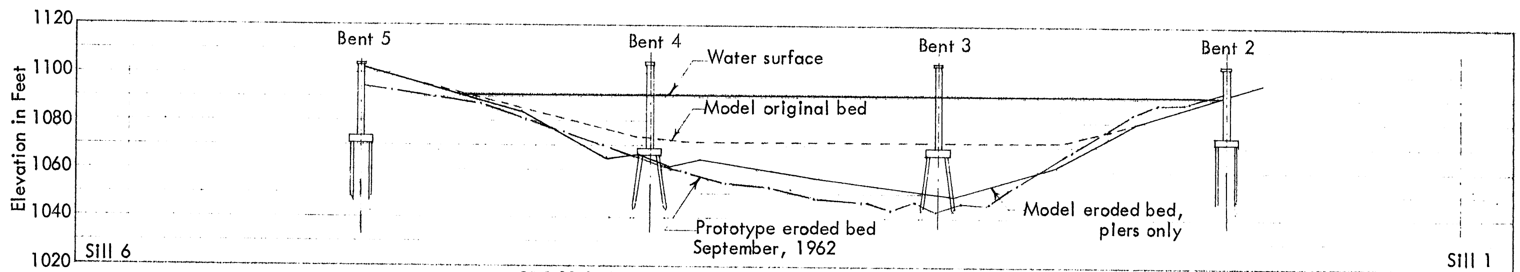
Iowa State Highway Commission
South Dakota Department of Highways
MODEL VERIFICATION TESTS

SAINT ANTHONY FALLS HYDRAULIC LABORATORY
UNIVERSITY OF MINNESOTA

DRAWN CWC	CHECKED	APPROVED
SCALE	DATE 8-11-64	NO. 142B442-22



CROSS SECTION ALONG UPSTREAM EDGE OF DOWNSTREAM BRIDGE

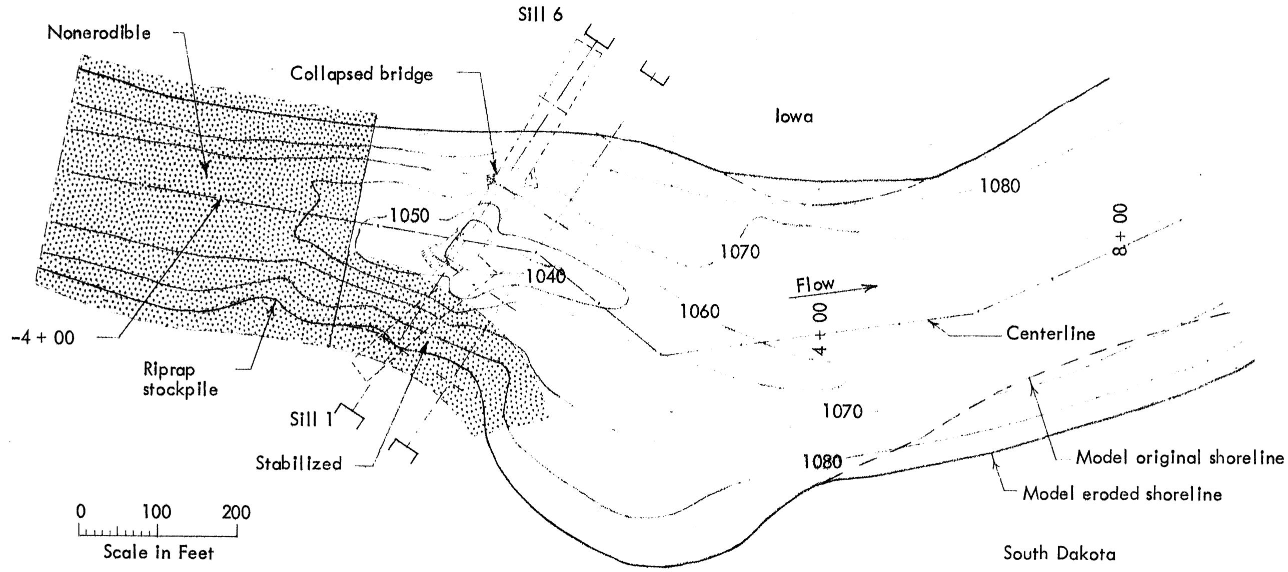


CROSS SECTION ALONG CENTERLINE OF UPSTREAM BRIDGE

Note: The stage at the bridges was maintained at elevation 1090.8 ft throughout the test.

TEST V 1
Cross Sections at Bridges
Q = 55,000 cfs
3.5 hr Model Test Run
Model Scale 1:75

BIG SIOUX PROJECT		
Iowa State Highway Commission South Dakota Department of Highways MODEL VERIFICATION TESTS		
SAINT ANTHONY FALLS HYDRAULIC LABORATORY UNIVERSITY OF MINNESOTA		
DRAWN ACY	CHECKED	APPROVED
SCALE	DATE 8-18-64	NO. 142B442-23

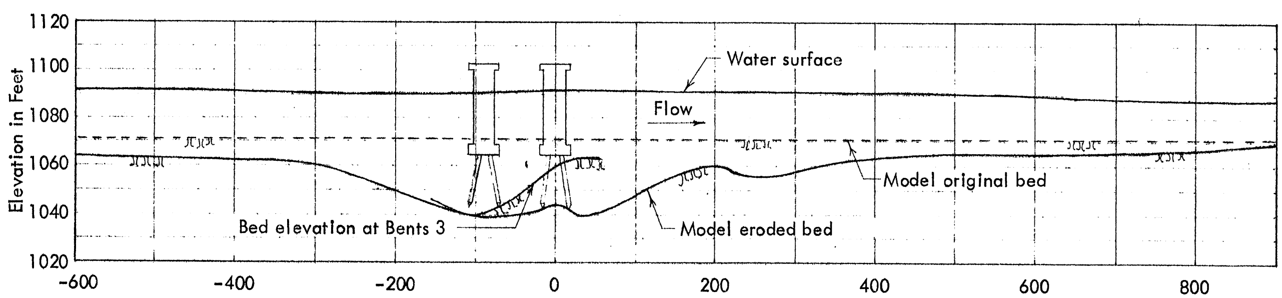


RIVER BED CONTOURS
PLAN

Legend:

- - - - Original conditions
——— Eroded conditions

Note: The stage at the bridge was maintained at elevation 1090.8 ft throughout the test.



WATER SURFACE AND RIVER BED PROFILES
CENTERLINE

TEST V2 - PART 1
Piers and Collapsed Bridge
River Bed Contours
Water Surface and River Bed Profiles
Q = 55,000 cfs
10 hr. Model Test Run
Model Scale 1:75

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Iowa State Highway Commission
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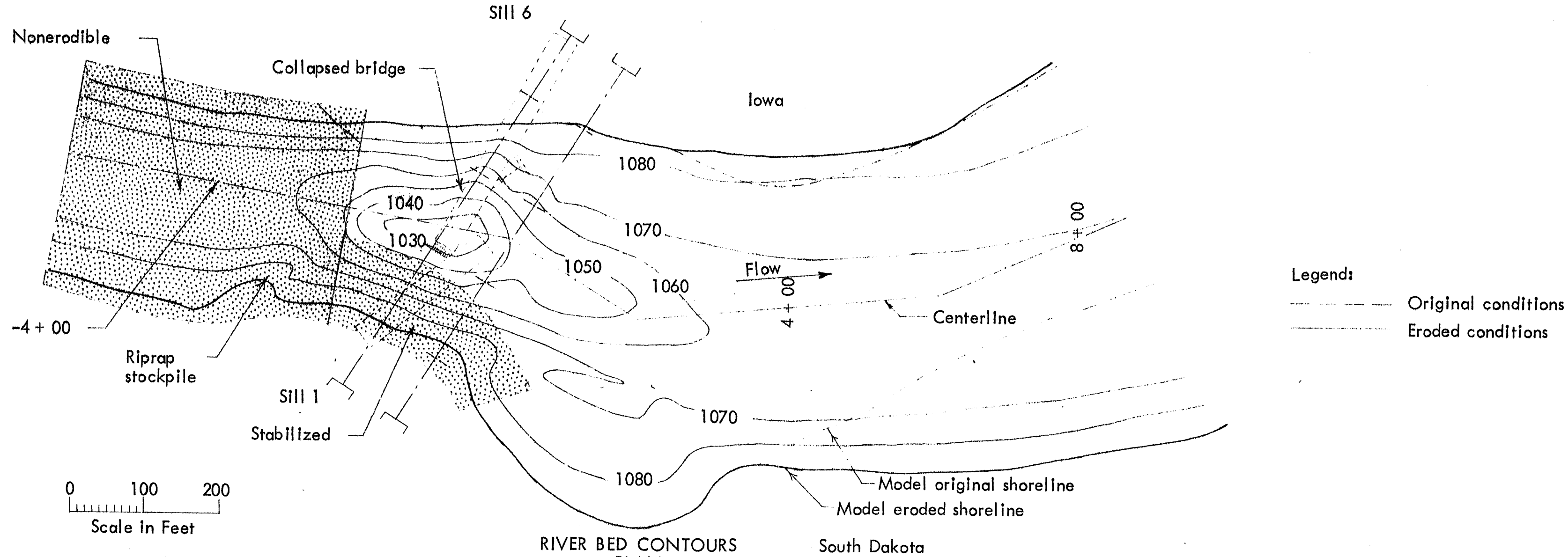
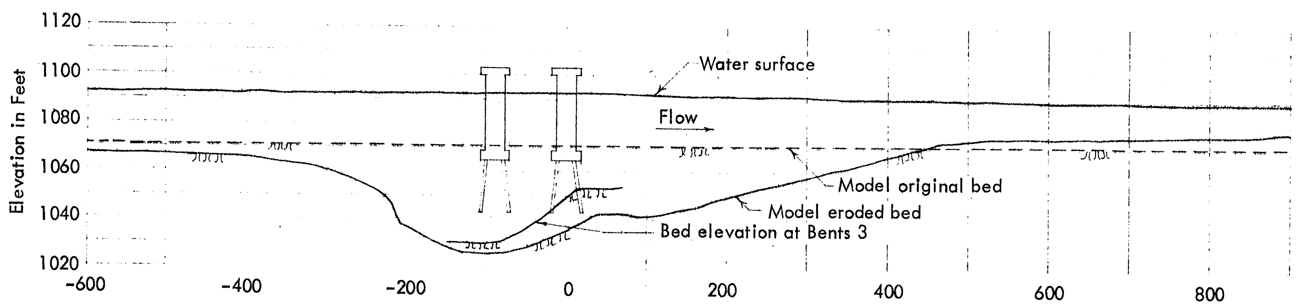
DRAWN PJN	CHECKED	APPROVED
SCALE	DATE 9-4-64	NO. 142B442-25

CHART 6



RIVER BED CONTOURS
PLAN



WATER SURFACE AND RIVER BED PROFILES
CENTERLINE

TEST V3
Piers and Collapsed Bridge
River Bed Contours
Water Surface and River Bed Profiles
Q = 65,000 cfs
Stage = 1092.4 ft
8.0 hr. Model Test Run
Model Scale 1:75

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Iowa State Highway Commission
South Dakota Department of Highways
MODEL VERIFICATION TESTS

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UNIVERSITY OF MINNESOTA

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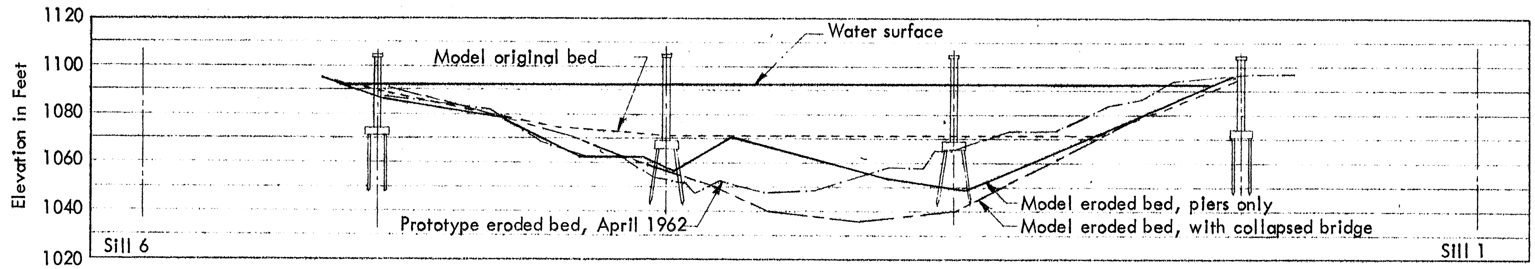
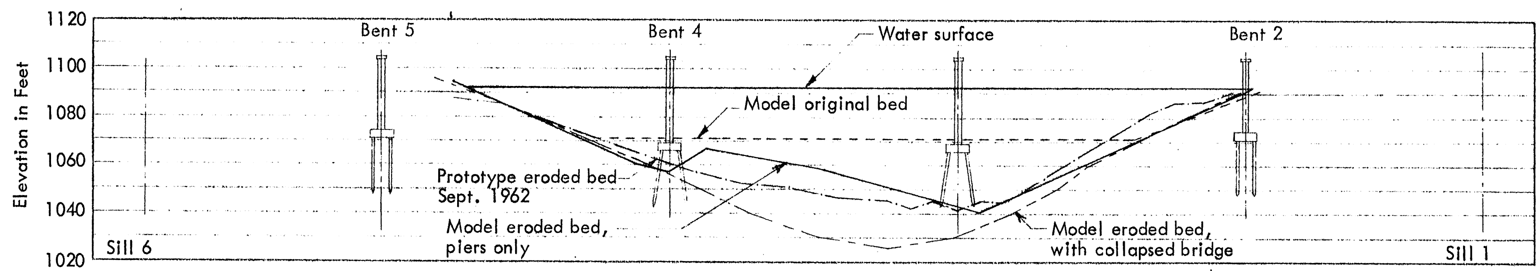
SCALE	DATE 9-4-64	NO. 142B442-27



CROSS SECTION ALONG UPSTREAM EDGE OF DOWNSTREAM BRIDGE



CROSS SECTION ALONG CENTERLINE OF UPSTREAM BRIDGE

TEST V3
Cross Sections at Bridges
Q = 65,000 cfs
Stage = 1092.4 ft
8.0 hrs. Model Test Run
Model Scale 1:75

Notes: The model cross sections for piers only were taken at 0.25 hrs., immediately after the failure of Bent 3 of the upstream bridge. Bed erosion had not ceased.

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MODEL VERIFICATION TESTS

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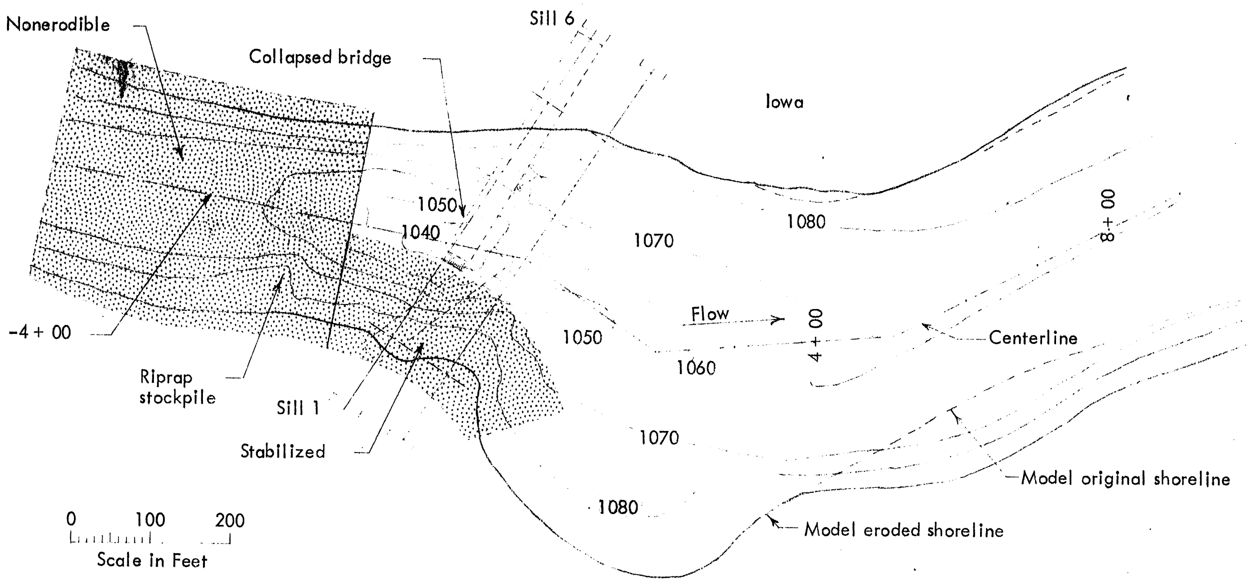
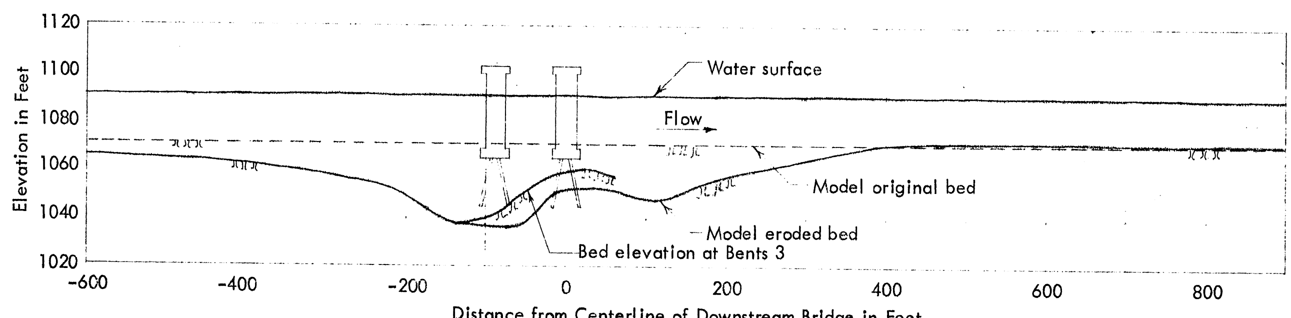
DRAWN GHC	CHECKED	APPROVED
SCALE	DATE 9-8-64	NO. 142B442-28



Legend:
Original conditions
Eroded conditions

RIVER BED CONTOURS
PLAN



WATER SURFACE AND RIVER BED PROFILES
CENTERLINE

TEST V4 - PART 2
Piers and Collapsed Bridge
River Bed Contours
Water Surface and River Bed Profiles
Q = 52,000 cfs
Stage = 1090.8 ft
10.0 hr. Model Test Run
Model Scale 1:75

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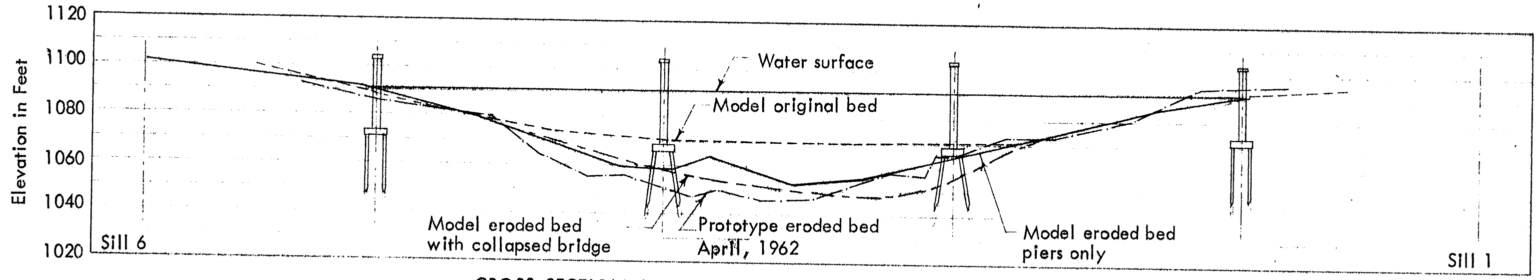
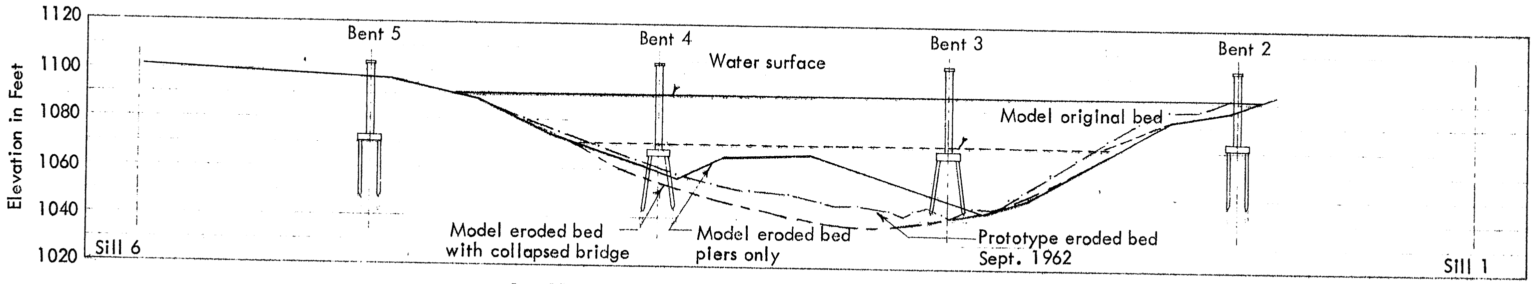
Iowa State Highway Commission
South Dakota Department of Highways
MODEL VERIFICATION TESTS

SAINT ANTHONY FALLS HYDRAULIC LABORATORY
UNIVERSITY OF MINNESOTA

DRAWN CLA	CHECKED ald	APPROVED
SCALE	DATE 9-8-64	NO. 142B442-30



CROSS SECTION ALONG UPSTREAM EDGE OF DOWNSTREAM BRIDGE



CROSS SECTION ALONG CENTERLINE OF UPSTREAM BRIDGE

Note: Test V4:
Part 1: Piers only; 3.5 hr. model test run
Part 2: Piers and collapsed bridge; 6.5 hr. model test run
Total testing time: 10.0 hrs.

TEST V4
Cross Sections at Bridge
Q = 52,000 cfs
Stage = 1090.8 ft
Model Scale 1:75

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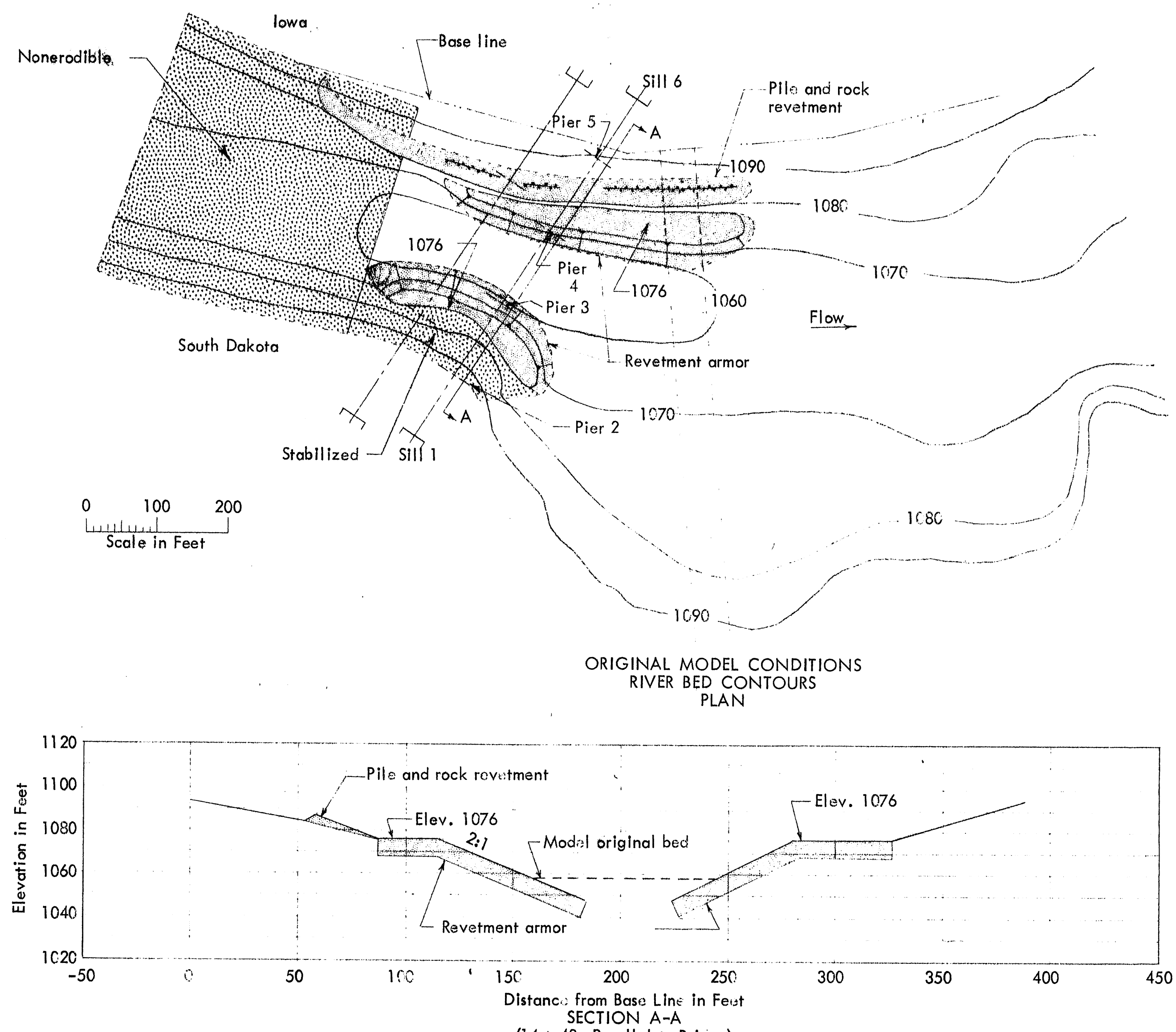
UNIVERSITY OF MINNESOTA

DRAWN ACY	CHECKED [illegible]	APPROVED
SCALE	DATE 8-18-64	NO. 142B442-31



Legend:

Rock revetment works

Notes:

1. Upstream channel straightened by excavation.
2. Bents 3 and 4 of the downstream bridge underpinned.
3. New bridge in place.
4. Pile and rock revetment in place.
5. Stabilization Plan 1 revetment armor.

ORIGINAL MODEL CONDITIONS
RIVER BED CONTOURS
PLAN

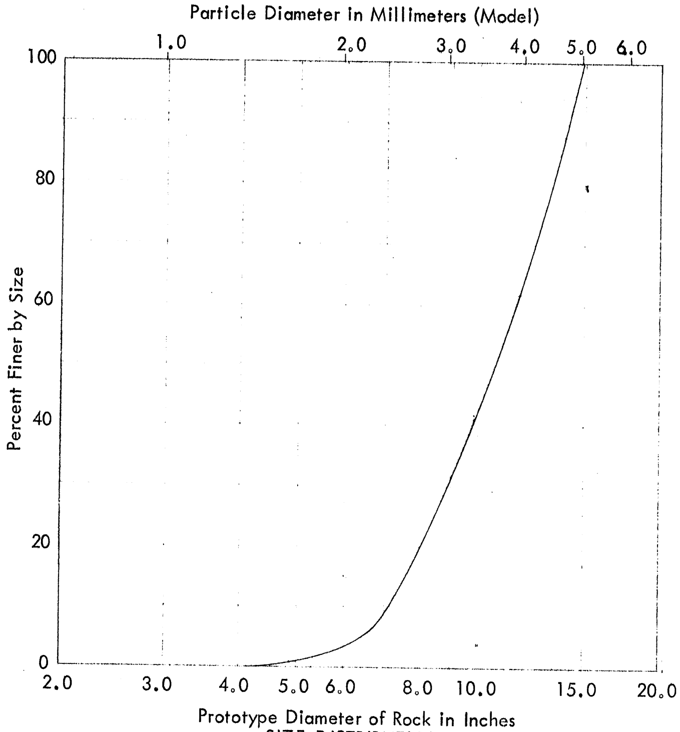
SECTION A-A
(14 + 40, Parallel to Bridge)

TEST S1
Stabilization Plan 1
Original Model Conditions
Model Scale 1:75

BIG SIOUX PROJECT
Iowa State Highway Commission
South Dakota Department of Highways
CHANNEL STABILIZATION TEST

SAINT ANTHONY FALLS HYDRAULIC LABORATORY
UNIVERSITY OF MINNESOTA

DRAWN KFW	CHECKED afd	APPROVED
SCALE	DATE 9-11-64	NO. 142B442-32



SIZE DISTRIBUTION
BY SIEVE ANALYSIS

$$d_g \text{ (mm-model) } \left(\frac{75}{25.4}\right) = d_g \text{ (inches-prototype)}$$

1 mm (model) = 2.95 inches (prototype)

Note: Size distribution obtained by standard sieve analysis of a 1540 gm sample.

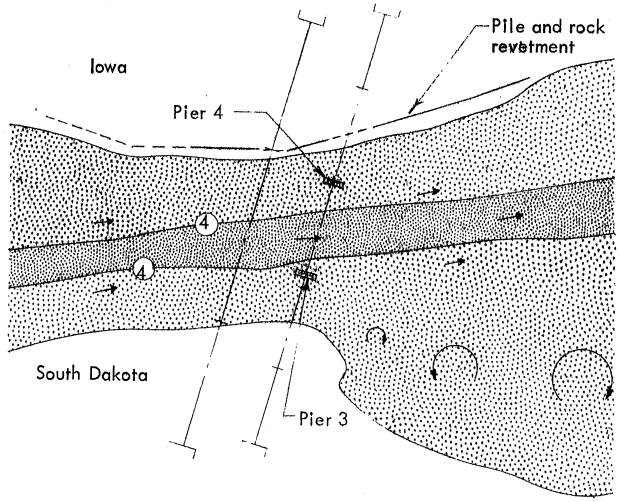
REVETMENT MATERIAL
Size Distribution by Sieve Analysis
Model Scale 1:75

BIG SIOUX PROJECT

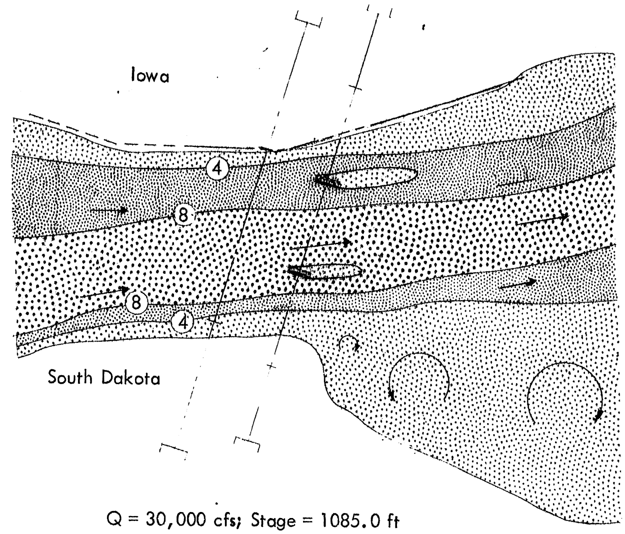
Iowa State Highway Commission
South Dakota Department of Highways
CHANNEL STABILIZATION TESTS

SAINT ANTHONY FALLS HYDRAULIC LABORATORY
UNIVERSITY OF MINNESOTA

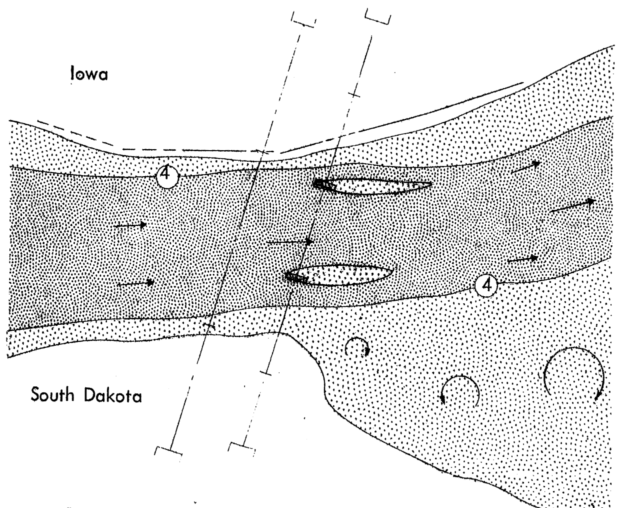
DRAWN CLA	CHECKED	APPROVED
SCALE	DATE 9-14-64	NO. 142B442-37



Q = 10,000 cfs; Stage = 1080.9 ft



Q = 30,000 cfs; Stage = 1085.0 ft



Q = 20,000 cfs; Stage = 1083.2 ft

0 10 20

Velocity
Feet per Second

Legend:

- 0 - 4 fps
- 4 - 8 fps
- above 8 fps

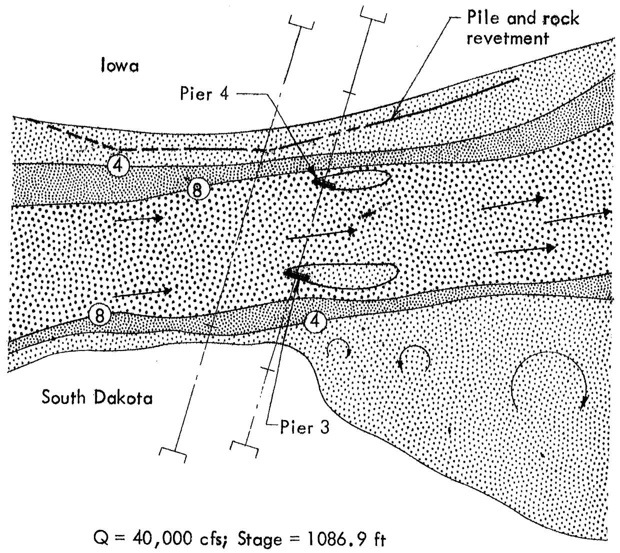
TEST S1
Surface Velocity Patterns
from Photographs
Model Scale 1:75

BIG SIOUX PROJECT

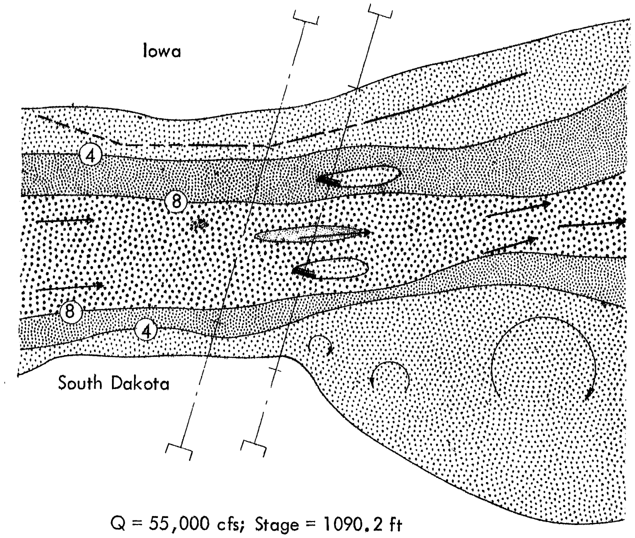
Iowa State Highway Commission
South Dakota Department of Highways
CHANNEL STABILIZATION TESTS

SAINT ANTHONY FALLS HYDRAULIC LABORATORY
UNIVERSITY OF MINNESOTA

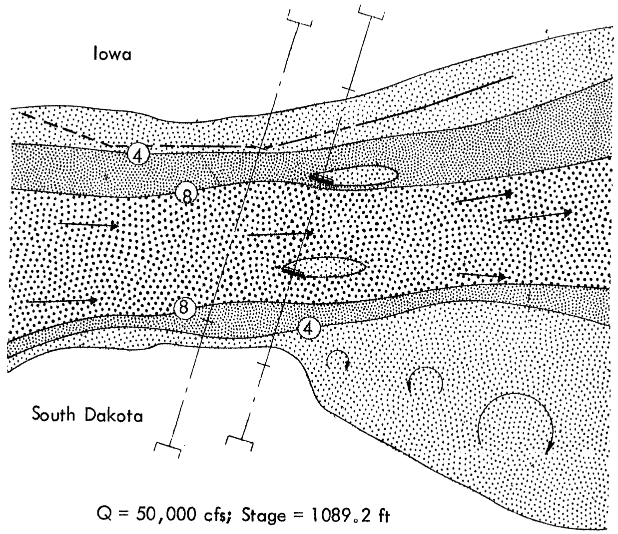
DRAWN ACY	CHECKED *[signature]*	APPROVED
SCALE	DATE 9-23-64	NO. 142B442-41



Q = 40,000 cfs; Stage = 1086.9 ft



Q = 55,000 cfs; Stage = 1090.2 ft



Q = 50,000 cfs; Stage = 1089.2 ft

0 10 20
Velocity
Feet per Second

Legend:

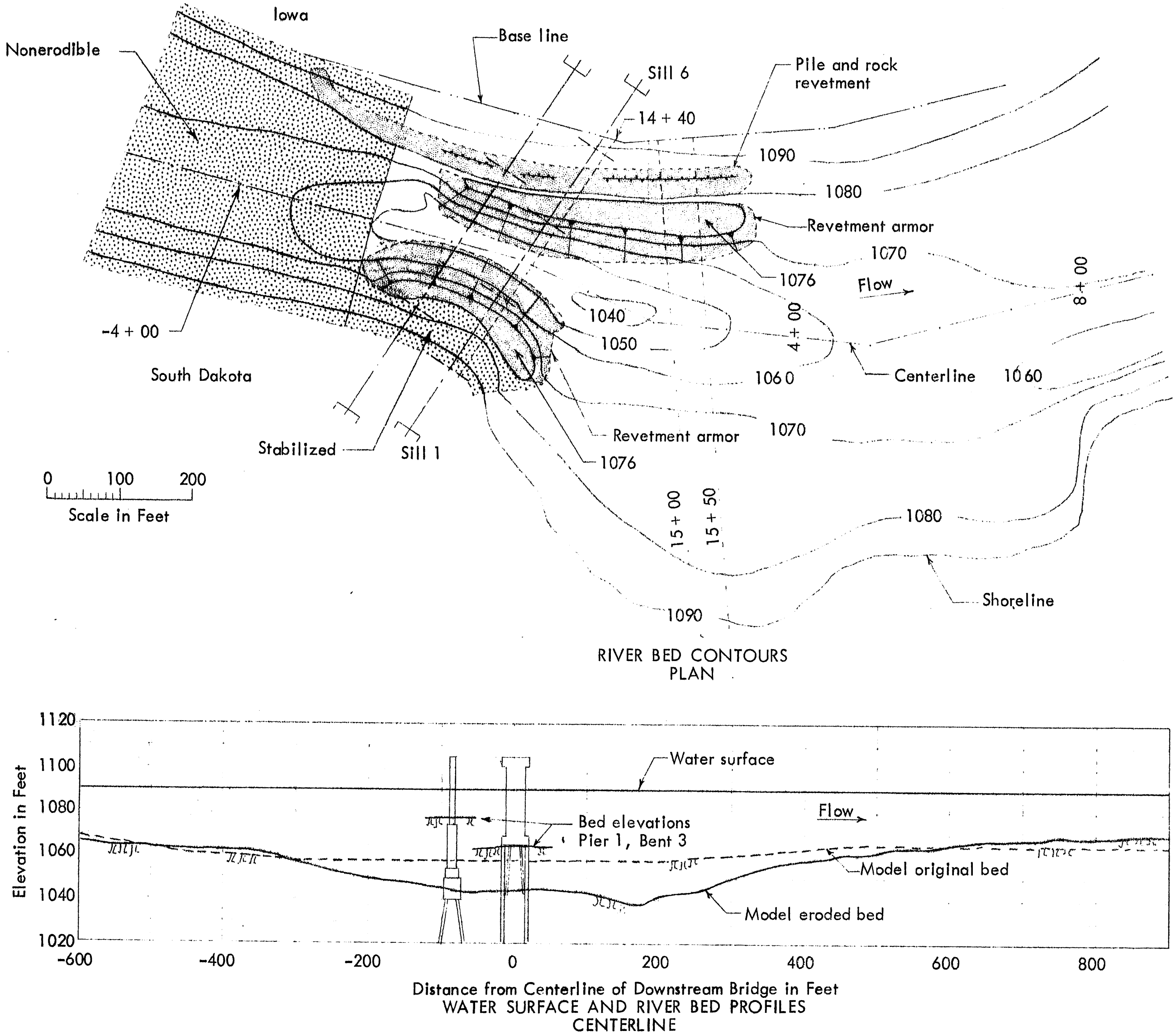
- 0 - 4 fps
- 4 - 8 fps
- 8 - 12 fps
- above 12 fps

TEST S1
Surface Velocity Patterns
from Photographs
Model Scale 1:75

BIG SIOUX PROJECT
Iowa State Highway Commission
South Dakota Department of Highways
CHANNEL STABILIZATION TESTS

SAINT ANTHONY FALLS HYDRAULIC LABORATORY
UNIVERSITY OF MINNESOTA

DRAWN ACY	CHECKED *alc*	APPROVED
SCALE	DATE 9-25-64	NO. 142B442-42



RIVER BED CONTOURS
PLAN

Distance from Centerline of Downstream Bridge in Feet
WATER SURFACE AND RIVER BED PROFILES
CENTERLINE

Legend:

Rock revetment works

Notes:

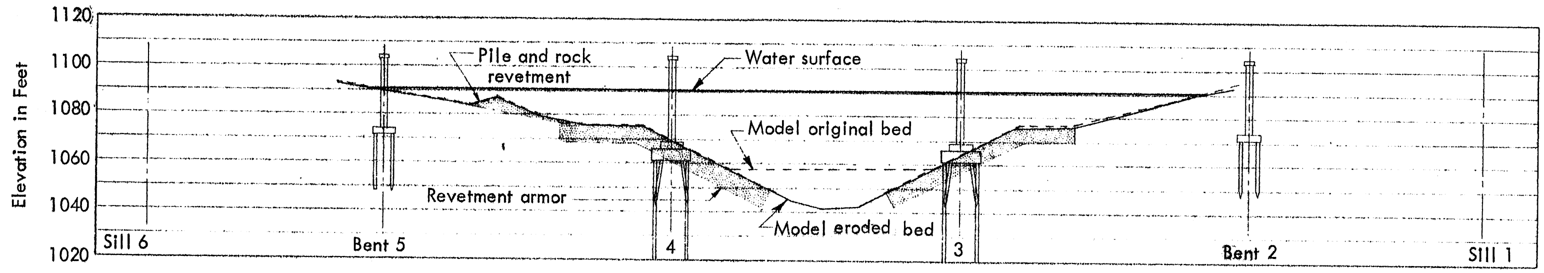
1. Upstream channel straightened by excavation.
2. Bents 3 and 4 of the downstream bridge underpinned.
3. New bridge in place.
4. Pile and rock revetment in place.
5. Stabilization Plan 1 revetment armor.

TEST S1
Stabilization Plan 1
River Bed Contours
Water Surface and River Bed Profiles
Q = 55,000 cfs
Stage = 1090.2 ft
Model Scale 1:75

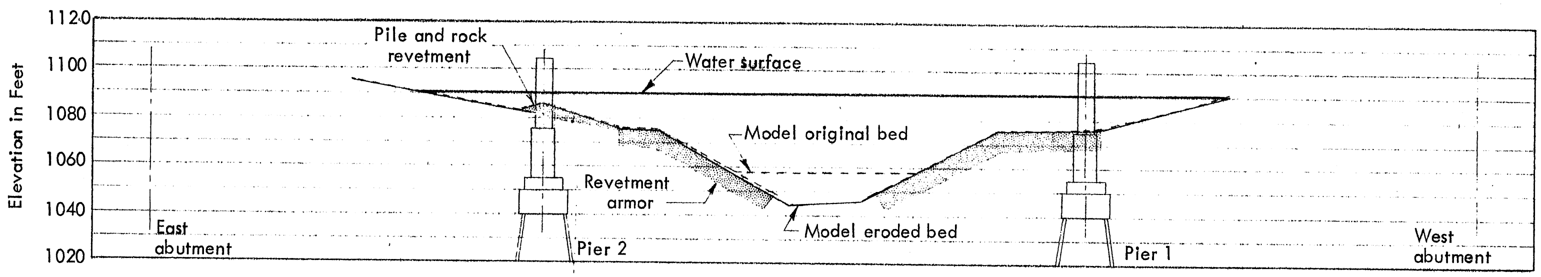
BIG SIOUX PROJECT
Iowa State Highway Commission
South Dakota Department of Highways
CHANNEL STABILIZATION TEST

SAINT ANTHONY FALLS HYDRAULIC LABORATORY
UNIVERSITY OF MINNESOTA

DRAWN KFW	CHECKED [illegible]	APPROVED
SCALE	DATE 9-23-64	NO. 142B442-33



CROSS SECTION ALONG CENTERLINE OF DOWNSTREAM BRIDGE



CROSS SECTION ALONG CENTERLINE OF UPSTREAM BRIDGE

TEST S1
STABILIZATION PLAN 1
Cross Sections at Bridges
Q = 55,000 cfs
Stage 1090.2 ft.
Model Scale 1:75

Note: During Test S1, flows of 10,000, 20,000, 30,000, 40,000, 50,000 and 55,000 cfs were established (in increasing order of magnitude) and maintained until erosion ceased in the test section.

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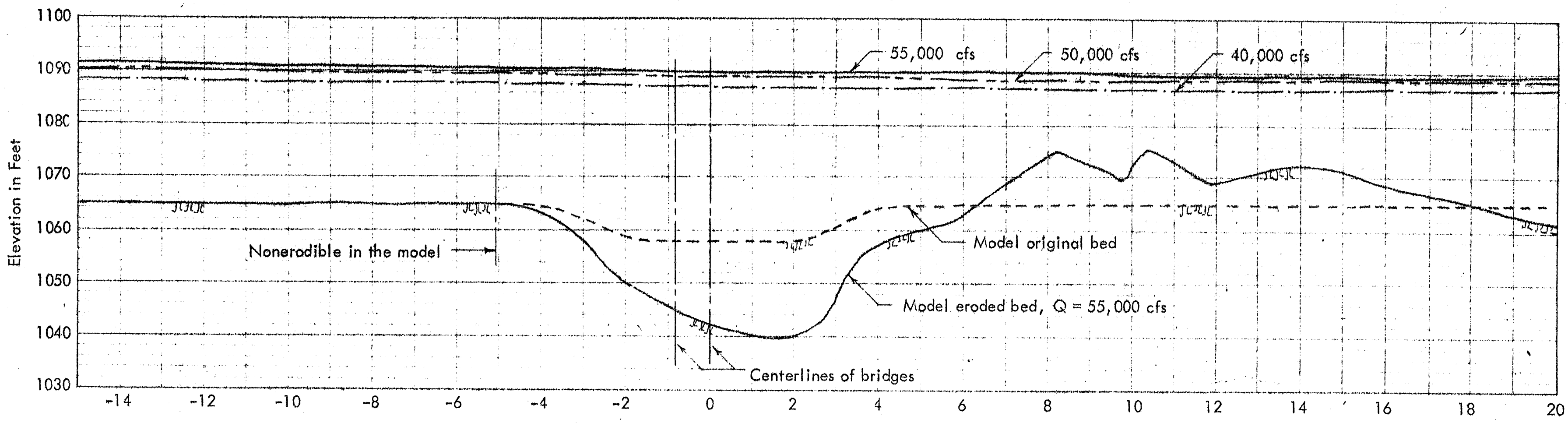
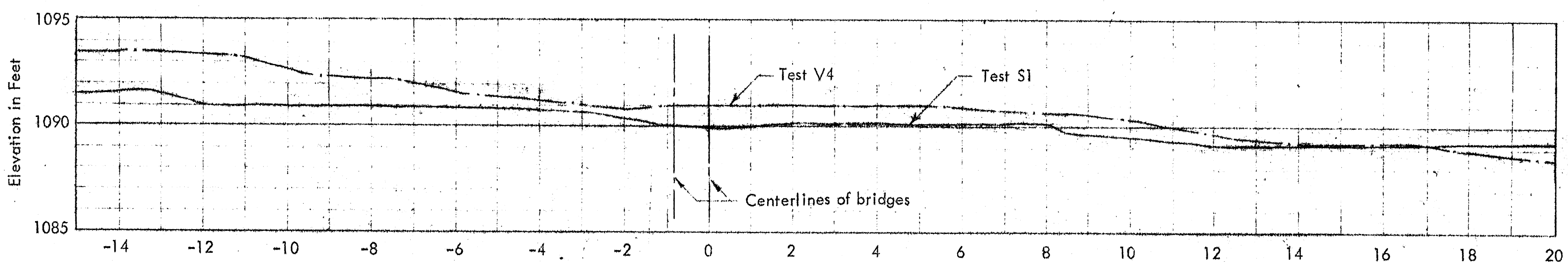
SCALE	DATE 9-10-64	NO. 142B442-34



LONGITUDINAL WATER SURFACE AND RIVER BED PROFILES



LONGITUDINAL WATER SURFACE PROFILES

Note: During Test S1, flows of 10,000, 20,000, 30,000, 40,000, 50,000, and 55,000 cfs were established (in increasing order of magnitude) and maintained until erosion ceased in the test section.

TEST S1
Stabilization Plan I
Longitudinal Water Surface and River Bed Profiles
Model Scale 1:75

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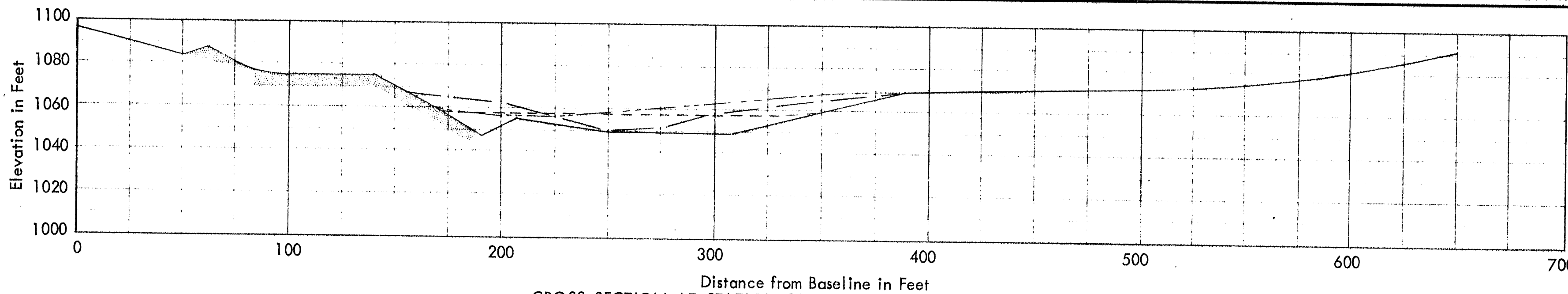
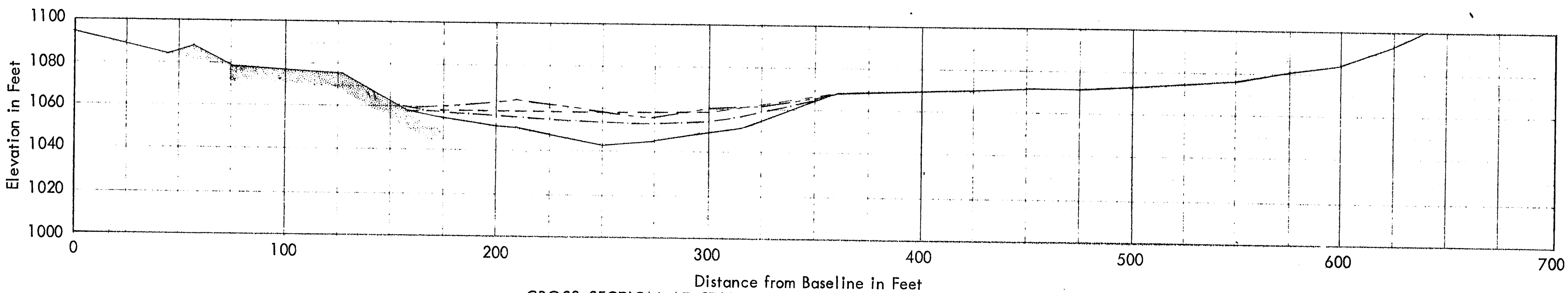
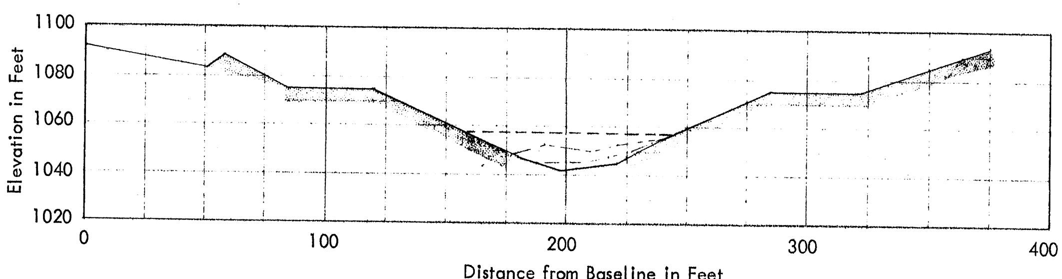


CROSS SECTION AT STATION 15 + 50 PERPENDICULAR TO BASELINE



CROSS SECTION AT STATION 15 + 00 PERPENDICULAR TO BASELINE



CROSS SECTION AT STATION 14 + 40 PARALLEL WITH BRIDGES

LEGEND:
- Original Bed
- Q = 40,000 cfs; Stage = 1086.9 ft
- Q = 50,000 cfs; Stage = 1089.2 ft
- Q = 55,000 cfs; Stage = 1090.2 ft

Revetment armor

Note: The locations of these cross sections are shown on Chart 142B442-33.

TEST S1
Stabilization Plan
Eroded River Bed Cross Sections
for Various Discharges
Model Scale 1:75

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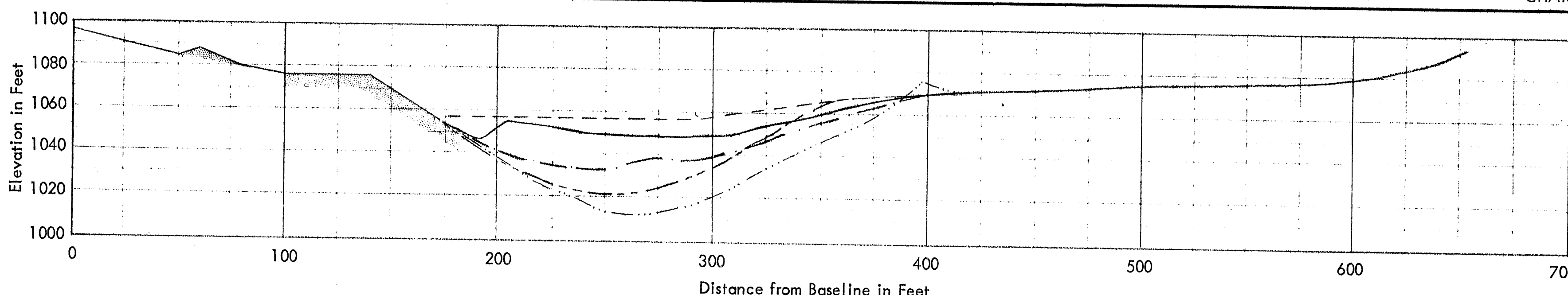
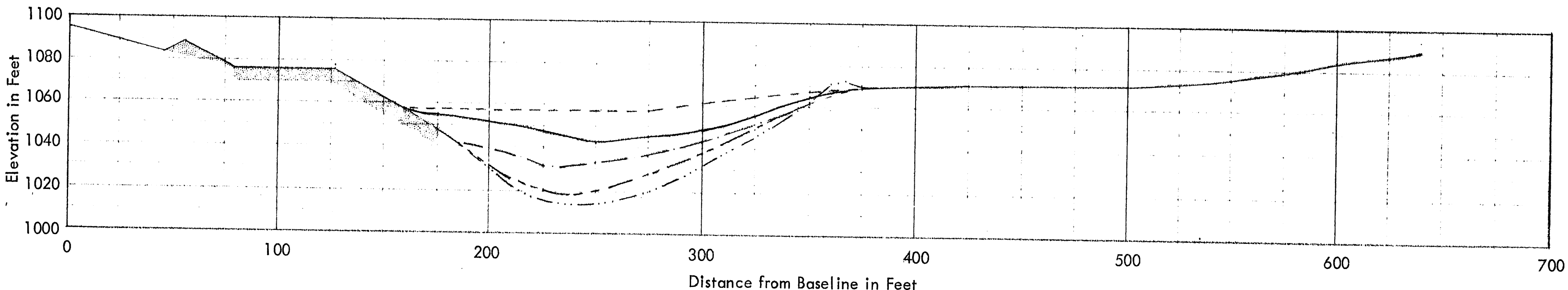
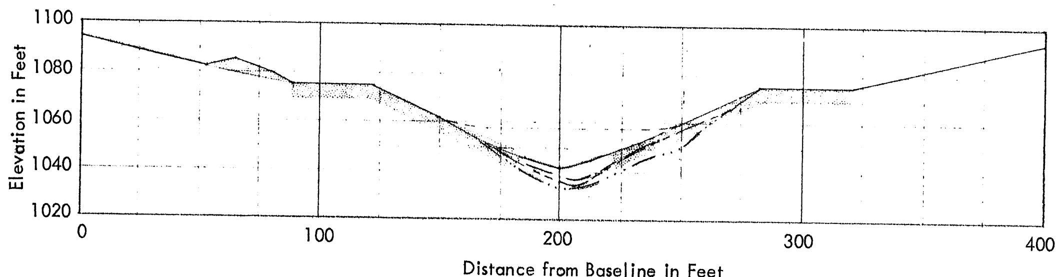
SCALE	DATE 9-23-64	NO. 142B442-38



CROSS SECTION AT STATION 15 + 50 PERPENDICULAR TO BASELINE



CROSS SECTION AT STATION 15 + 00 PERPENDICULAR TO BASELINE



CROSS SECTION AT STATION 14 + 40 PARALLEL WITH BRIDGES

TESTS S1, S2, S3, and S4
Stabilization Plan 1
Eroded River Bed Cross Sections
Q = 55,000 cfs
Various Stages
Model Scale 1:75

LEGEND:
- – – – – – Original bed
- ———— Test S1: Q = 55,000 cfs; Stage = 1090.2 ft
- — · — Test S2: Q = 55,000 cfs; Stage = 1088.7 ft
- – · · – Test S3: Q = 55,000 cfs; Stage = 1087.4 ft
- – · · · – Test S4: Q = 55,000 cfs; Stage = 1085.5 ft

Revetment armor

Note: The locations of these cross sections are shown on Chart 142B442-33.

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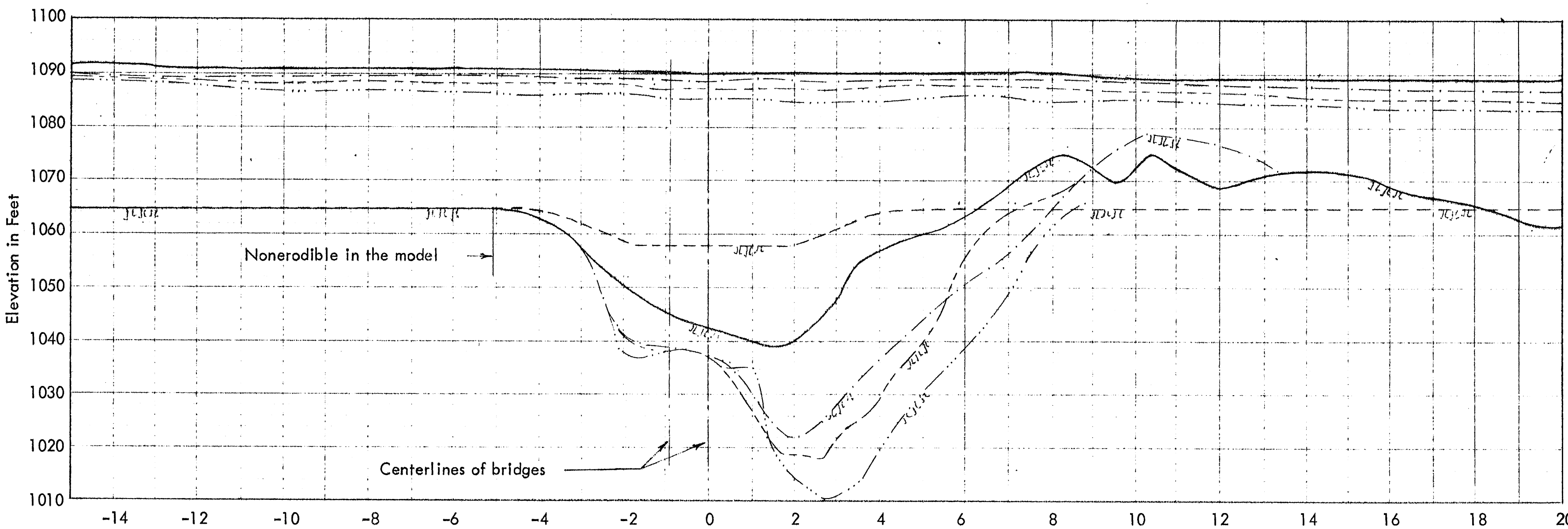

Stations in Hundreds of Feet from Centerline of Downstream Bridge

LONGITUDINAL WATER SURFACE AND RIVER BED PROFILES

LEGEND:
- — — — — — Original bed
- ———— Test S1: Q = 55,000 cfs; Stage = 1090.2 ft
- — · — · — Test S2: Q = 55,000 cfs; Stage = 1088.7 ft
- — - — - — Test S3: Q = 55,000 cfs; Stage = 1087.4 ft
- — · · · — Test S4: Q = 55,000 cfs; Stage = 1085.5 ft

TESTS S1, S2, S3, and S4
Stabilization Plan 1
Longitudinal Water Surface and River Bed Profiles
Q = 55,000 cfs
Various Stages
Model Scale 1:75

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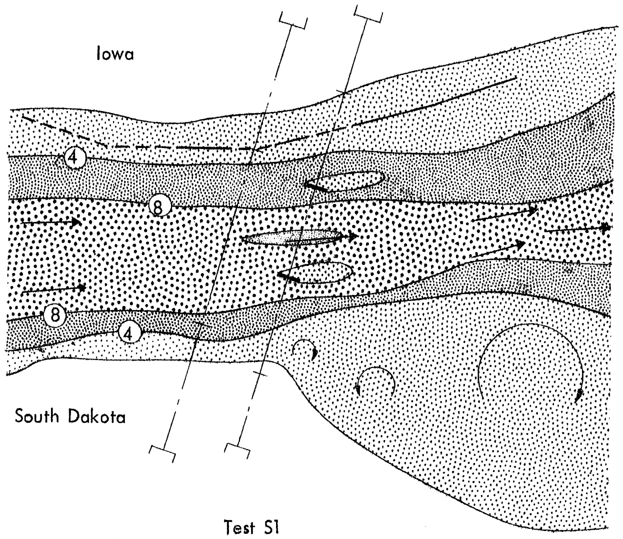
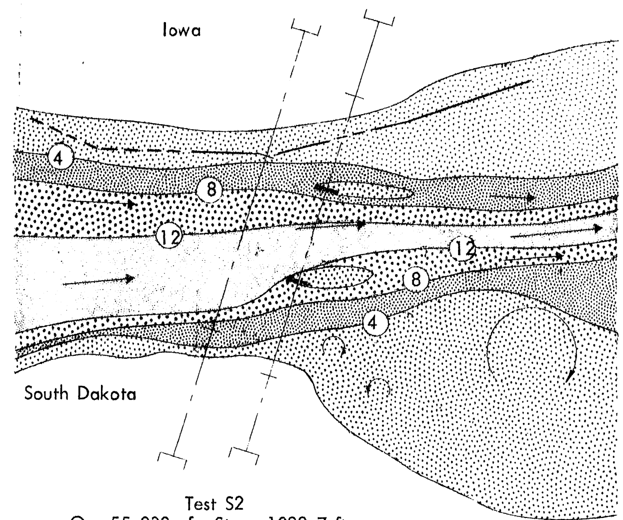
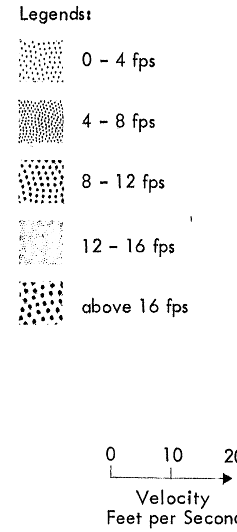
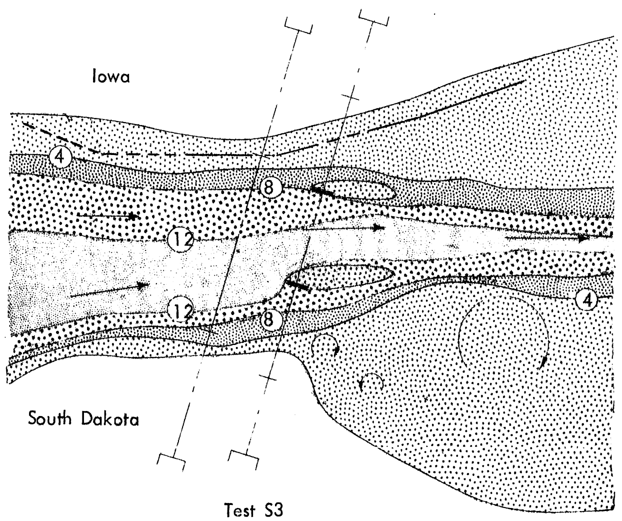


Test S1
Q = 55,000 cfs; Stage 1090.2 ft

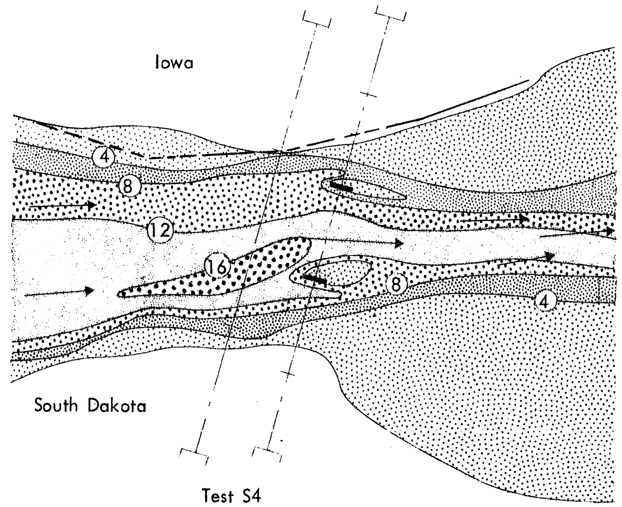


Test S2
Q = 55,000 cfs; Stage 1088.7 ft





Test S3
Q = 55,000 cfs; Stage 1087.4 ft



Test S4
Q = 55,000 cfs; Stage 1085.5 ft

TESTS S1, S2, S3, and S4
Surface Velocity Patterns
from Photographs
Q = 55,000 cfs
Model Scale 1:75

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CHANNEL STABILIZATION TESTS

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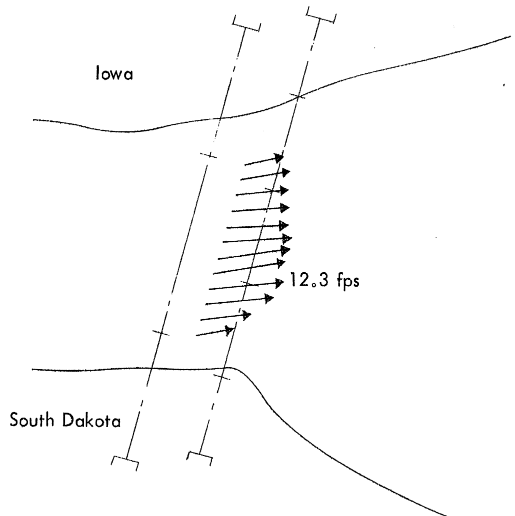
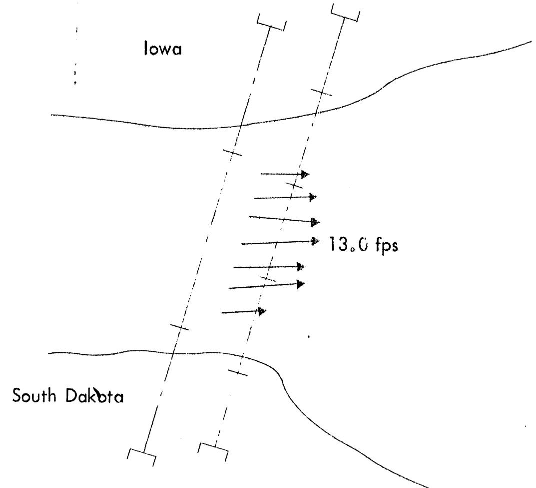
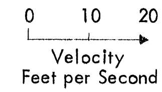
SCALE	DATE 10-7-64	NO. 142B442-44



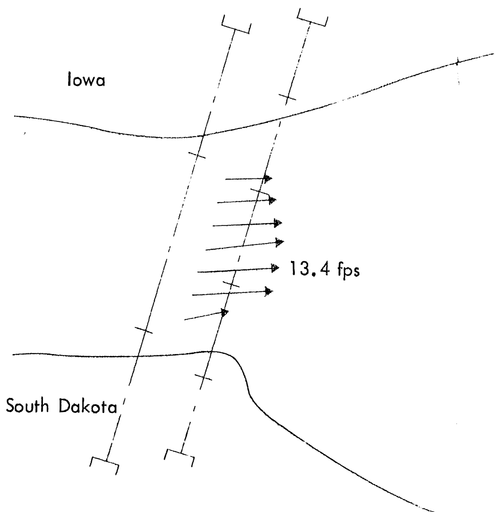
TEST S1
Q = 55,000 cfs; Stage = 1090.2 ft



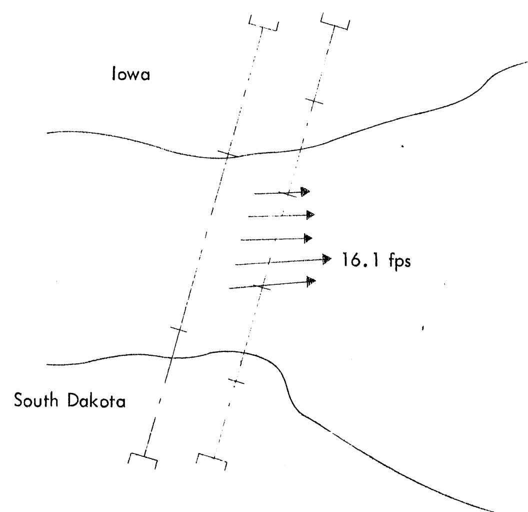
TEST S2
Q = 55,000 cfs; Stage = 1088.7 ft



Note: The velocities were measured with a propeller meter at a depth of 5 ft.



TEST S3
Q = 55,000 cfs; Stage = 1087.4 ft



TEST S4
Q = 55,000 cfs; Stage 1085.5 ft

TESTS S1, S2, S3, and S4
Velocity Distributions
Q = 55,000 cfs
Model Scale 1:75

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CHANNEL STABILIZATION TESTS

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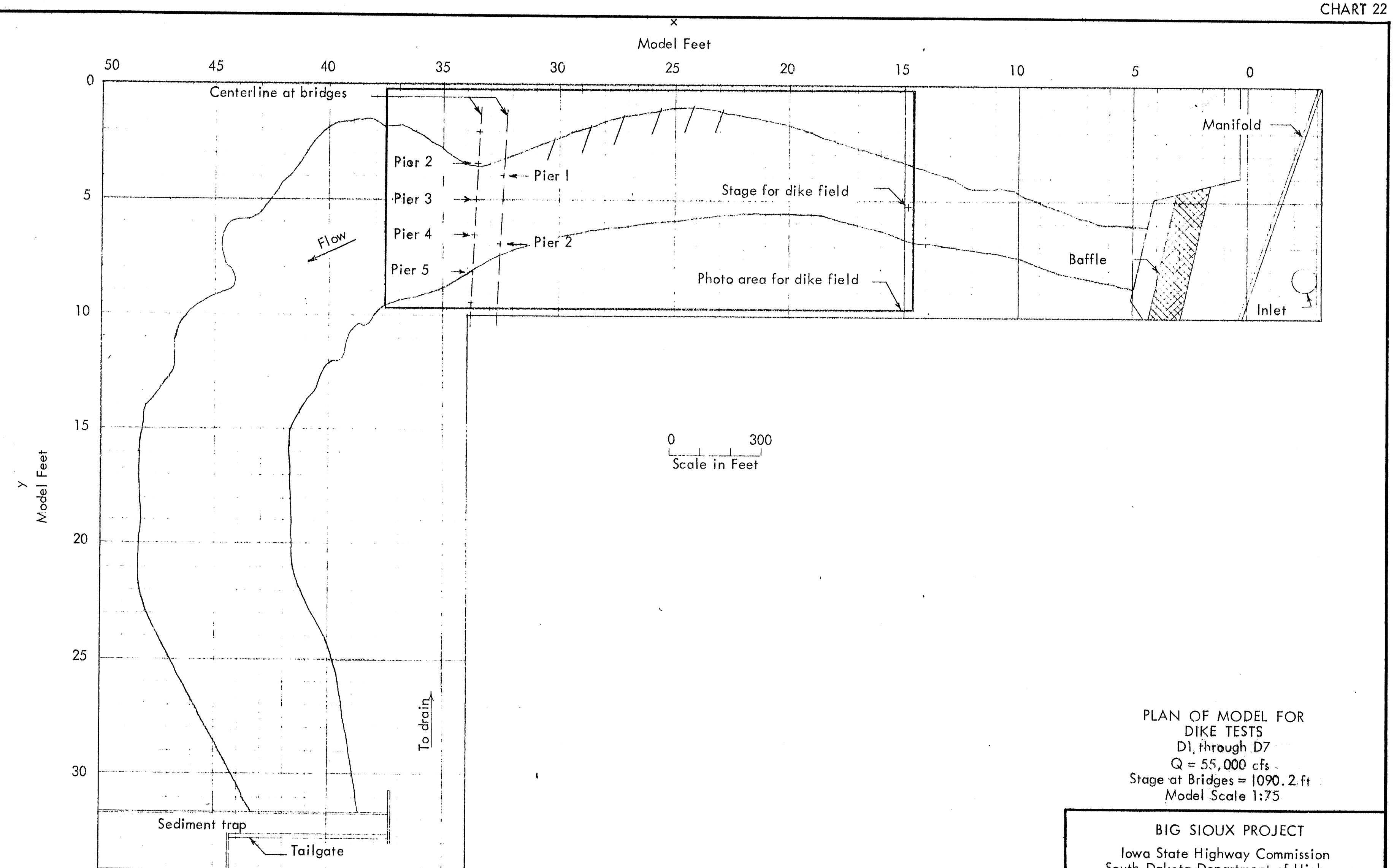
SCALE	DATE 10-7-64	NO. 142B442-45



PLAN OF MODEL FOR
DIKE TESTS
D1 through D7
Q = 55,000 cfs
Stage at Bridges = 1090.2 ft
Model Scale 1:75

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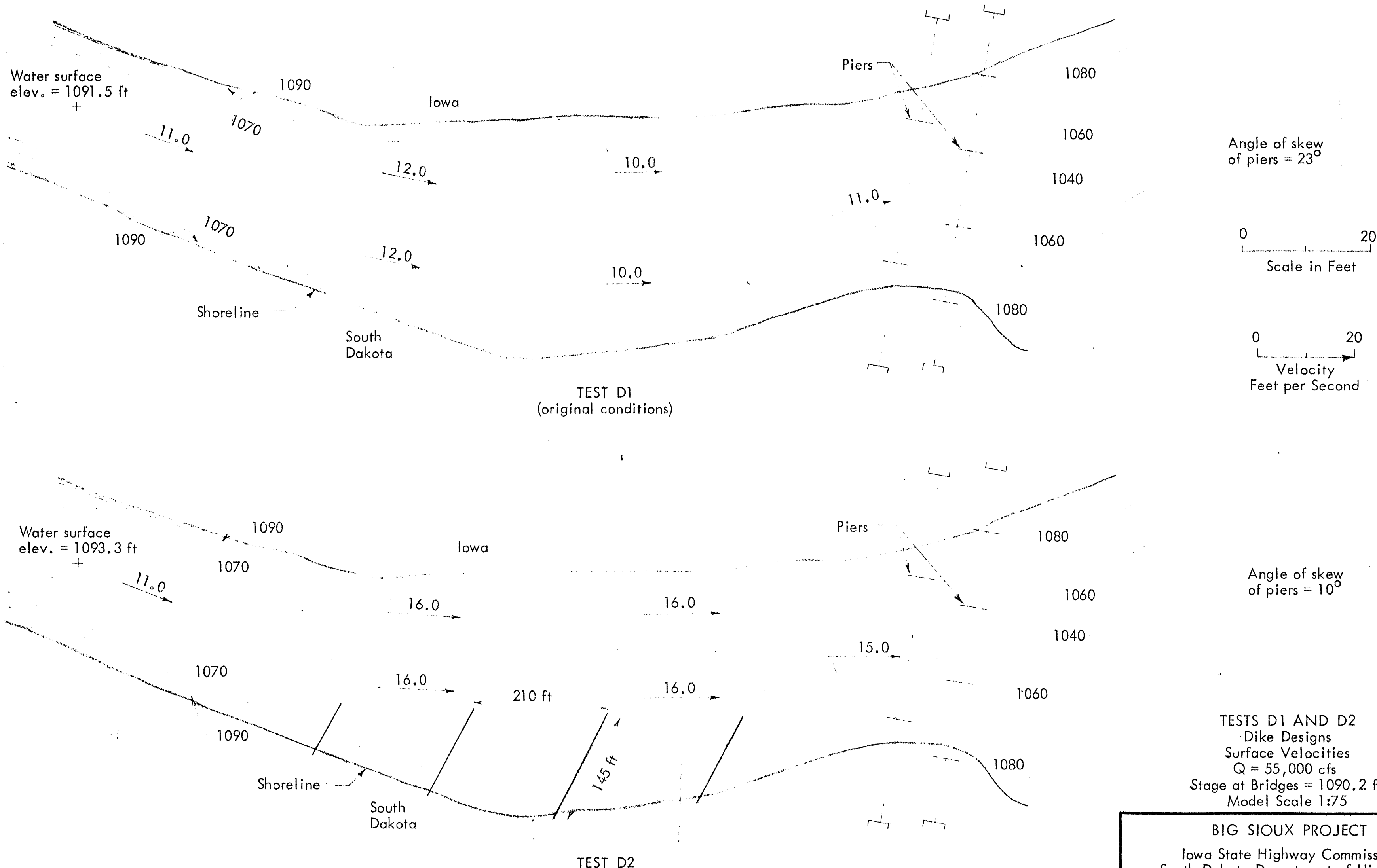

TESTS D1 AND D2
Dike Designs
Surface Velocities
Q = 55,000 cfs
Stage at Bridges = 1090.2 ft
Model Scale 1:75

Notes:
1. The contours delineate the eroded bed after test S1.
2. The arrows show surface velocities obtained from photos.
3. The upstream stage was measured 1400 feet upstream of the bridges.

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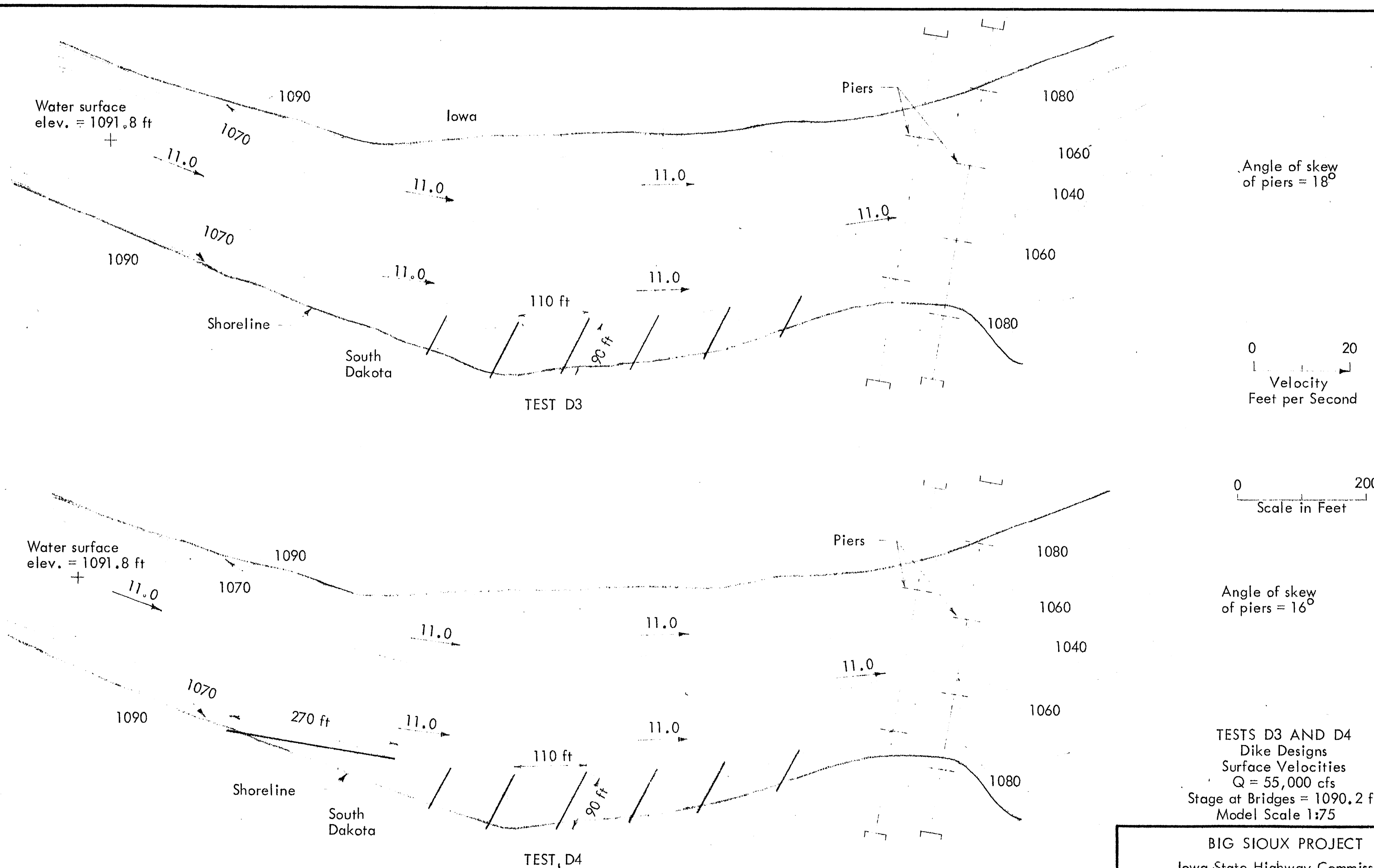

TESTS D3 AND D4
Dike Designs
Surface Velocities
Q = 55,000 cfs
Stage at Bridges = 1090.2 ft
Model Scale 1:75

Notes: 1. The contours delineate the eroded bed after test S1.
2. The arrows show surface velocities obtained from photos.
3. The upstream stage was measured 1400 feet upstream of the bridges.

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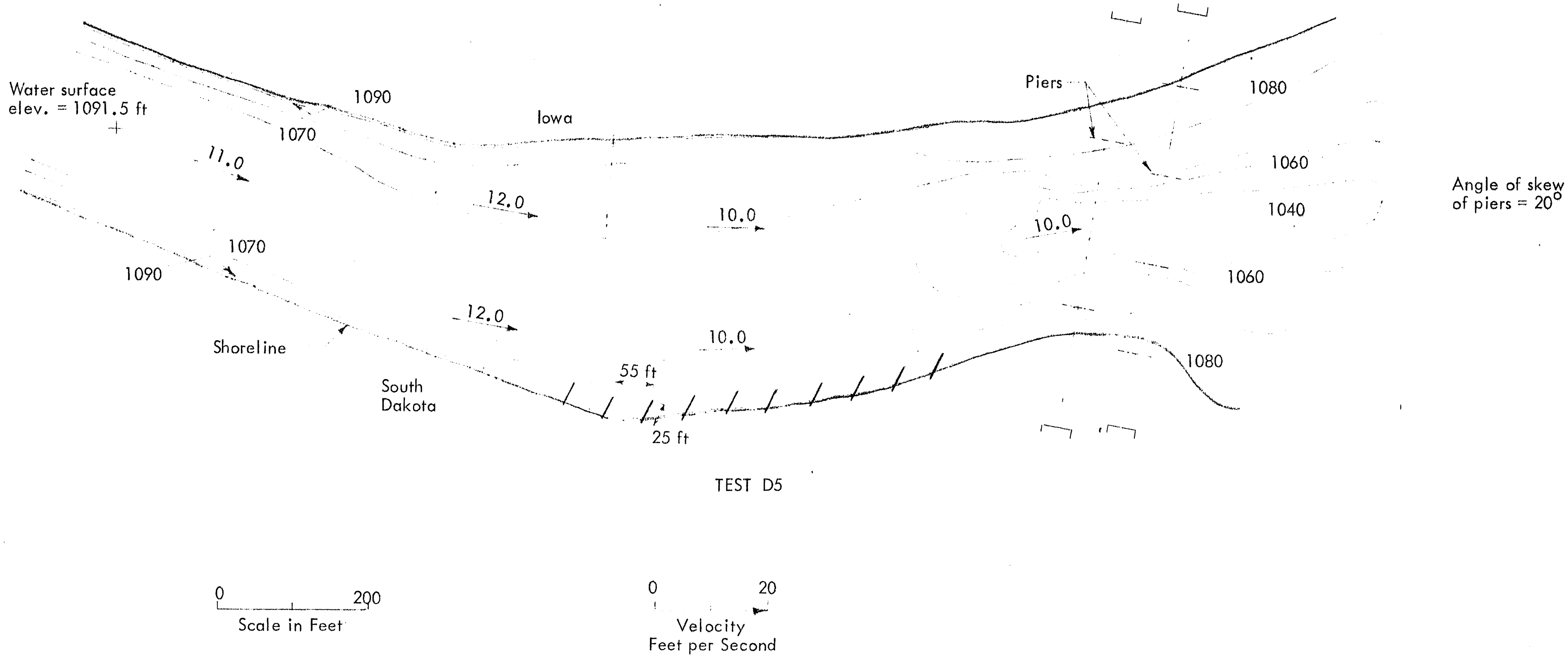

Notes: 1. The contours delineate the eroded bed after test S1.
2. The arrows show surface velocities obtained from photos.
3. The upstream stage was measured 1400 feet upstream of the bridges.

TEST D5
Dike Designs
Surface Velocities
Q = 55,000 cfs
Stage at Bridges = 1090.2 ft
Model Scale 1:75

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PERMEABLE DIKE TESTS

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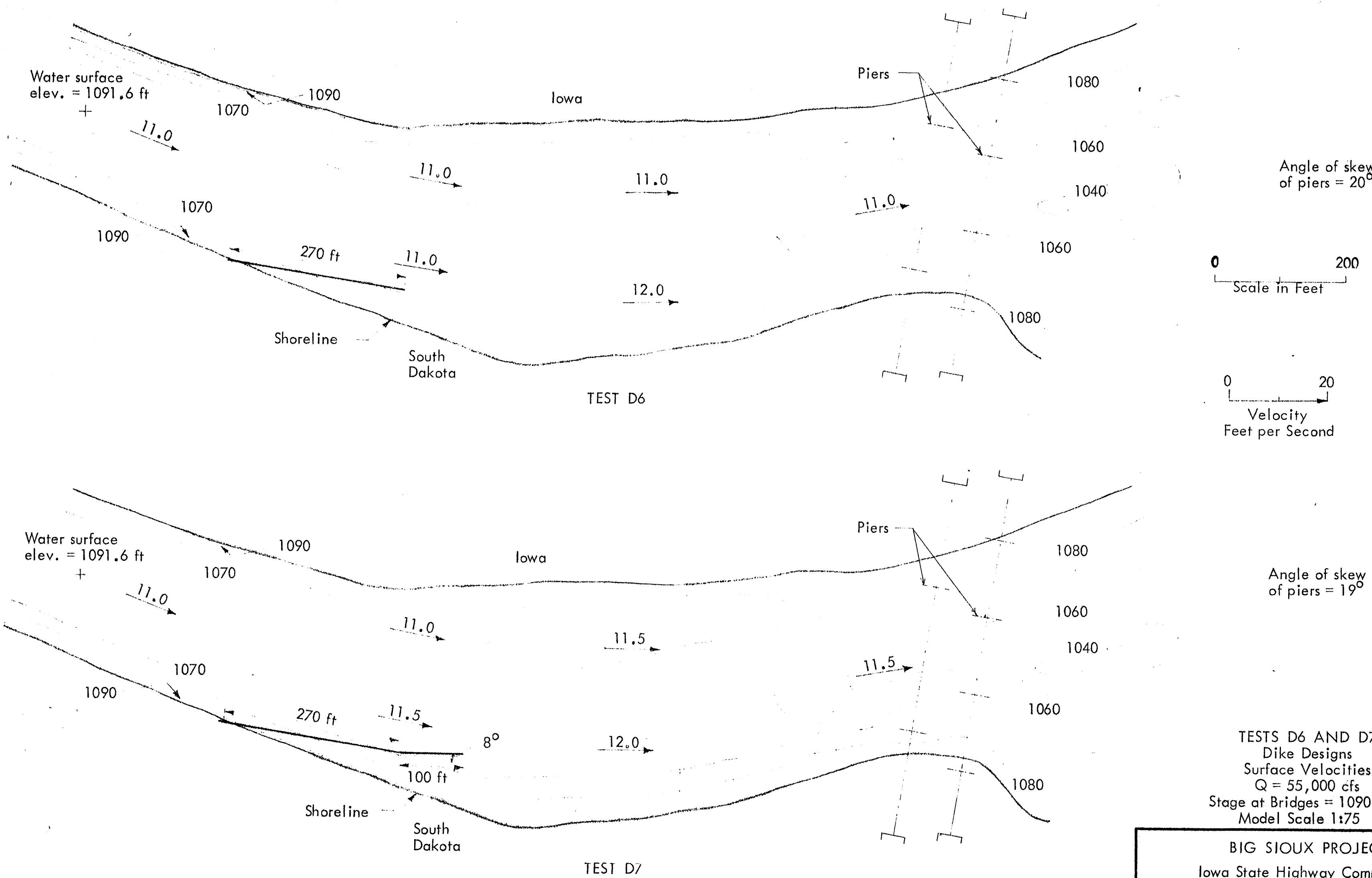

TESTS D6 AND D7
Dike Designs
Surface Velocities
Q = 55,000 cfs
Stage at Bridges = 1090.2 ft
Model Scale 1:75

Notes:
1. The contours delineate the eroded bed after test S1.
2. The arrows show surface velocities obtained from photos.
3. The upstream stage was measured 1400 feet upstream of the bridges.

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PERMEABLE DIKE TESTS

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DRAWN ALC	CHECKED	APPROVED
SCALE	DATE 11-9-64	NO. 142B442-50

Project Report No. 78

Report on

STUDIES OF THE STABILIZATION OF THE BIG SIOUX RIVER AT THE INTERSTATE 29 BRIDGE CROSSING

ADDENDUM 1.
EFFECT OF UPSTREAM DIKE FIELD ON FLOW PATTERN

by
Alvin G. Anderson

Prepared for
South Dakota Department of Highways
and the
Iowa State Highway Commission

December 1965

CONTENTS

Report on

STUDIES OF THE STABILIZATION OF THE BIG SIOUX RIVER AT THE INTERSTATE 29 BRIDGE CROSSING

ADDENDUM 1. EFFECT OF UPSTREAM DIKE FIELD ON FLOW PATTERN

INTRODUCTION

The original report on the stabilization of the Big Sioux River at the Interstate I-29 bridge crossing was concerned primarily with the stabilization of the river bank in the neighborhood of the bridges and the protection of the bridge piers. It also described some exploratory experiments on the effect of dikes on the pattern of flow approaching the bridges. Since that time additional studies to establish optimum location, spacing, and length have been made and are the subject of this addendum.

The bridges carrying I-29 across the Big Sioux River are located about 1.5 miles above the confluence of the Big Sioux with the Missouri River near Sioux City, Iowa. Just downstream of the crossing the river makes a sharp turn toward the left. The river crossing consisted of two parallel bridges which crossed the river at an angle to the main flow in the channel. They were supported on web piers spaced 120 ft apart so that the two center piers of each bridge were founded in the river bottom. On April 1, 1962, during the flood on the Big Sioux River the upstream bridge of the pair collapsed because pier 3 was undermined. This in turn caused the rupture of the adjacent spans and the consequent failure of pier 4. An hydraulic characteristic of the failure aside from the undermining of the piers was the development of a very deep and extensive scourhole downstream of the structure and the later subsidence of the left bank opposite the large scourhole. The upstream bridge was replaced with another bridge having a longer central span supported upon cylindrical piers, while the downstream bridge was retained but strengthened with additional underpinning in the form of longer steel piles. The initial studies were undertaken to study the effect on the flow pattern of the proposed works to protect the bridge piers and to stabilize the banks to prevent further subsidence.

Although the earlier studies indicated that the bridge structure would probably be safe from damage due to future floods, the alignment of the river was unchanged and the piers of the downstream bridge were still at an appreciable angle of attack to the flow. It was considered worthwhile by the sponsor that in the course of the research a study be included on the use of dikes to change the channel alignment and thus improve the flow pattern in the bridge section. The objectives of this second study were to provide for better alignment of the flow through the bridge sections, to protect the west bank from erosion, and to observe the effect of the dikes on the erosion of the east bank.

BRIEF DESCRIPTION OF THE MODEL

The original model was used for the present studies. It consisted of a movable bed model encompassing about a mile of the river centered on the highway bridges and constructed to a scale 1:75. A portion of the model in the neighborhood of the bridges is shown in Photo 1. Chart 1 shows the relationship between the model baseline used to determine locations with the baseline used in the field for prototype construction. A revision of the west bank over a reach from approximately 1000 ft to 1800 ft upstream of the bridges was made to correct the model alignment in accordance with field data obtained in 1964, which were more extensive than the previous field data used for the original design. In the region of the bridges the model was molded with riprap to conform to the protective geometry described in the previous report. The model bed consisted of sand having a mean diameter of approximately 0.18 mm, which was then covered with a fine gravel in order to prevent excessive erosion of the sand and to simulate qualitatively the cohesive characteristics of the prototype bed material.

The training dikes used for this study were made of expanded metal with a porosity of approximately 65 per cent. A rectangular cell was made of two of these sheets and filled with a fibrous material to further decrease the porosity to approximately 20 per cent. In the experiments these model dikes of different lengths were positioned at various selected points perpendicular to the flow.

PRELIMINARY TESTS OF MODEL WITHOUT DIKES

Before any experiments involving dikes were made the model was operated at a discharge corresponding to 55,000 cfs for approximately six prototype days in order to allow the newly molded model to become stabilized. The flow pattern developed in the river for this discharge is shown by means of the confetti streaks in Photo 2. The photograph shows the long curve of the river from the footbridge to the bridges of I-29, a distance of approximately 1/2 mile. The photo also shows the natural alignment of the downstream bridge piers to the flow. The angle between the streamlines and the pier is approximately 19 degrees. For this test with a discharge of 55,000 cfs the water surface elevation at the bridge section was maintained at 1090.2 ft. At the upstream end of the model in the neighborhood of the footbridge the water surface elevation was 1092 ft.

The bed geometry as developed by this flow for a period equivalent to six days in the prototype is shown in Photo 3. Contour lines have been drawn to indicate bed elevations. Although the bed in this region had been covered with a fine gravel to prevent erosion of the underlying sand, some leaching did occur and sand was transported along the bed near the east bank of the river. This streak of sand during one stage of its motion can be seen on the bed in Photo 3. The sand leached through the gravel moved freely in regions where the velocity was relatively high and tended to be deposited after it entered areas in which the velocity was lower.

During these preliminary experiments velocity measurements were made at different crest sections using a cup type current meter. These velocities in conjunction with photographs showing the confetti streaks on the surface provided information to establish the velocity vectors at various points in the model. The transverse velocity profiles based upon these measurements and showing both direction and magnitude are given in Chart 2. This chart also shows the change in direction of the current as it flows along the curve in this reach.

DIKE EXPERIMENTS

The experiments to determine dike position and length consisted of the installation of successive dikes beginning at the upstream end near the footbridge and observation of the resultant flow pattern and velocity distribution. The location and length of dike were determined by successive trials until it appeared that the flow pattern represented the optimum for the conditions to be satisfied. The factors to be considered in this instance were the degree of alignment improvement and the increased velocity due to channel constriction. The tests will be described consecutively as successive dikes were added to the channel.

Test D-8, Dike 2B

In the first test a dike approximately 75 ft long was placed at station 15. This location was based on visual observations of the flow pattern generated by dikes of different lengths and placed at various locations. It appeared that a dike at station 15 would be more effective in altering the flow pattern and the direction of the currents entering the test reach. A photograph of the flow pattern for this arrangement is shown in Photo 4. The photo shows that a low velocity region was created both upstream and downstream of the dike. On the upstream side the dike causes flow separation from the west bank at a point about 150 ft upstream of the dike. Downstream the flow velocity was decreased to about 5 ft per second or less for a distance of approximately 500 ft. The transverse extent of the flow velocity region is approximately equal to the length of the dike. This is shown in Chart 3 for stations 15 and 17.5. At station 15 the transverse velocity profile is deformed somewhat in the region of the dike, while at station 17.5 the dike has pushed the velocity profile somewhat more towards the center of the stream and appreciably decreased the velocity in the neighborhood between the dikes.

Test D-9, Dikes 2 and 2B

One objective of the study was to place the dikes in such position that natural excavation of the east bank would occur and thereby correct the stream alignment at the bridge sections. To do this, excavation would have to begin approximately at station 15 where an appreciable separation zone (see Photo 2) occurs at the east bank for the uninhibited flow conditions. The velocity profiles at station 15 (Chart 3) and the dike at station 15 had little effect on the flow velocity or flow pattern at the east bank. In an effort to improve this situation dike 2 was placed upstream approximately at station 12.5. This additional dike effectively shifted the velocity profile to the left at stations 15 and 17.5. (The transverse profile generated by dikes 2 and 2B is designated as a triangle in Chart 3.) The flow pattern generated by this system of dikes is shown in Photo 5.

Test D-10, Dikes 2 and 3

The distance between dikes 2 and 2B was about 175 ft. In order to have as few dikes as possible, the dike at station 15 (2B) was moved downstream so the spacing would be about 300 ft, or double the length of the deadwater zone noticed above the dikes in Photos 4 and 5. Several different spacings were attempted but 300 ft seemed to be the most satisfactory, giving a low velocity region between the dikes which was relatively free of eddies and which would probably induce deposition. This flow pattern is shown in Photo 6 where the low velocities are indicated by the slight movements of the confetti particles on the surface. The pattern of flow within the dike intervals exhibited a regular movement pattern somewhat like a figure eight.

The transverse velocity profiles generated by the presence of dikes 2 and 3 are shown at successive stations moving downstream in Charts 3, 4, and 5. The effect of dike 3 at station 17 was to create a wider, low velocity zone on the right or west side of the river within the area of the dikes and downstream some 600 ft. The shift in the velocity profile toward the left or east bank was noticeable only for about 300 ft (to station 25) as shown in Chart 5. This shift tended to reduce the thickness of the separation zone along the left bank beginning at station 16.

Test D-11, Dikes 2, 3, and 4

Although dikes 2 and 3 placed in the previous tests had created a low velocity zone near the right bank down to station 25 that would probably protect the bank from erosion, the velocity along the left bank was not appreciably increased beyond station 22. Dike 4 was therefore placed at station 21 in order to extend the quiet water zone along the right bank and to deflect the current towards the left bank, thus reducing the degree of separation and increasing the velocity along the bank. The placement of dike 4 did increase the low velocity region along the right bank down to station

30 and did shift the transverse velocity profile towards the left bank so that the higher velocity now extended downstream approximately to station 27. This shift in transverse velocity profile is clearly shown in Chart 5. Chart 6 shows that at station 27.5 the shift towards the left is negligible, and at station 30 it appears that the transverse profile has actually drifted towards the right bank. The shift in the transverse profile towards the left bank was sufficient to move some of the sand previously deposited in this area, and the increased velocity due to the constriction was sufficient to start the movement of some of the gravel particles in the armor layer over the sand. This erosion can be seen in Photo 7 where some of the particles colored black and placed in transverse strips on the gravel bed have been removed from the area near the centerline of the river.

Test D-12, Dikes 2, 3, 4, and 5

The results of the previous experiments indicated that the region of streamflow adjustment moved downstream as additional dikes were placed and that the effect of dike 4 was not evident past station 30. In order to continue the progression of dikes, dike 5 was placed at station 25. The positioning of the successive dikes was governed by the developing transverse velocity profile and the general flow pattern so that the spacing was the maximum capable of continuing the adjustment in velocity profiles and patterns. With the placement of dike 5 at station 25 the low velocity zone along the right bank now extended down to the bridge and along the east bank. The high velocity flow was shifted enough to have an even distribution as the flow entered that portion of the channel protected by riprap just upstream of the bridge. This was a desirable condition because then the heavy armoring revetment, particularly on the right or west side of the river, would not be subjected to attack by the high velocities that previously existed along this bank. Photo 8 is a view of the flow pattern generated by the installation of dike 5 and shows that the region of quiet water behind dike 5 has extended down to the bridge sections. Further, the introduction of dike 5 has changed the angle of the streamlines relative to the pier alignment from 19 degrees to approximately 13 degrees. Because of the constriction, the velocity in this region has increased to approximately 16 ft per second for this discharge. The transverse velocity profile generated by the placement of dike 5 in addition to the previous dikes is shown in Chart 6. This chart shows that the transverse velocity profile has been very appreciably shifted towards the left bank in the regions near stations 27.5 and 30. The profile at station 32 shows that at the bridge sections the velocity profile is symmetrical with the channel cross section.

Test D-13, Dikes 2, 3, 4, 5, and 6

An additional dike, dike 6, was placed at station 29 to determine if any further improvement could be obtained by such additional dikes. From Chart 6 the transverse velocity profile appears to have been shifted too far towards the left bank and probably would be erosive to the Iowa side of the channel as it passes into the bridge sections. Photo 9 shows the rather excessive deflection of the flow caused by the presence of dike 6 so that the zone of quiet water downstream of dike 6 has become considerably enlarged. It appears that benefits obtained by the use of dike 6 in this area are inappreciable.

Influence of Dike Field on Erosion of Sediment

Upon the completion of the experiments to establish the dike field based upon transverse velocity patterns, a test was made to qualitatively examine the erosion and transport of bed sediment. Since the bed of the Big Sioux River contained a high clay-silt content with a relatively high cohesiveness, it is not possible to duplicate it in the model. In order to delineate the erosion pattern a fine sand was used to mold part of the model in the area near the left bank. This area is now subjected to greater erosive forces because of the presence of the dikes on the right bank. With dikes 2, 3, 4, 5, and 6 in place a discharge of 10,000 cfs was run through the model. There was no apparent movement of the sand at this discharge so the flow was increased to 20,000 cfs. The velocities created by this discharge caused an immediate movement of the sand as bedload. The transport of sediment was rather general so that detailed geometry of the erosion pattern could not be established. For this discharge much of the sand was finally deposited in the deeper channel just upstream of the bridges. Photo 9 is an overall view of the flow pattern upstream of the bridges with the dikes in place for a discharge of 20,000 cfs. Seen through the water is the fine sand bed, the material of which is being transported down the river towards the bridge section. The photograph shows the relatively quiet water and the flow pattern between the dikes on the left bank.

Miscellaneous Tests on the Dikes

An experiment was made to observe the effect of length of dike on immediate flow pattern. The length of the dike as finally established for the test was determined by the characteristics of the transverse velocity profiles and the magnitudes of the velocities. This was done by trial until the flow patterns seemed to be the most appropriate for the situation. In order to compare and determine the effect of length of a dike on the pattern, dike 2 was lengthened by 10 ft and velocity profiles were taken upstream and downstream of the dike. From these observations it appeared that there was a negligible upstream effect while downstream the velocity profile was shifted to the left in proportion to the increase in length of the dike. Photo 11 shows the flow pattern as generated by this dike and the only apparent effect is to further constrict the flow in this region.

An additional test was also run to determine if an additional dike upstream of dike 2 would be needed or useful. Dike 1 was set at station 9 above the footbridge. Chart 4 shows the velocity patterns caused by this dike. The only appreciable effect on the velocity was to shift the transverse profile at station 10 away from the right bank without appreciably shifting it towards the left bank. From this it might be concluded that the benefits of dike 1 on the flow pattern in the system were negligible.

A final test was made of the effect of the angle of the dike on the streamlines of the flow pattern. This test was strictly exploratory and was not sufficiently definitive to provide useful data on the influence of dike alignment different from the perpendicular. From the velocity profiles and the flow pattern there was no appreciable improvement in this instance. Further study of this aspect of dike systems would be useful.

CONCLUSIONS

The results of the experiments to determine optimum spacing and length of dikes along the right bank of the river are given in Chart 7 which shows the positions of the proposed dikes and the transverse velocity profiles generated by these dikes. It is apparent from the chart that the high velocities have been shifted towards the left bank and that an appreciable region of quiet water has been generated along the right bank.

ACKNOWLEDGEMENT

The expert assistance of Mr. Roy M. Kuha, engineer, in the performance of these experiments and the preparation of charts and photographs is acknowledged.

LIST OF PHOTOS

PHOTO 8 (Serial No. 142-145A) Test D-12. In this test dike 5 at station 25 has been added to the dike field. The relatively quiet water on the right bank has been extended down to the bridges by the addition of dike 5. The angle of the streamlines relative to the pier alignment has been reduced to 13 degrees from 19 degrees that existed prior to the placement of dike 5. The flow has been shifted towards the left bank and the velocity has been increased. The high velocity flow now enters the section of the bridges more symmetrically.

PHOTO 9 (Serial No. 142-149) Test D-13. In order to determine whether another dike between dike 5 and the bridge would be helpful to the flow pattern, dike 6 has been added. The resulting flow pattern is shown. The effect of the dike was to increase the transverse depth of the area of quiet water and to shift the transverse velocity profile towards the left bank. The beneficial effects of dike 6 were negligible and in fact may be somewhat detrimental.

PHOTO 10 (Serial No. 142-158) This mosaic shows the character of erosion along the left bank when the gravel has been replaced by fine sand. The discharge is 20,000 cfs but the increased velocity along the left bank is conducive to considerable erosion.

PHOTO 11 (Serial No. 142-161) Test D-18. This test was to determine the effectiveness of an added length of the dike on the realignment of the flow pattern. Dike 2 has been increased 10 ft in length. The added length constricts the flow and increases the transverse depth of the quiet zone but had little effect on the reduction of the separation zone along the left bank. The shift in the transverse velocity profile appeared to be directly proportional to the increase in length of the dike. Q = 55,000 cfs.

PHOTO 1 (Serial No. 142-112) The model of the Big Sioux River in the neighborhood of Interstate I-29 bridge crossing consisted of a stretch of about one mile centered at the bridges and included the downstream bend. The model had a movable bed with a gravel armor layer to simulate the cohesiveness of the prototype sediment. The confetti streaks illustrate the flow pattern.

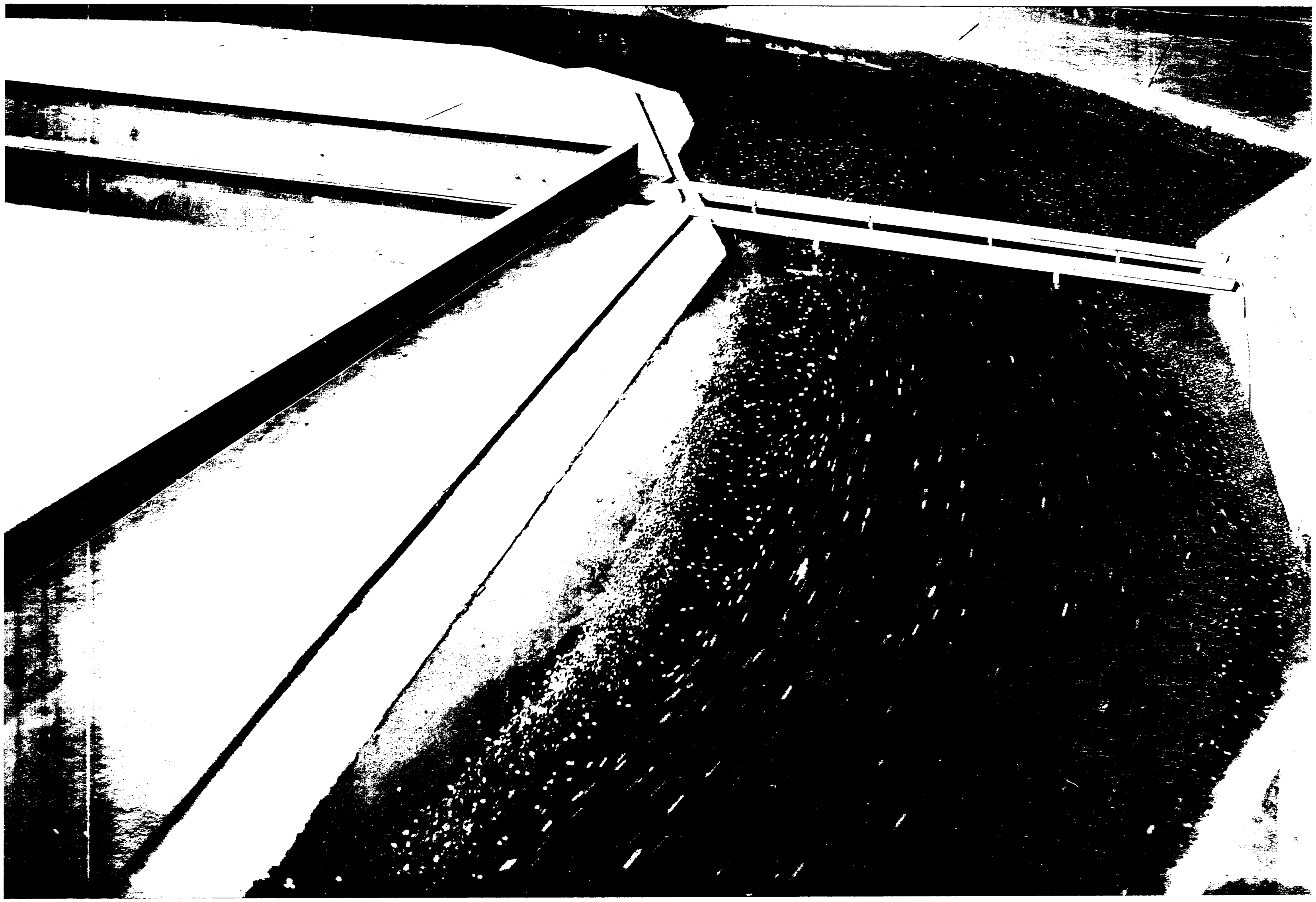


Photo 1

PHOTO 2 (Serial No. 142-121) This photograph is a mosaic of the portion of the model upstream of the bridges. It shows the flow pattern for discharge of 55,000 cfs prior to the installation of the experimental dikes. The flow patterns resulting from the dike experiments should be compared with this photo to illustrate the effects of the dikes. The confetti streaks indicate both the velocity and direction of the streamline. The angle of attack of the bridge piers with the flow is shown in this photo.

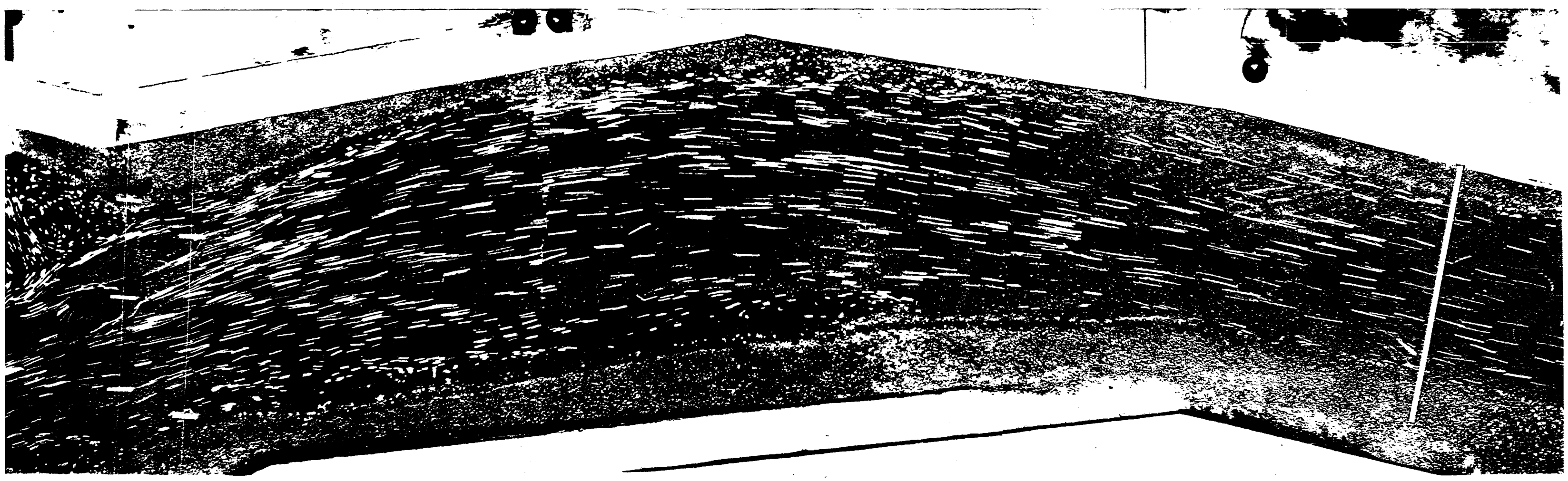


Photo 2

PHOTO 3 (Serial No. 142-126) This photograph is a mosaic of the bed in the upstream portion of the model prior to the installation of dikes. It shows the fine sand in motion and the bed contours which have been drawn on the photograph. Fine gravel was placed on the bed to prevent the erosion of the sediment and to simulate qualitatively cohesive prototype bed sediment.

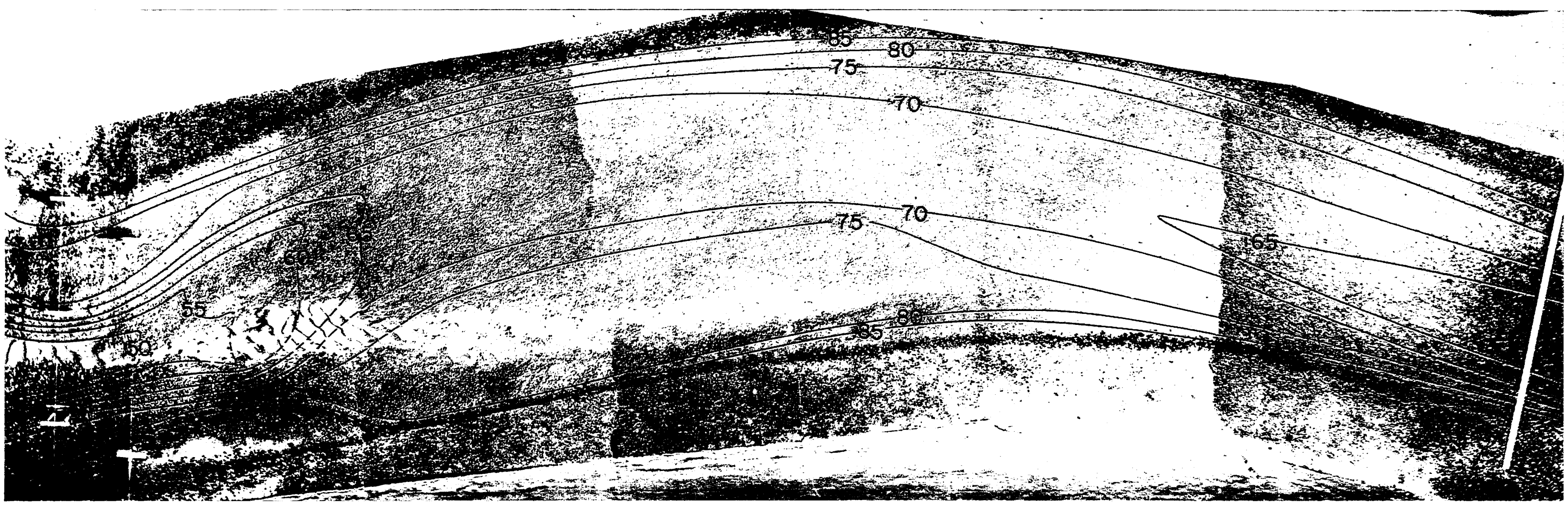


Photo 3

PHOTO 4 (Serial No. 142-134) Test D-8. Dike 2B has been placed at station 15 downstream of the footbridge. The dike was 75 ft long. The flow pattern is shown by confetti streaks. The photo shows the relatively quiet water downstream of the dike and the deflection of the main flow away from the right bank.

PHOTO 5 (Serial No. 142-136) Test D-9. Dike 2 has been added upstream of the dike shown in Photo 4 in order to extend the zone of quiet water and to reduce the width of the separation zone along the left bank.

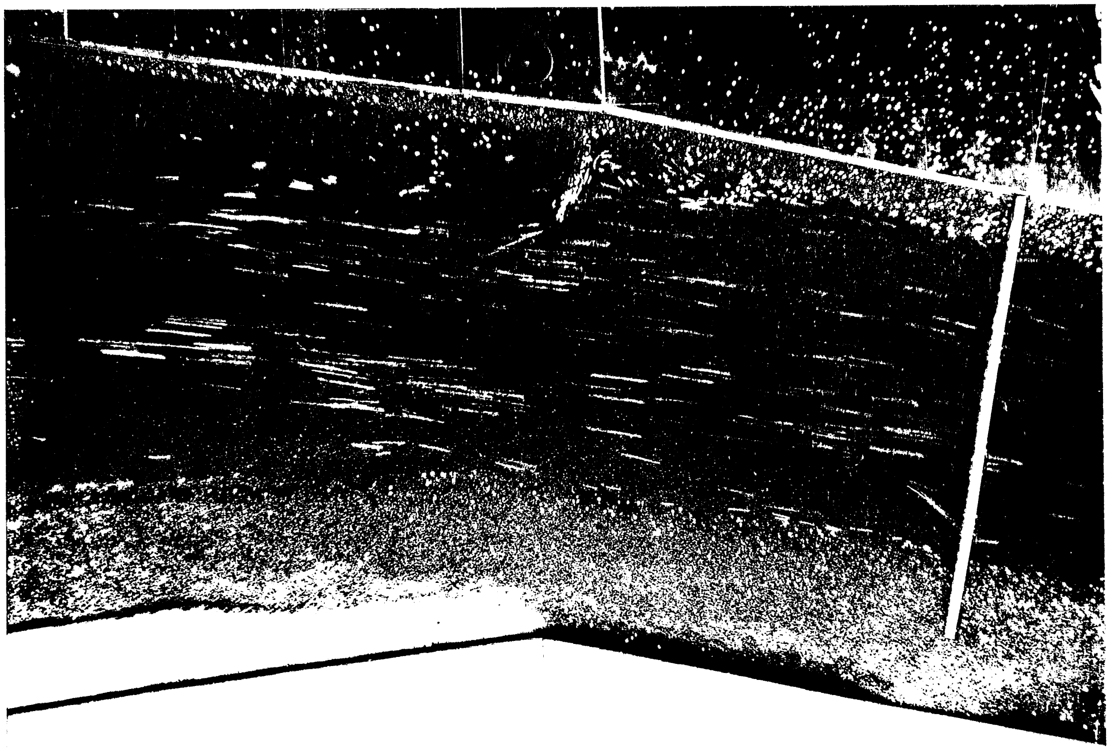


Photo 4

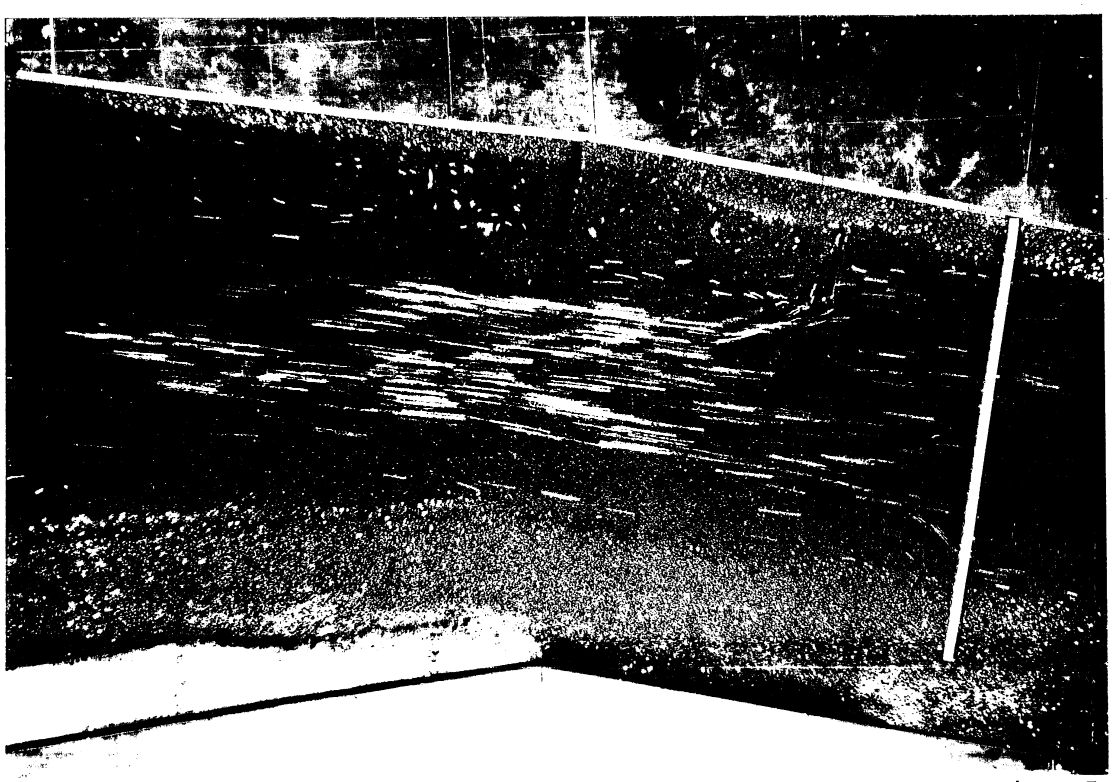


Photo 5

PHOTO 6 (Serial No. 142-138) Test D-10. Dike 2B has been removed and dike 3 has been added for this test. The shift downstream was made in order to observe the effect of an increase in spacing of the flow pattern with the hope of reducing the number of dikes necessary. The increased spacing was effective in increasing the zone of quiet water within which a relatively regular slow movement somewhat like a figure eight developed.

PHOTO 7 (Serial No. 142-143) Test D-11. Dike 4 has been added downstream of station 20 in order to extend the zone of quiet water. The use of dike 4 has constricted the flow between the dikes and the left bank and increased the velocity. The black streaks in the photograph are strips of colored gravel particles placed in this area in order to observe the erosion of the gravel caused by the increased velocity. Some of the particles on the strip downstream of dike 4 have been removed.

Photo 6

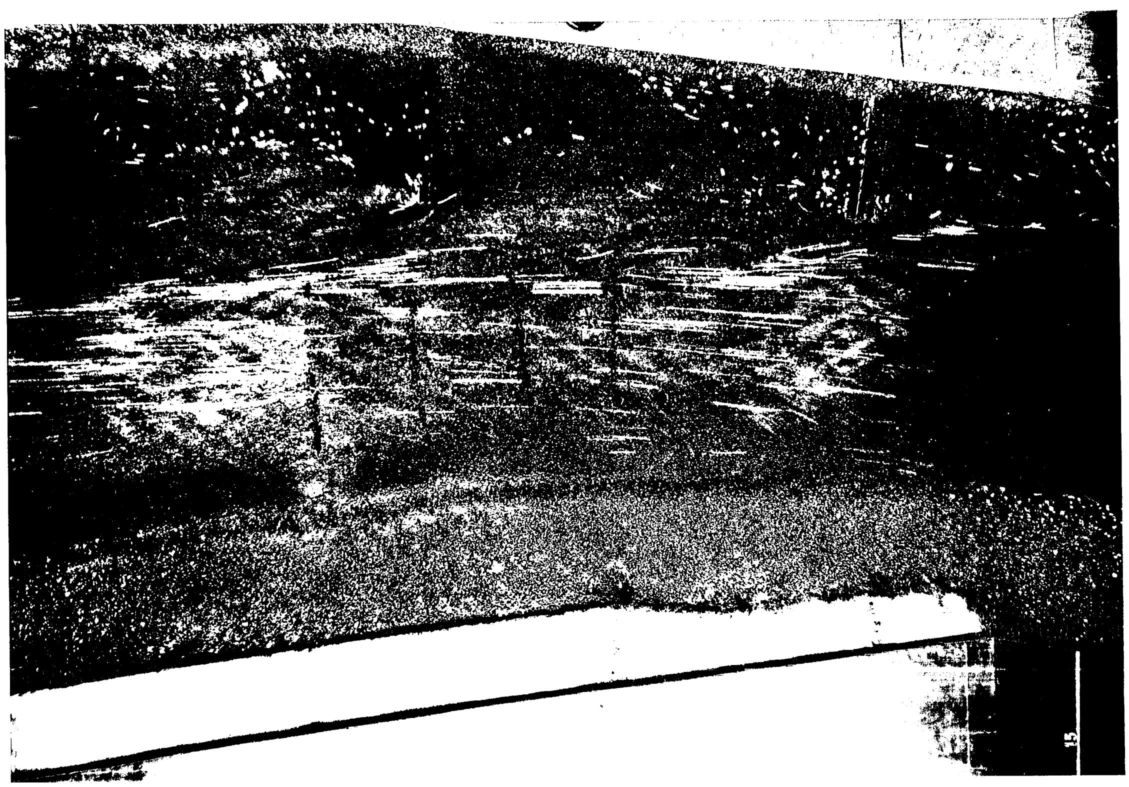


Photo 7

PHOTO 8 (Serial No. 142-145A) Test D-12. In this test dike 5 at station 25 has been added to the dike field. The relatively quiet water on the right bank has been extended down to the bridges by the addition of dike 5. The angle of the streamlines relative to the pier alignment has been reduced to 13 degrees from 19 degrees that existed prior to the placement of dike 5. The flow has been shifted towards the left bank and the velocity has been increased. The high velocity flow now enters the section of the bridges more symmetrically.

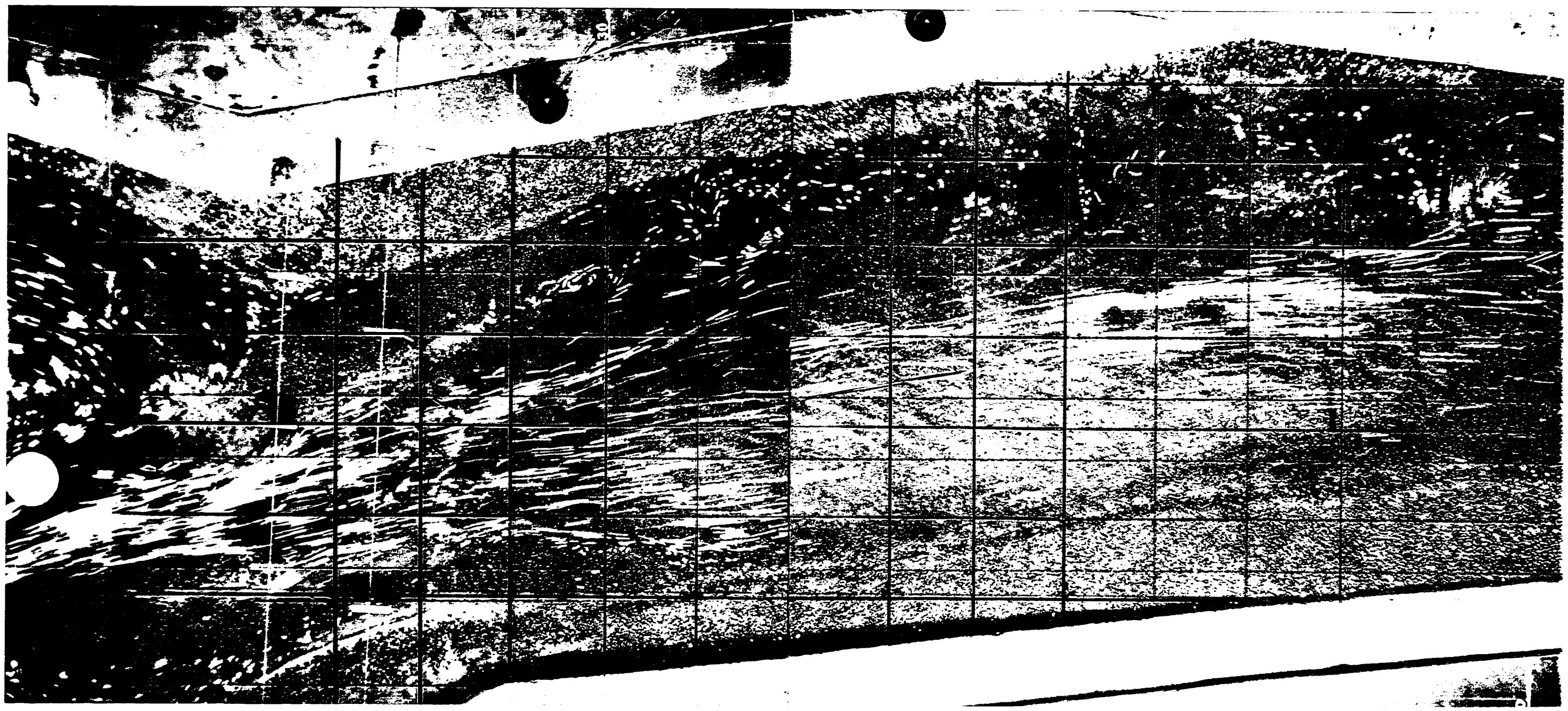


Photo 8

PHOTO 9 (Serial No. 142-149) Test D-13. In order to determine whether another dike between dike 5 and the bridge would be helpful to the flow pattern, dike 6 has been added. The resulting flow pattern is shown. The effect of the dike was to increase the transverse depth of the area of quiet water and to shift the transverse velocity profile towards the left bank. The beneficial effects of dike 6 were negligible and in fact may be somewhat detrimental.

Photo 9

PHOTO 10 (Serial No. 142-158) This mosaic shows the character of erosion along the left bank when the gravel has been replaced by fine sand. The discharge is 20,000 cfs but the increased velocity along the left bank is conducive to considerable erosion.

Photo 10

PHOTO 11 (Serial No. 142-161) Test D-18. This test was to determine the effectiveness of an added length of the dike on the realignment of the flow pattern. Dike 2 has been increased 10 ft in length. The added length constricts the flow and increases the transverse depth of the quiet zone but had little effect on the reduction of the separation zone along the left bank. The shift in the transverse velocity profile appeared to be directly proportional to the increase in length of the dike. Q = 55,000 cfs.

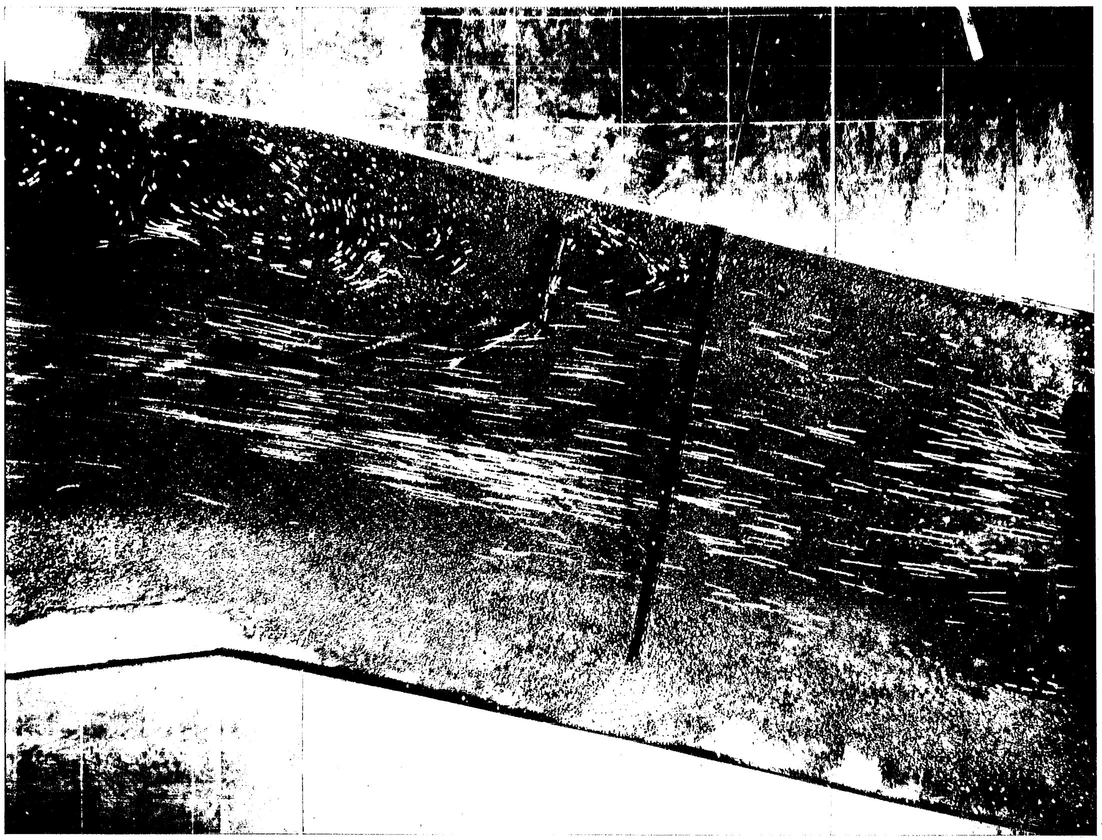
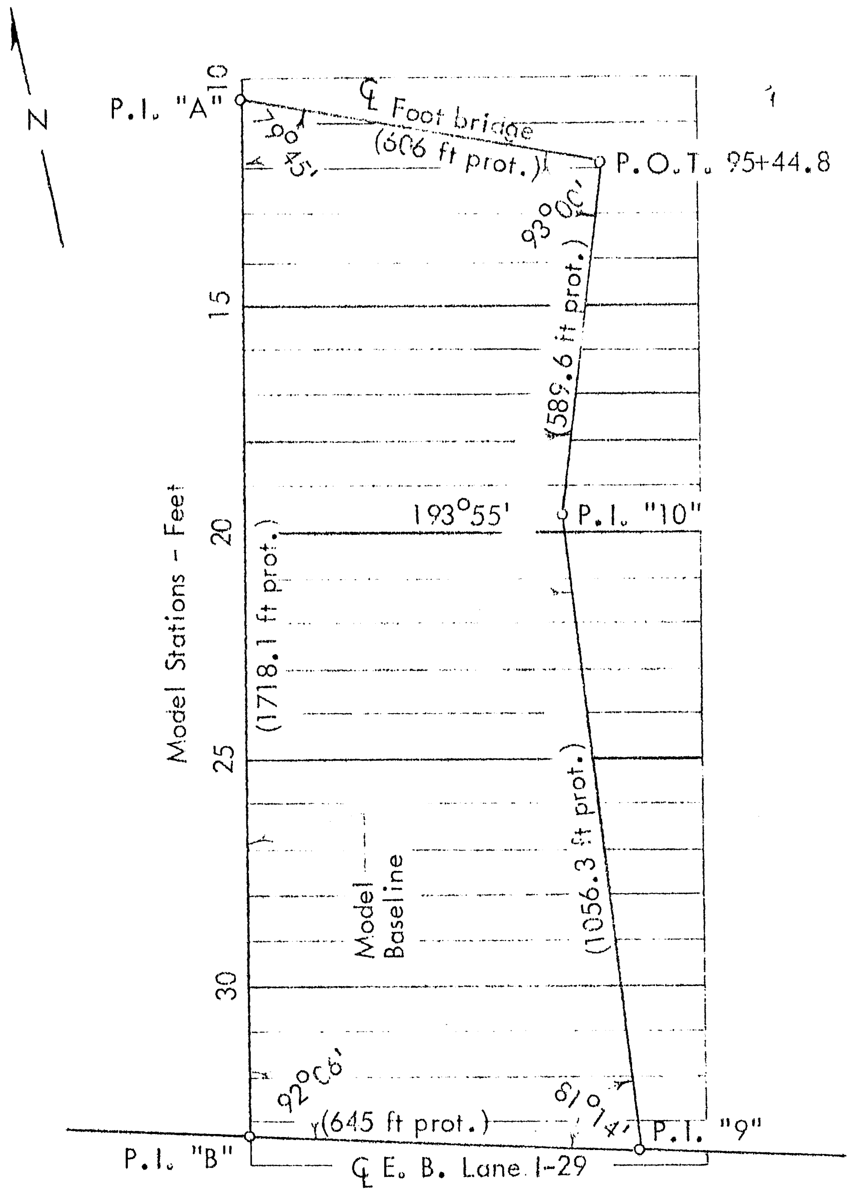


Photo 11

LIST OF CHARTS



P.I. "9" (Corps of Engr. Baseline) Sta. 78+99.02 = P.O.T. I-29 E. B. Lane Sta. 100+52.7

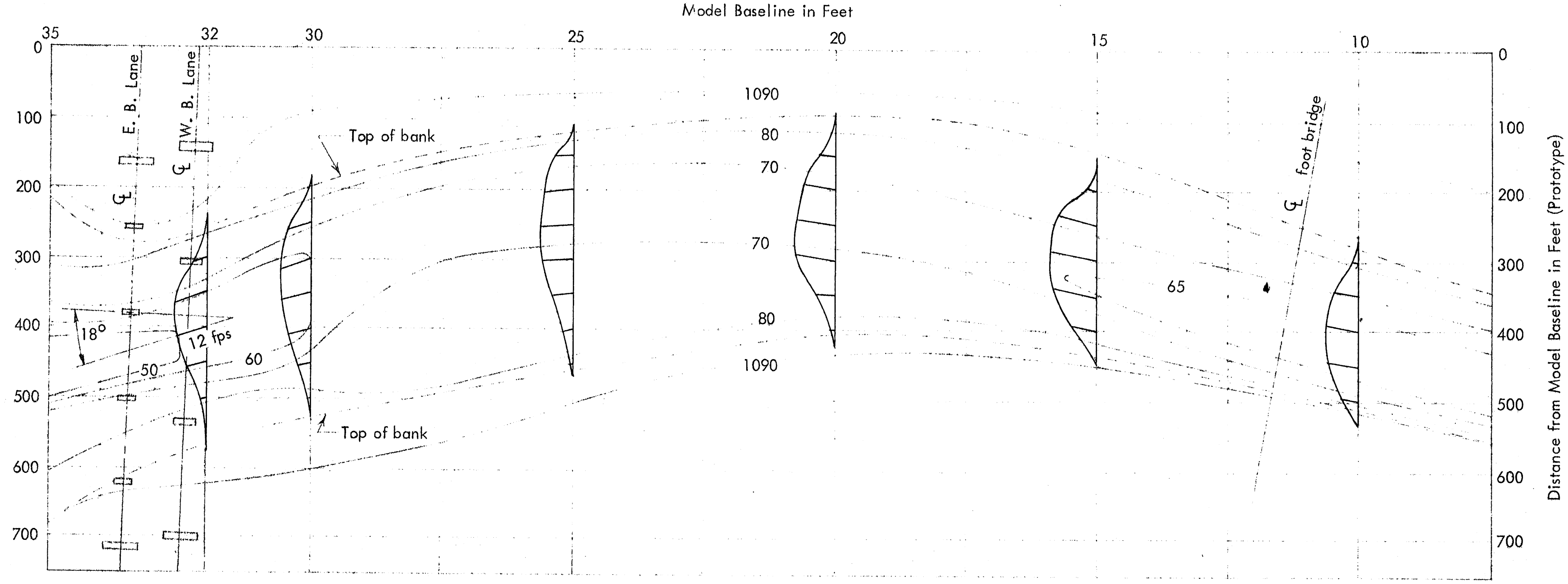
P.I. "10" (Corps of Engr. Baseline) Sta. 89+55.32

P.O.T. (Corps of Engr. Baseline) Sta. 95+44.8 = A point on ℄ of Foot Bridge
606 ft Left of Model P.I. "A"

P.I. "A" (Model Baseline) Sta. 10.47 =
A point on ℄ of Foot Bridge

P.I. "B" (Model Baseline) Sta. 33.38 =
P.O.T. I-29 E. B. Lane Sta. 9+07.7

BIG SIOUX PROJECT Iowa State Highway Commission South Dakota Department of Highways PERMEABLE DIKE TEST		
SAINT ANTHONY FALLS HYDRAULIC LABORATORY **UNIVERSITY OF MINNESOTA**		
DRAWN RMK	CHECKED	APPROVED
SCALE	DATE 11-30-65	NO. 142A2795-1



Notes:

1. Plotted velocities are the average of the velocities in the vertical and were obtained with a cup type current meter.
2. Direction of flow on velocity profiles obtained from photos.
3. Contours were obtained from the Corps of Engineers and I.S.H.C. data of 1964.

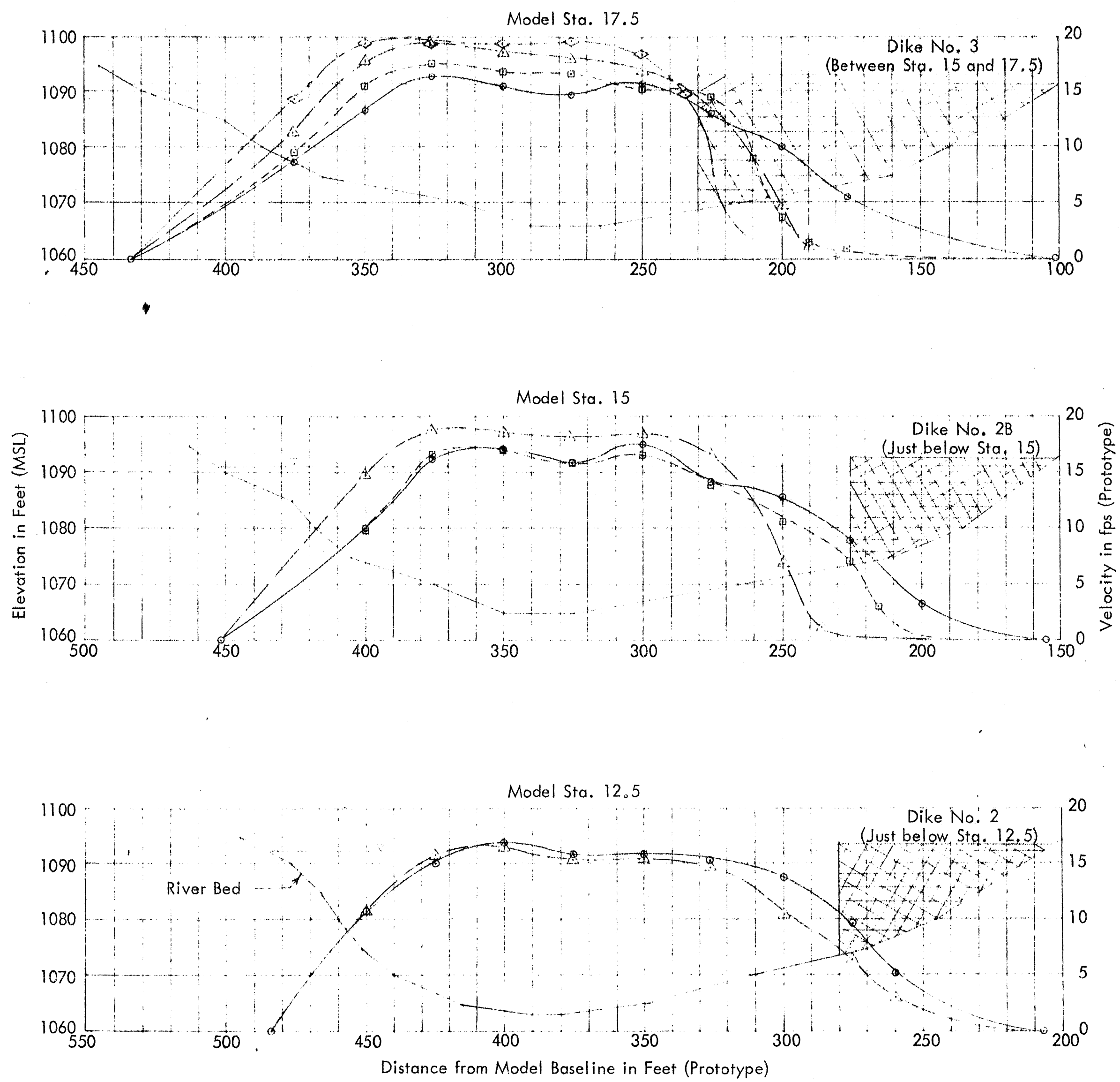
Plan Scale: 1 in. = 150 ft (Prototype)
Velocity Scale: 1 in. = 40 fps

CONTOURS OF RIVER BED AND TRANSVERSE VELOCITY PROFILES PRIOR TO DIKE INSTALLATION
Q = 55,000 cfs
Stage at Bridges = 1090.2 ft
Model Scale 1:75

BIG SIOUX PROJECT
Iowa State Highway Commission
South Dakota Department of Highways
PERMEABLE DIKE TEST

SAINT ANTHONY FALLS HYDRAULIC LABORATORY
UNIVERSITY OF MINNESOTA

DRAWN RMK	CHECKED	APPROVED
SCALE	DATE 12-3-65	NO. 142B442-52



LEGEND

- ○ Velocity profiles prior to installation of dikes
- □ Velocity profiles for test D-8. Dike no. 2B in place
- △ Velocity profiles for test D-9. Dikes no. 2 and 2B in place
- ◇ Velocity profiles for test D-10. Dikes no. 2 and 3 in place

Notes:

1. Velocities shown are the average of the velocities in the vertical and were obtained with a cup type current meter.
2. The river bed cross-sections were obtained from I.S.H.C. and Corps of Engineers contour maps of 1964.
3. Velocity profiles are not shown if there is no change from velocity profile prior to dike installation.

AVERAGE TRANSVERSE VELOCITY
DISTRIBUTION FOR VARIOUS
DIKE PLACEMENTS
Q = 55,000 cfs
Stage at Bridges = 1090.2 ft
Model Scale 1:75

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Iowa State Highway Commission
South Dakota Department of Highways
PERMEABLE DIKE TEST

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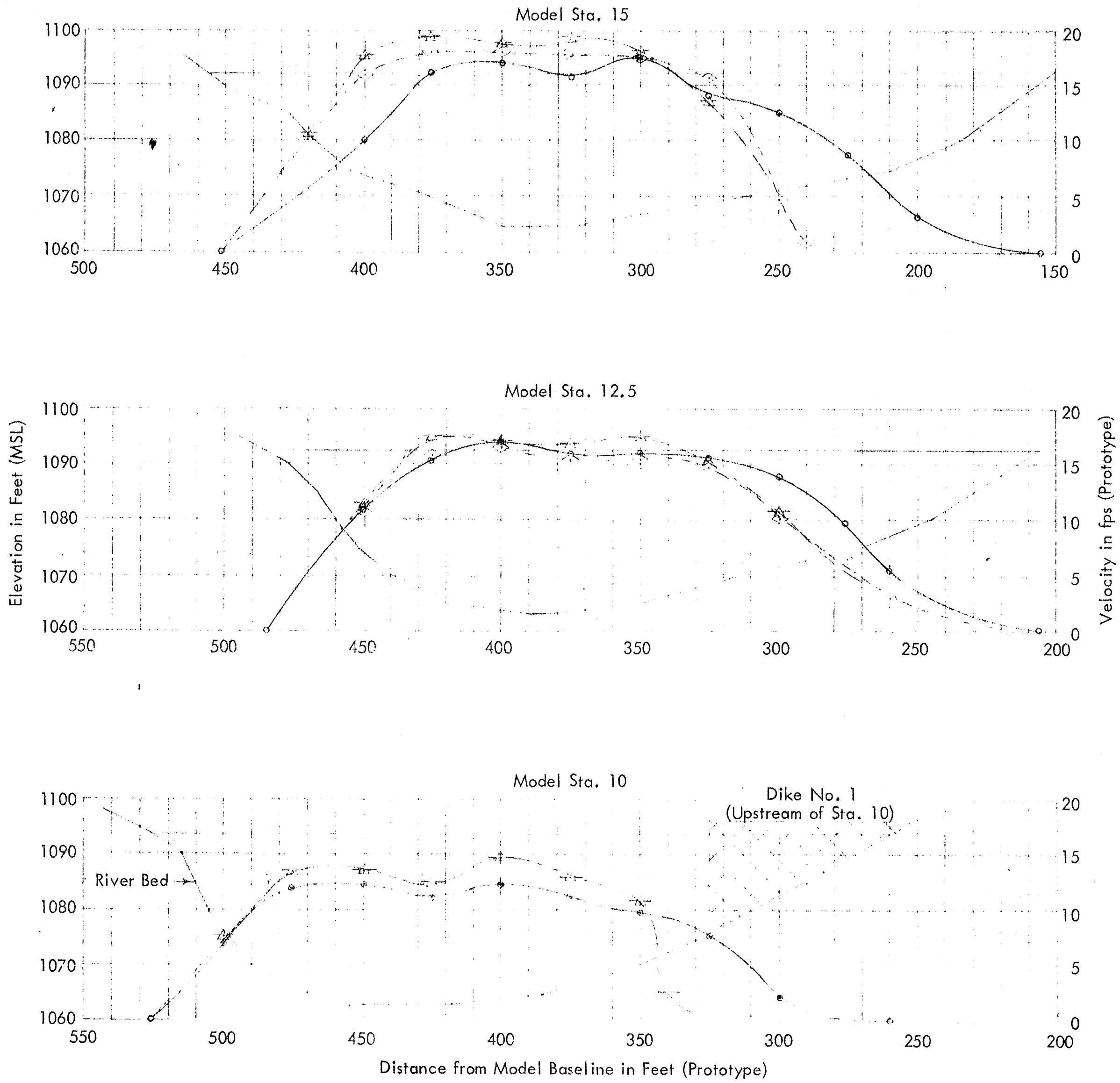
DRAWN RMK	CHECKED	APPROVED
SCALE	DATE 12-1-65	NO. 142B442-54



LEGEND

- Velocity profiles prior to installation of dikes
- Velocity profiles for test D-10. Dikes no. 2 and 3 in place
- Velocity profiles for test D-19. Dikes no. 1, 2, and 3 in place

Notes:

1. Velocities shown are the average of the velocities in the vertical and were obtained with a cup type current meter.
2. The river bed cross-sections were obtained from I.S.H.C. and Corps of Engineers contour maps of 1964.
3. Velocity profiles are not shown if there is no change from velocity profile prior to dike installation.

AVERAGE TRANSVERSE VELOCITY DISTRIBUTION FOR TEST D-19
(Effect of Placing Dike No. 1 Upstream of Bend in River)
Q = 55,000 cfs
Stage at Bridges = 1090.2 ft
Model Scale 1:75

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PERMEABLE DIKE TEST

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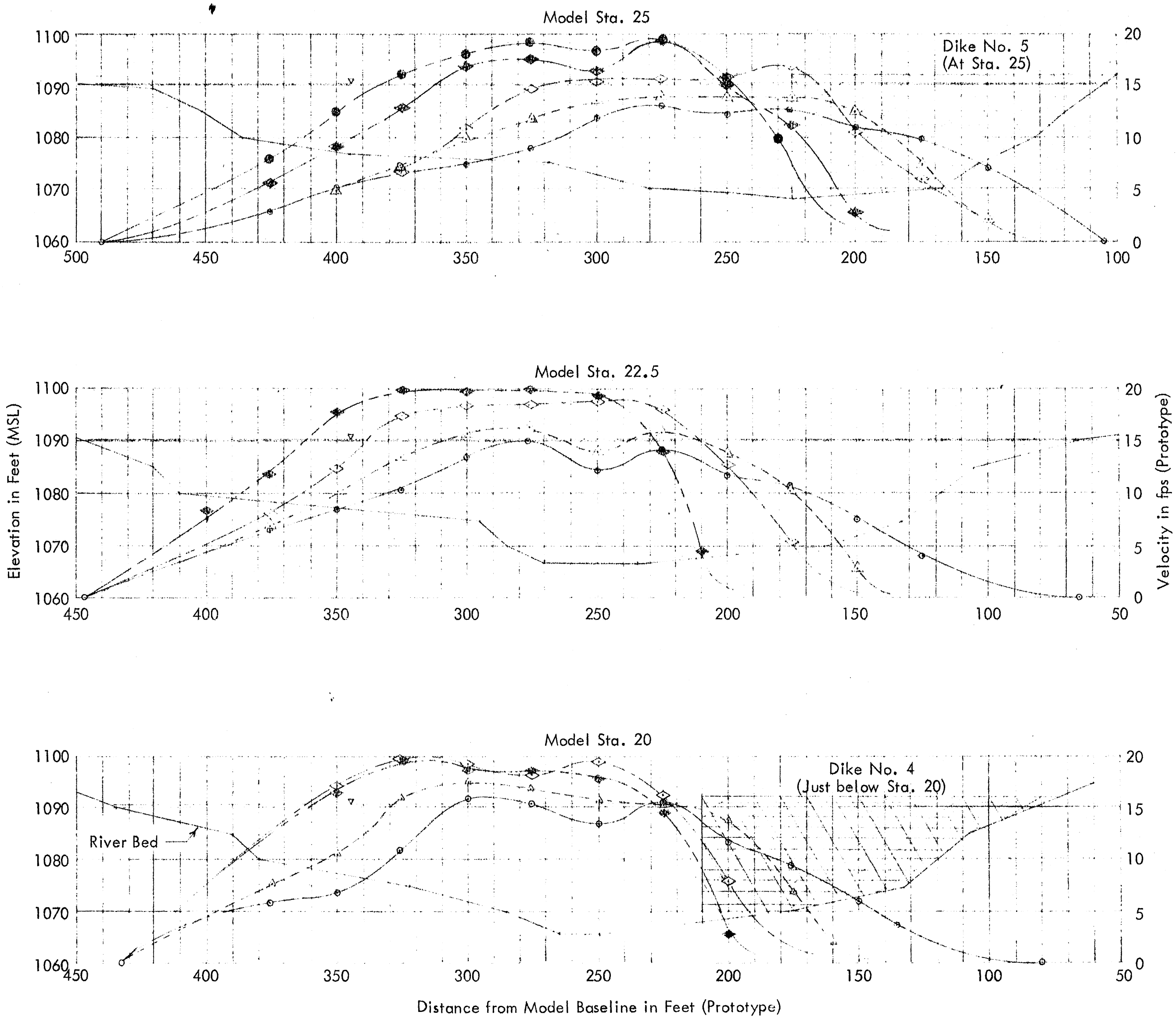
DRAWN RMK	CHECKED	APPROVED
SCALE	DATE 12-1-65	NO. 142B442-57



LEGEND

- ○ Velocity profiles prior to installation of dikes
- △ Velocity profiles for test D-9. Dikes no. 2 and 2B in place
- ◇ Velocity profiles for test D-10. Dikes no. 2 and 3 in place
- ◆ Velocity profiles for test D-11. Dikes no. 2, 3, and 4 in place
- ● Velocity profiles for test D-12. Dikes no. 2, 3, 4 and 5 in place

Notes:

1. Velocities shown are the average of the velocities in the vertical and were obtained with a cup type current meter.
2. The river bed cross-sections were obtained from I.S.H.C. and Corps of Engineers contour maps of 1964.
3. Velocity profiles are not shown if there is no change from velocity profile prior to dike installation.

AVERAGE TRANSVERSE VELOCITY DISTRIBUTION FOR VARIOUS DIKE PLACEMENTS
Q = 55,000 cfs
Stage at Bridges = 1090.2 ft
Model Scale 1:75

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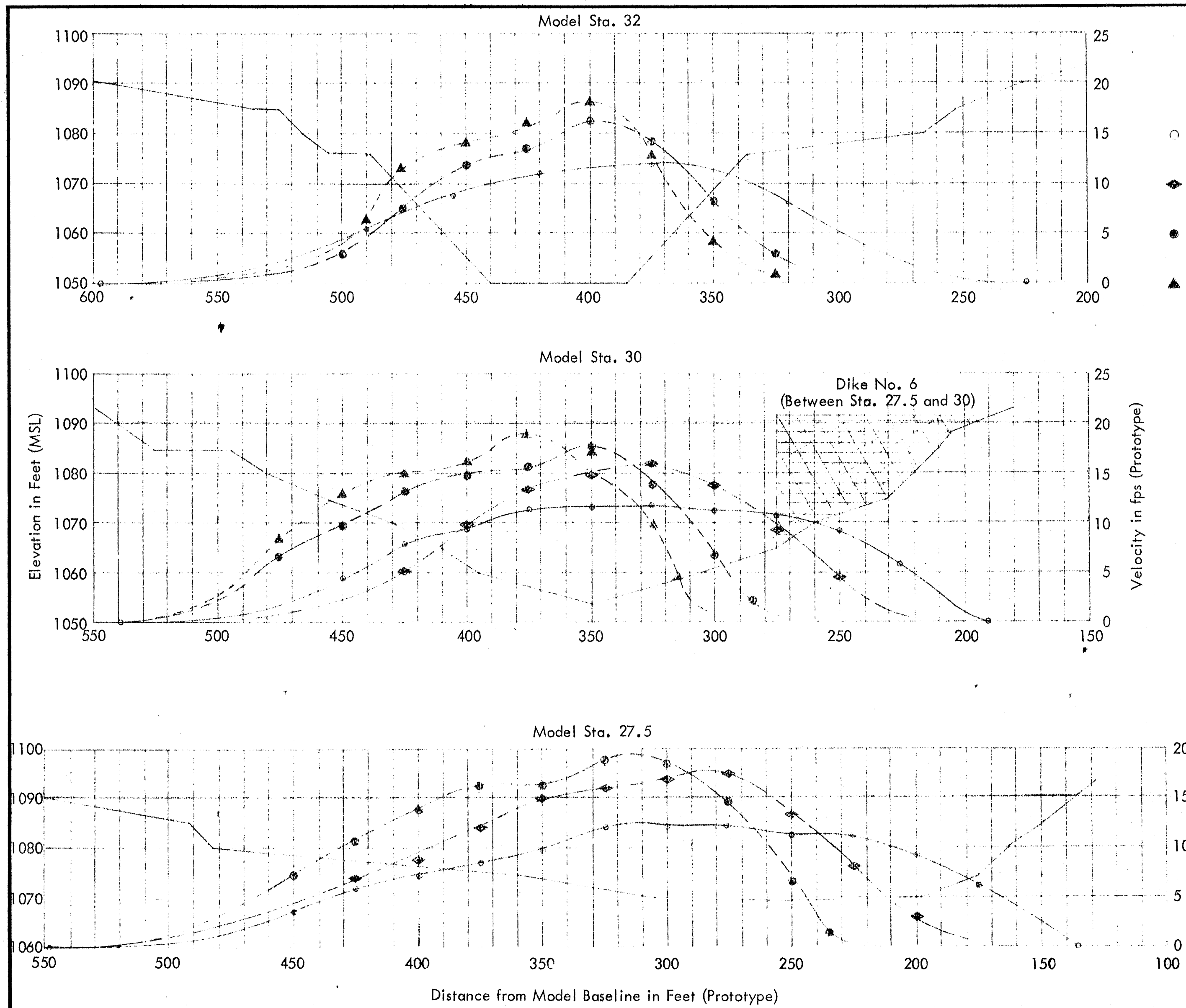
UNIVERSITY OF MINNESOTA

DRAWN RMK	CHECKED	APPROVED
SCALE	DATE 12-2-65	NO. 142B442-55



LEGEND

○ Velocity profiles prior to installation of dikes

◆ Velocity profiles for test D-11. Dikes no. 2, 3 and 4 in place

● Velocity profiles for test D-12. Dikes no. 2, 3, 4 and 5 in place

▲ Velocity profiles for test D-13. Dikes no. 2, 3, 4, 5 and 6 in place

Notes:

1. Velocities shown are the average of the velocities in the vertical and were obtained with a cup type current meter.
2. The river bed cross-sections were obtained from I.S.H.C. and Corps of Engineers contour maps of 1964.
3. Velocity profiles are not shown if there is no change from velocity profile prior to dike installation.

AVERAGE TRANSVERSE VELOCITY DISTRIBUTION FOR VARIOUS DIKE PLACEMENTS
Q = 55,000 cfs
Stage at Bridges = 1090.2 ft
Model Scale 1:75

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PERMEABLE DIKE TEST

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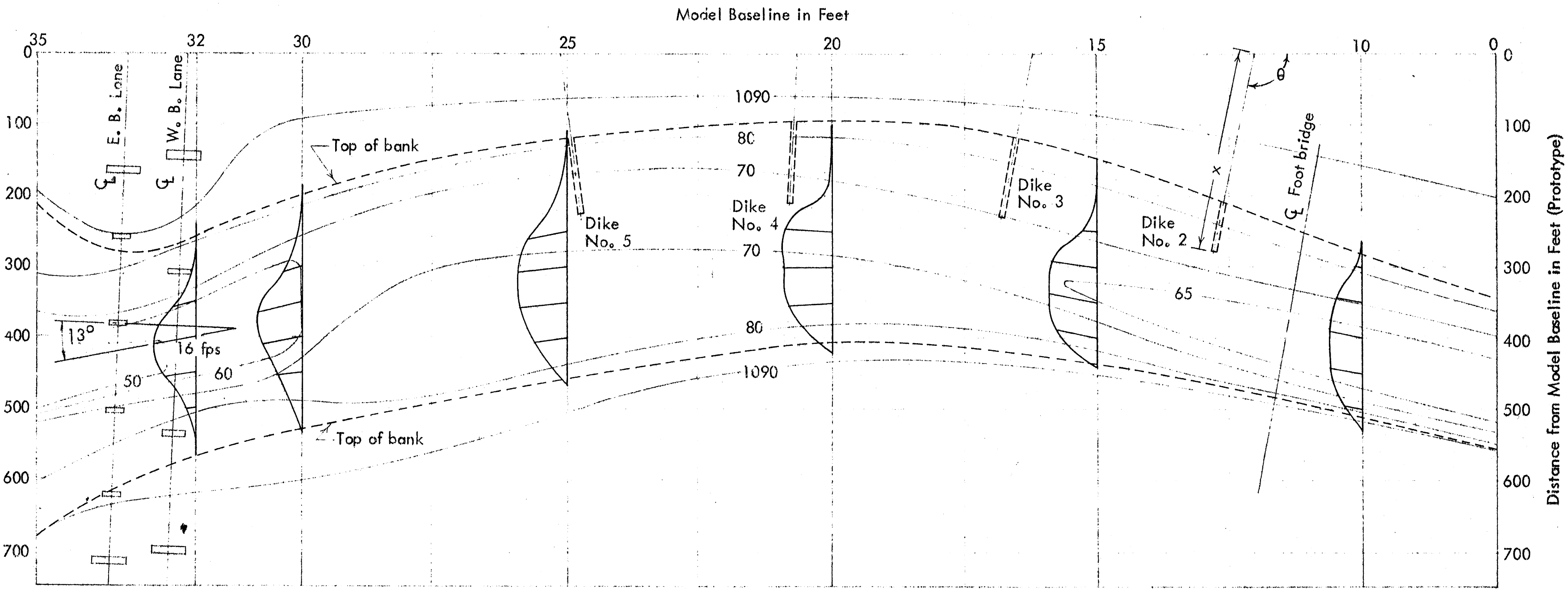
DRAWN RMK	CHECKED	APPROVED
SCALE	DATE 12-2-65	NO. 142B442-56



Notes:

1. Plotted velocities are the average of the velocities in the vertical and were obtained with a cup type current meter.
2. Direction of flow on velocity profiles obtained from photos.
3. Contours were obtained from the Corps of Engineers and I.S.H.C. data of 1964.

MODEL DIKE LOCATIONS

Dike Number	2	3	4	5
Model Baseline Station	12.0	16.21	20.65	25.12
Angle with Baseline θ	102°	101°	93°	83°
Distance "x" in Ft (Prototype)	285	235	210	225
Dike Length in Ft (Prototype)	75	135	135	125

Plan Scale: 1 in. = 150 ft (Prototype)
Velocity Scale: 1 in. = 40 fps

TRANSVERSE VELOCITY PROFILES
FOR FINAL DIKE CONFIGURATION
Q = 55,000 cfs
Stage at Bridges = 1090.2 ft
Model Scale 1:75

BIG SIOUX PROJECT
Iowa State Highway Commission
South Dakota Department of Highways
PERMEABLE DIKE TEST

SAINT ANTHONY FALLS HYDRAULIC LABORATORY
UNIVERSITY OF MINNESOTA

DRAWN RMK	CHECKED	APPROVED
SCALE	DATE 11-30-65	NO. 142B442-53