G. J. Hanson
R. A. Lohnes
F. W. Klaiber

Final Report on Phases II and III

Evaluation of Control Structures for Stabilizing Degrading Stream Channels in Western Iowa

July 1985

Iowa DOT HR-208A
Project 1502
ISU-ERI-Ames-86050

Sponsored by the Iowa Department of Transportation, Highway Division, and the Iowa Highway Research Board

report

College of Engineering
Iowa State University

G. J. Hanson
R. A. Lohnes
F. W. Klaiber

Final Report on Phases II and III

Evaluation of Control Structures for Stabilizing Degrading Stream Channels in Western Iowa

July 1985

Submitted to
the Highway Division of
the Iowa Department of Transportation
and
the Iowa Highway Research Board

Iowa DOT HR-208A
Project 1502
ISU-ERI-Ames-86050

DEPARTMENT OF CIVIL ENGINEERING
ENGINEERING RESEARCH INSTITUTE
IOWA STATE UNIVERSITY, AMES, IOWA 50011

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF PHOTOPLATES

APPENDIX 1

THE PROBLEM

Since the turn of the century, tributaries to the Missouri River in western Iowa have entrenched their channels to as much as six times their original depth. This channel degradation is accompanied by widening as the channel side slopes become unstable and landslides occur. The deepening and widening of these streams have endangered about 25% of the highway bridges in 13 counties [Lohnes et al. 1980].

Grade stabilization structures have been recommended as the most effective remedial measure for stream degradation [Brice et al., 1978]. In western Iowa, within the last seven years, reinforced concrete grade stabilization structures have cost between $300,000 and $1,200,000. Recognizing that the high cost of these structures may be prohibitive in many situations, the Iowa Department of Transportation (Iowa DOT) sponsored a study at Iowa State University (ISU) to find low-cost alternative structures. This was Phase I of the stream degradation study. Analytical and laboratory work led to the conclusion that alternative construction materials such as gabions and soil-cement might result in more economical structures [Lohnes et al. 1980]. The ISU study also recommended that six experimental structures be built and their performance evaluated. Phase II involved the design of the demonstration structures, and Phase III included monitoring and evaluating their performance.

The Shelby and Pottawattamie County Supervisors agreed to participate in the construction of these demonstration structures with the counties providing 25% of the construction costs and Iowa DOT Highway

Research Board providing 50%. It was expected that the remaining 25% would come from a third agency that would have some interest in the results of a full-scale field experiment. Because the agricultural sector would benefit from stream channel stabilization, attempts were made by the ISU research team to obtain funding from various agencies within the U.S. Department of Agriculture. These attempts to obtain the final funding increment were unsuccessful; however, the Iowa State Water Resources Research Institute (ISWRRI) provided sufficient funds for 25% of two structures, one in Shelby County and the other in Pottawattamie County.

CONSTRUCTION COSTS AND PROBLEMS

Plans were developed for a soil-cement structure in Shelby County and a gabion structure in Pottawattamie County. The original cost estimate of the Shelby County structure was about $60,000; however, the final cost estimate doubled because of problems anticipated with the excavation of the stilling basin. In spite of the large increase in the estimated cost, no bids were received at the scheduled March 1982 letting. The construction money allocated to this project reverted to ISWRRI and was reallocated to other projects within the Institute.

Although the laboratory studies at ISU suggest that soil-cement is a feasible construction material for grade stabilization structures [Litton and Lohnes 1982], the absence of any contractor willing to bid on the project indicates that a major practical problem exists with the use of soil-cement in this type of structure. The problem may be associated with conditions at this specific site or with the lack of contractor experience in constructing soil-cement water control structures. Another possibility is that the contractors did not accept the results of the lab studies and needed evidence of the field performance of such structures. If lack of construction experience is the reason for no bids, then specifications outlining construction procedures need to be developed. If the third reason is the primary cause for the lack of bids on the Shelby County structure, field scale research needs to be conducted to support or reject the validity of the laboratory work. It is the Shelby county engineer's opinion that lack of contractor experience in mixing and placing soil-cement is the major problem; in addition, the practical construction problems may drive the cost of the structure so high they will offset any savings in material cost [Eldo Schornhorst, personal communication, Nov. 7, 1984].

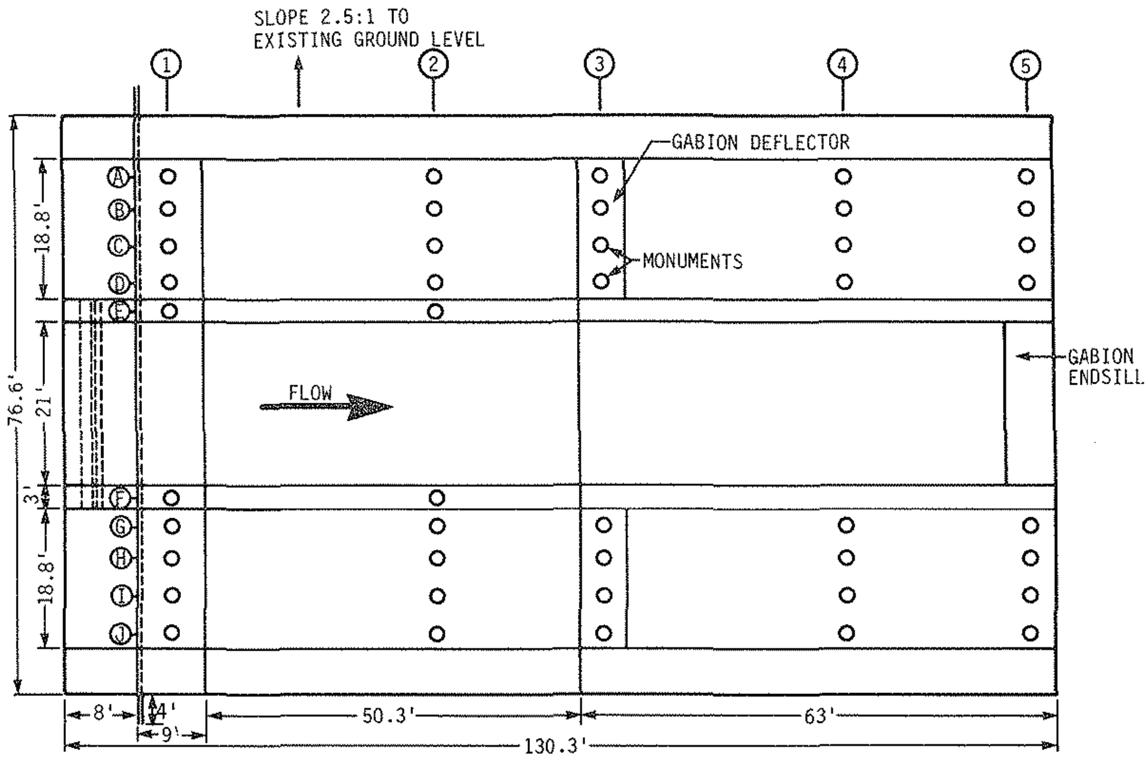
The Potawattamie County gabion structure was originally estimated at a cost of $60,000; but after detailed design and modifications suggested by the county engineer, the cost estimate increased to $85,000. Bid letting was September 16, 1982, when with three bids were received; the lowest was $97,000. The required additional funds were provided by the county supervisors, and construction began November 29. The photographs in Appendix 1 provide documentation of the construction's progress. Except for four weeks during January, construction continued through the winter. Although several problems were encountered during construction, the contractor was able to get water through the structure by May 16. The structure was completed by June 30, 1983 at a final cost of \$108,000. The cost overrun was due largely to construction problems. A comparative cost analysis of this gabion structure with reinforced concrete structures follows in this report.

DESCRIPTION OF THE GABION GRADE STABILIZATION STRUCTURE

The demonstration grade stabilization structure is located on Keg Creek, three miles east of McClelland at Sec 1-75-42. The drop structure is situated 100 ft downstream from a bridge where, since 1958, 14 ft of channel degradation had exposed bridge piers and caused landslides that removed soil from the east abutment. The drainage area of Keg Creek at this location is approximately 90 sq mi. Prior to construction, the stream gradient was from 6 to 8 ft/mi with a channel width of about 50 ft at top.

The structure consists of a gabion weir and ramp with a net drop of 12.6 ft, which is intended to reduce the effects of the degradation at the bridge site. Fig. 1 shows the plan and profile of the structure. The bottom width of the weir and ramp is 21 ft with 2:1 side slopes extending 27 ft upward. The ramp is 51 ft long with a 4:1 downstream slope. The stilling basin has a length of 63 ft. Photoplate 1 is a photograph of the structure as it appeared in Summer 1985.

The structure was designed for the 50-year-frequency flood of 9,930 cfs to contain the hydraulic jump and avoid overbank flooding. As a point of comparison, the two-year-frequency flood is estimated at



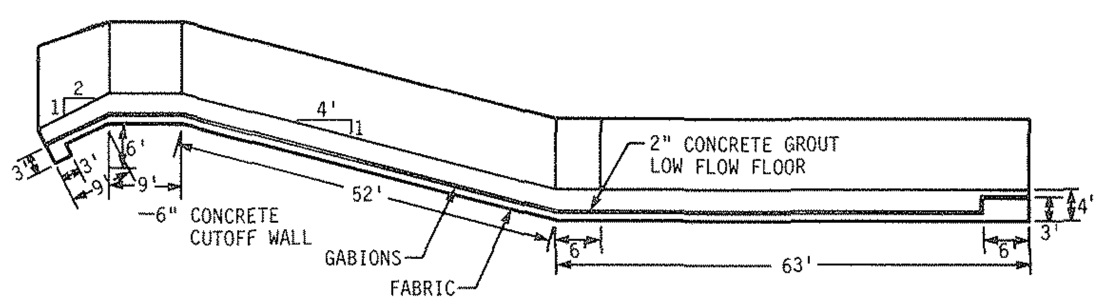


Figure 1. Plan of gabion drop structure showing location of monuments.

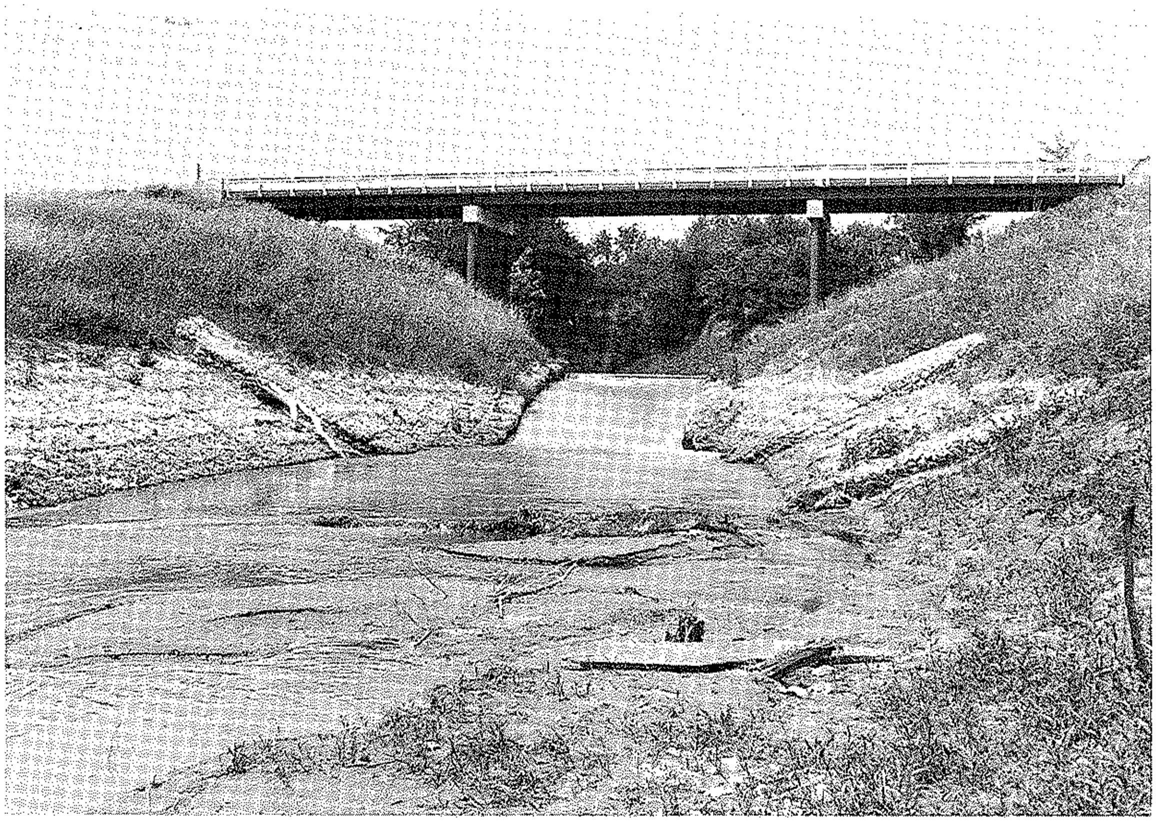


Plate 1. Completed gabion grade control structure as it appeared in June 1985.

2,190 cfs. Prior to construction of the structure the channel had the capacity to contain the 100-year-frequency flood of about 12,000 cfs. It is expected that the structure will cause a four-ft rise in the water surface elevation upstream of the structure during the 100-year flood, but will not cause overbank flow.

MONITORING PERFORMANCE OF THE STRUCTURE

The monitoring of the structure included differential settlement measurements of the structure, measurements of upstream aggradation and downstream degradation subsequent to placement of the structure, measurements of stream flow through the structure, and qualitative observations of structural deterioration.

SETTLEMENT MEASUREMENTS

In order to monitor the differential movement of the structure, concrete monuments were placed on the surface as shown in Fig. 1. Elevations of the monuments were measured at five different times: 6/29/83, 11/7/83, 6/8/84, 8/22/84, and 6/5/85. The elevation data were used to plot all of the transverse cross sections at the various dates. These plots revealed that virtually no differential settlement has occurred within the structure throughout the course of the investigation. Figure 2 is a typical cross section, and Fig. 3 shows the cross section that exhibited the maximum amount of settlement. In Fig. 3

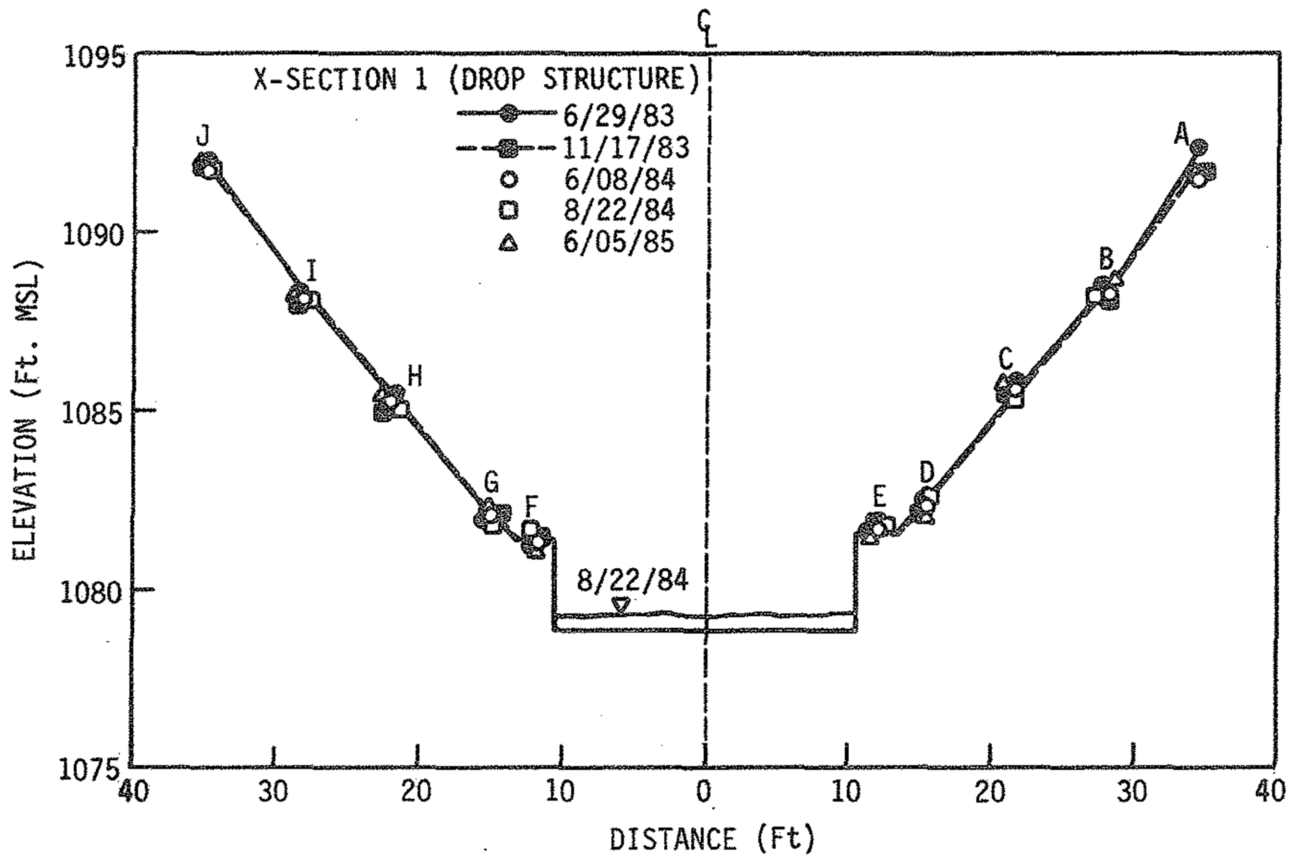


Figure 2. Cross section at top of structure. Note differential movement at monument A. This is the only measurable movement observed in the structure.

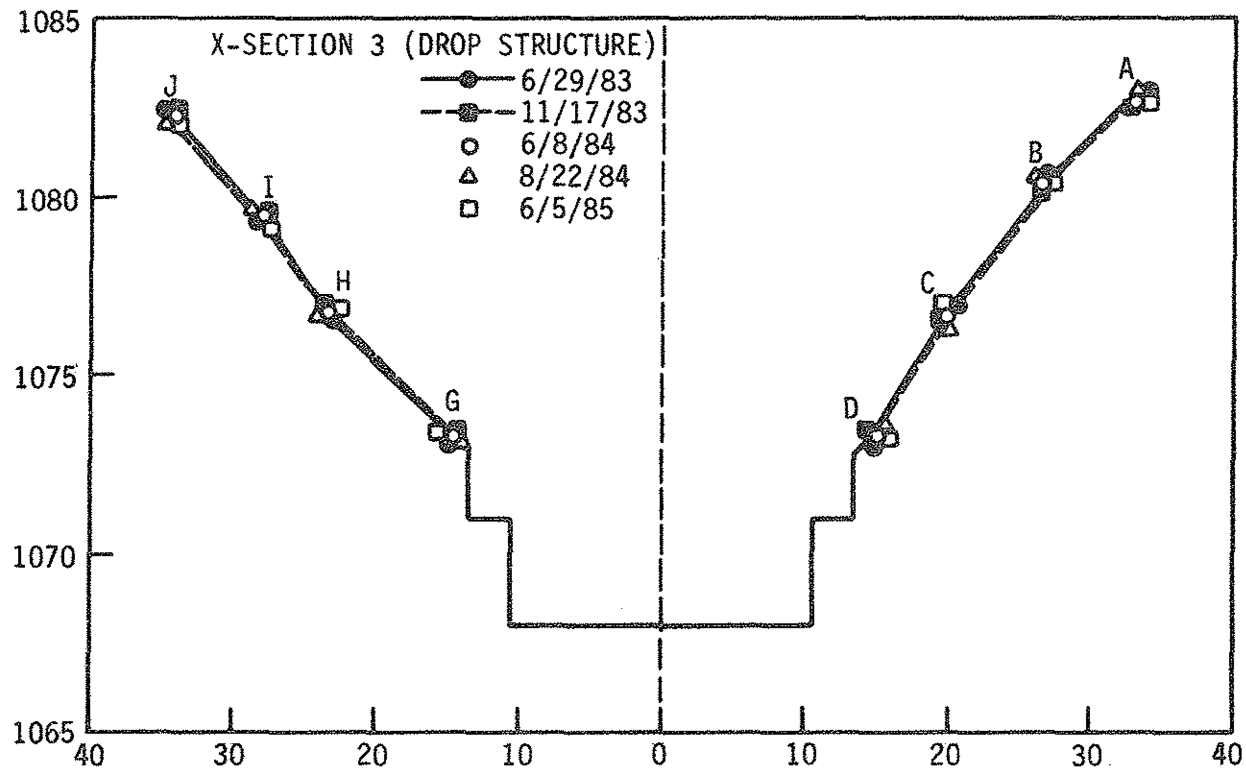


Figure 3. Cross section at about middle of structure. Virtually no differential movement is observed, which is typical of all 5 cross sections.

the monument 1A, at the top of the sideslope immediately downstream from the crest, settled about four inches between the first two observation dates. No differential movement has been observed since 11/7/83; thus, it has been concluded that differential settlement is not a problem. Because soil consolidation occurs most rapidly soon after loading, it appears settlement will not be a problem.

OBSERVATIONS OF DETERIORATION

Minor deterioration of the structure is being observed visually and has been recorded on photographs. Some deformation of the side slope is apparent in the vicinity of monuments 2D and 2E, but this movement occurred during construction after a very high runoff event. Because runoff had filled the channel, the contractor, in an effort to dewater the site and resume work quickly, pumped the water out of the channel in a very short time. It is interpreted that this rapid drawdown condition created instability and caused slippage. A rapid drawdown condition is not likely to occur during normal operation of the structure; consequently, the side slopes are expected to be stable in the future. The stability of the side slopes is verified by the constant elevations of the monuments. Although the observed slope deformation is not of great concern, anchors were placed on the slope as a precaution.

A scour hole, shown in Photoplate 2, has developed immediately downstream of the stilling basin on the west bank. The hole is roughly ten ft long parallel to the channel and extends about three ft into the bank. This area is where the construction diversion channel was located, and the backfill in this area may have been improperly compacted. This area should be continuously observed for any expansion of the scour hole. If erosion proceeds in the upstream direction, it may undermine the side slope and stilling basin. If the hole expands, it should be protected with riprap or additional gabions.

Photoplate 3 reveals that flow is being deflected toward the west bank. The structure was built with a gabion apron extending nine ft downstream from the end sill. The apron has experienced some differential settlement with the west side lower than the east. This seems to be diverting the flow toward the west bank, thereby compounding the scour problem.

FLOW ESTIMATES

Seven gages to measure stream flow were placed on the bridge piers and on posts upstream and downstream of the bridge within a three-mile reach. Each gage consists of a vertical tygon tube attached to a staff gage. The tube has a one-way valve at the bottom which allows water to flow into but not out of the tube. The gages were positioned on the posts to measure high flow events only. Water enters the bottom of the tygon tubing through the one-way valve and rises in the tube as the stream stage rises. After the maximum stage has been reached, the water is trapped in the tube by the one-way valve. This allows measurement

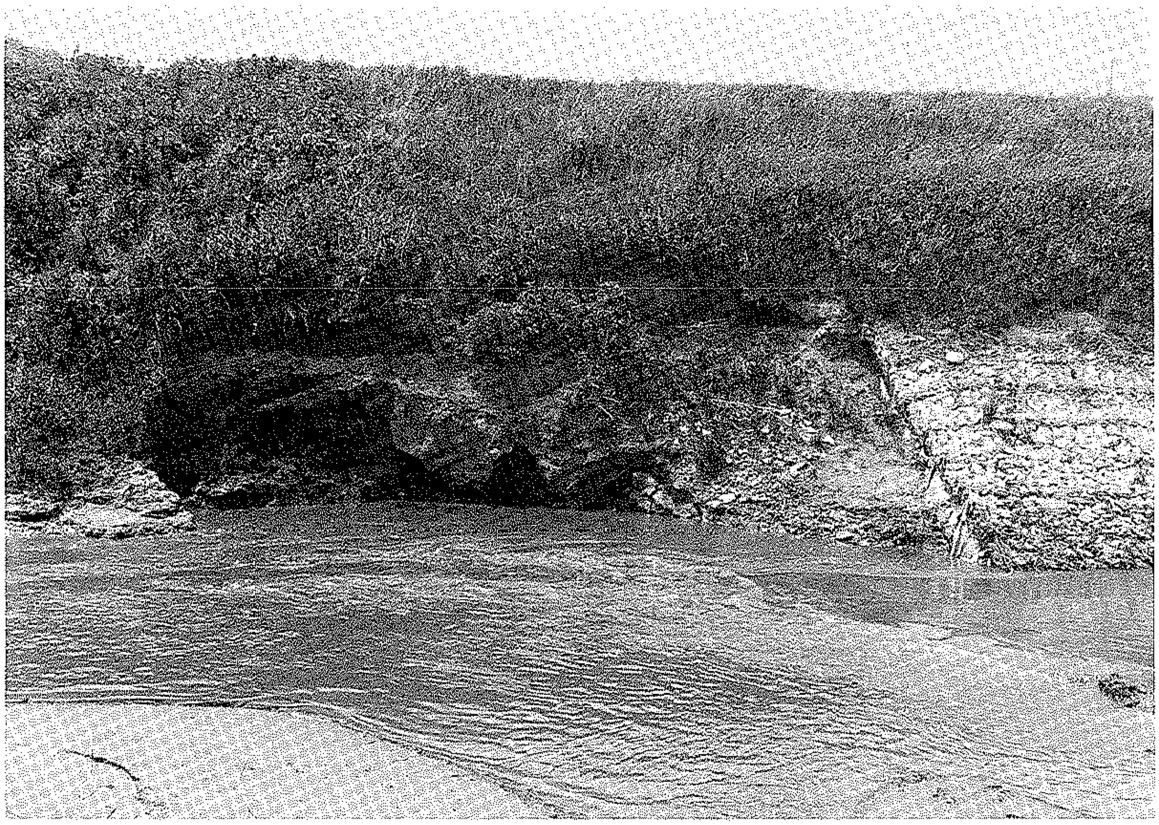


Plate 2. Scour hole on west bank immediately downstream of stilling basin, June 1985.

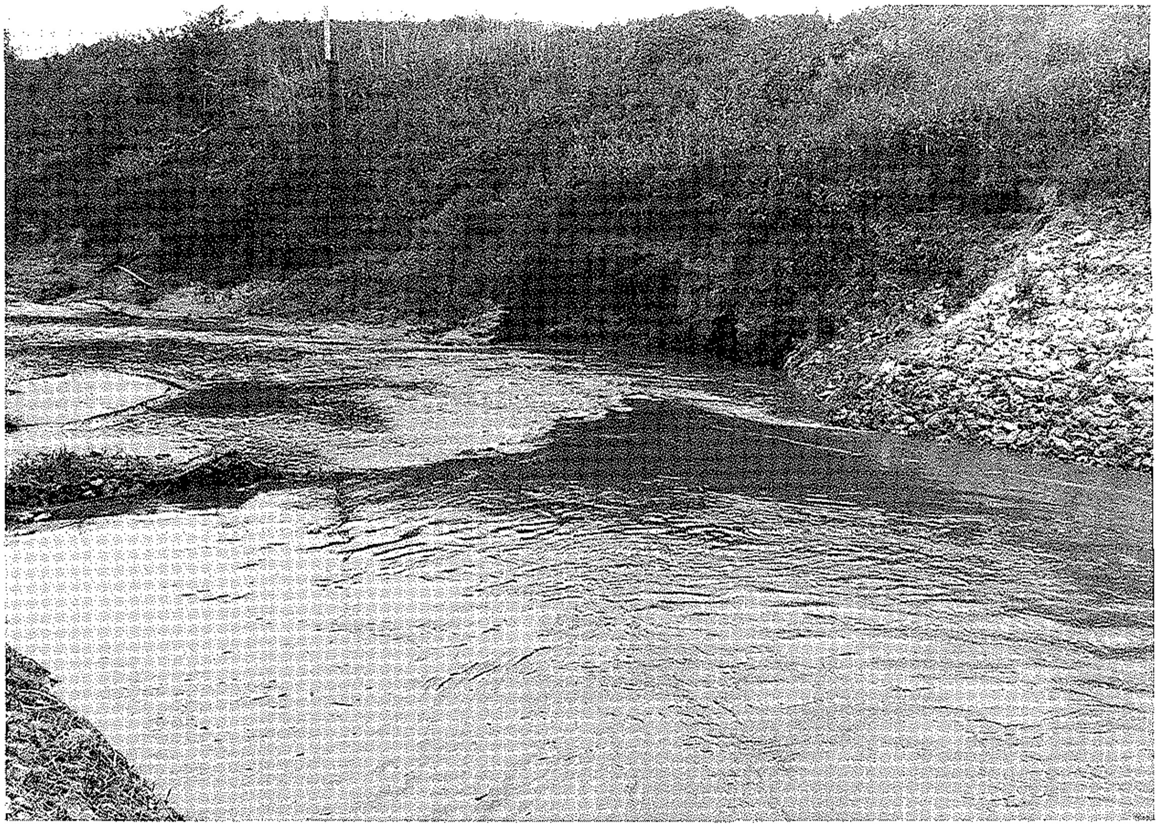


Plate 3. Diversion of flow toward scour hole, probably due to some differential settlement of end of stilling basin, June 1985.

of the maximum stage from the staff gage attached to the tube. The gages have not functioned as well as anticipated. Debris has plugged the one-way valves; consequently, no data were collected from the gages. For future applications of these gages, an attempt should be made to design some type of debris trap on the intake end of the system.

Because the gages failed to perform adequately, an alternate method of estimating flows was devised. The spillway structure produces critical flow at its crest and therefore acts as a control in the stream channel. Controls are defined as certain features in a channel that tend to produce critical flow [Henderson, 1966]. At any feature which acts as a control, the discharge can be calculated if the flow depth is known by using the following relationship:

$$Q = A\sqrt{\frac{gA}{B}}$$

where Q is the discharge flowing through the crest of the spillway in cfs, A is the area of the wetted section in sq ft, B is the corresponding width of the water surface in ft, and g is the acceleration due to gravity (32.2 ft/sec^2). The geometry of the spillway crest of the gabion grade control structure was used to calculate the discharge for various depths of flow from the previous equation; that relationship is shown graphically in Fig. 4. Details of the calculation are in Appendix 2. Note that for the 50-year flood frequency with a discharge of 9,930 cfs, the depth of flow through the structure is 14 ft. For the design of this structure the HEC-2 backwater calculation program

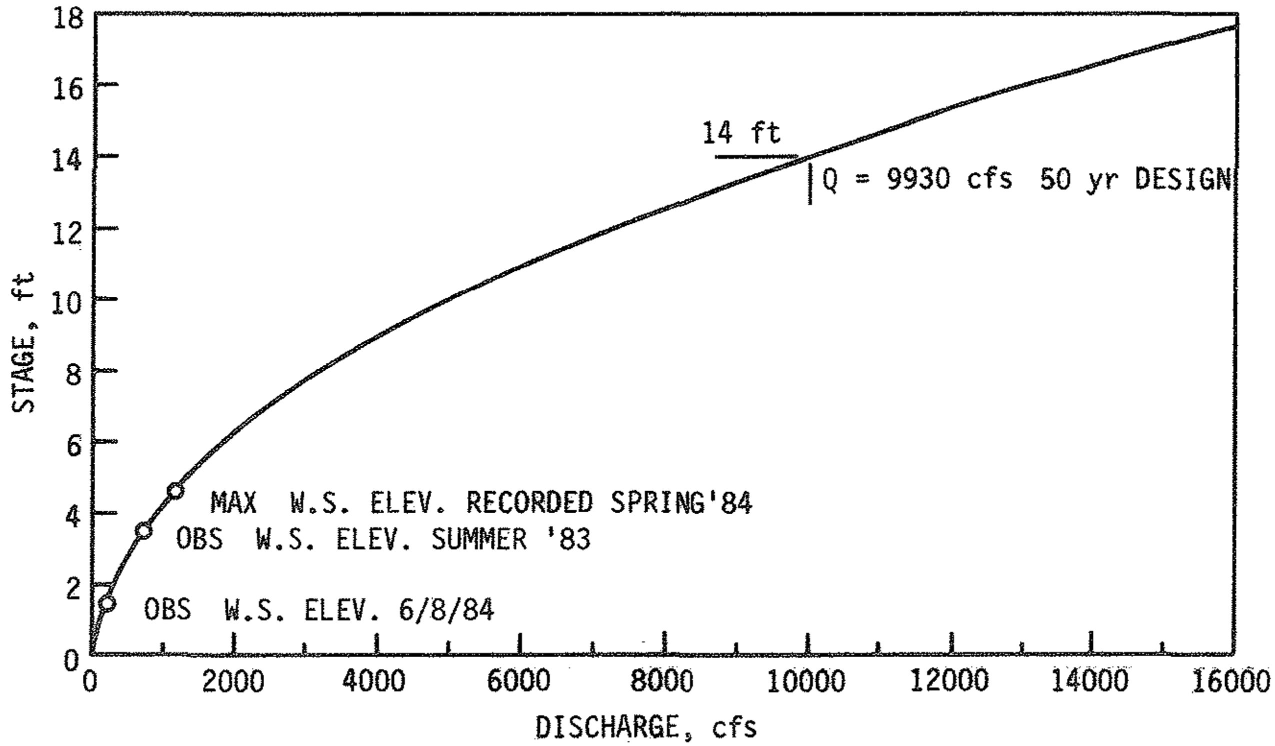


Figure 4. Stage-discharge relationship for drop structure at its crest. Crest elevation is 1079 ft MSL.

was used, and it estimated the flow depth at 13.5. The data shown in Fig. 4 give good agreement with the design estimates.

Debris deposited on the sidewalls of the structure during a flood event are physical evidence of the maximum stage for that event. The elevations of the debris lines were measured during Summer 1983, Spring 1984, on 6/8/84, and on 6/5/85. These depths of flow are plotted on Fig. 4, and indicate that, to date, the flows have been well below the design flood with discharges less than 1,200 cfs.

The water surface profile for the design flow of 9,900 cfs was estimated using the HEC-2 program. The design water surface profile and the water surface profiles for 120 and 1,200 cfs discharges, which were estimated from the debris lines, are shown in Fig. 5. These curves suggest that the downstream effects of the structure and stream force the hydraulic jump upstream onto the spillway to create a submerged jump.

SEDIMENTATION OBSERVATIONS

Changes in the upstream channel geometry caused by sedimentation have been monitored by surveying transverse profiles at the bridge and at 500-ft intervals upstream to a distance of 5,000 ft. Transverse profiles at the bridge were measured on 11/12/83, 6/28/83, 6/8/84, 8/22/84, and 6/5/85. A set of transverse profiles is shown in Fig. 6. These sections show that sedimentation to a depth of 6 ft had occurred prior to 6/28/83, but little change has been noticed since that date.

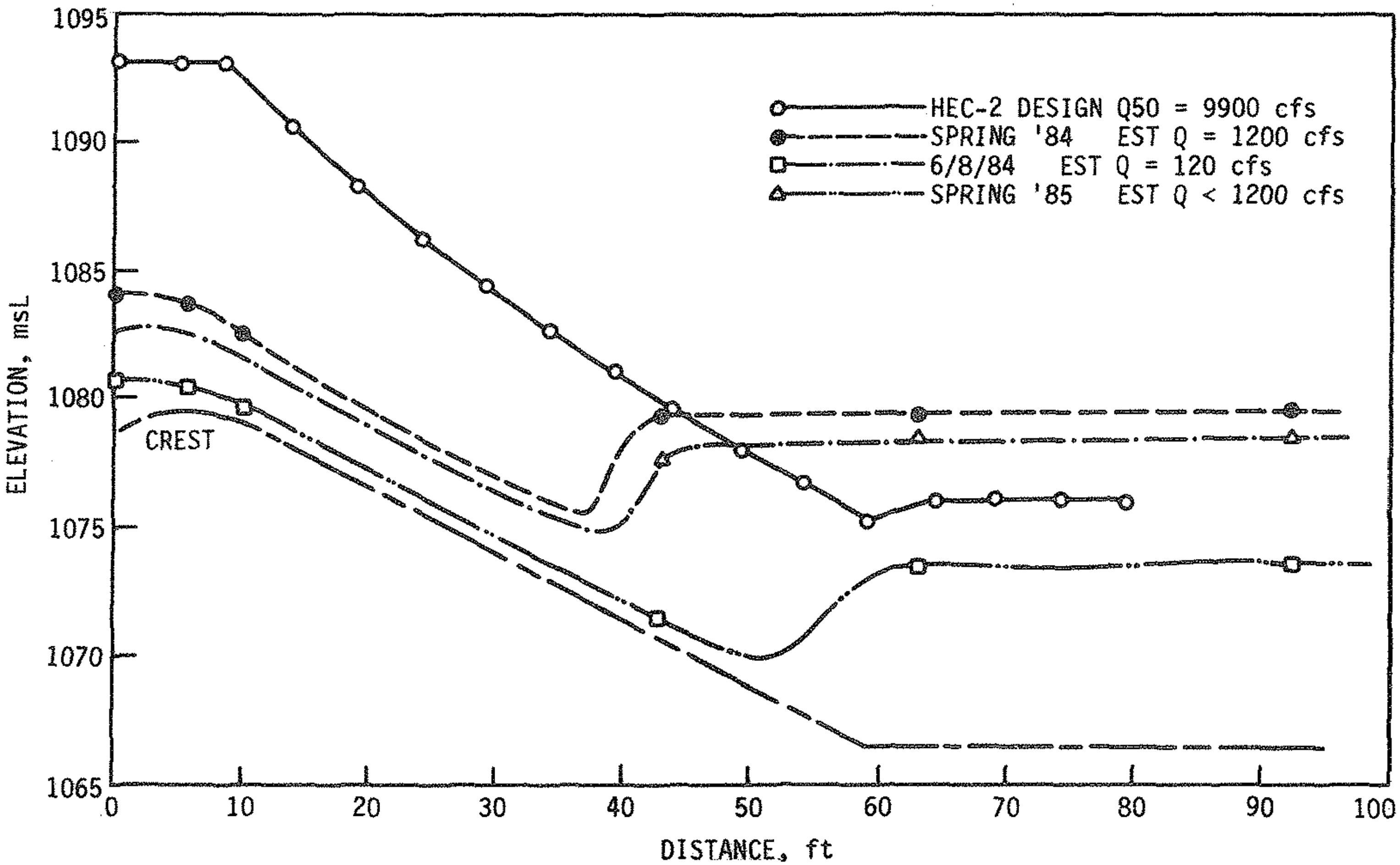


Figure 5. Water surface profiles from HEC-2 for design and as estimated from debris lines at various dates.

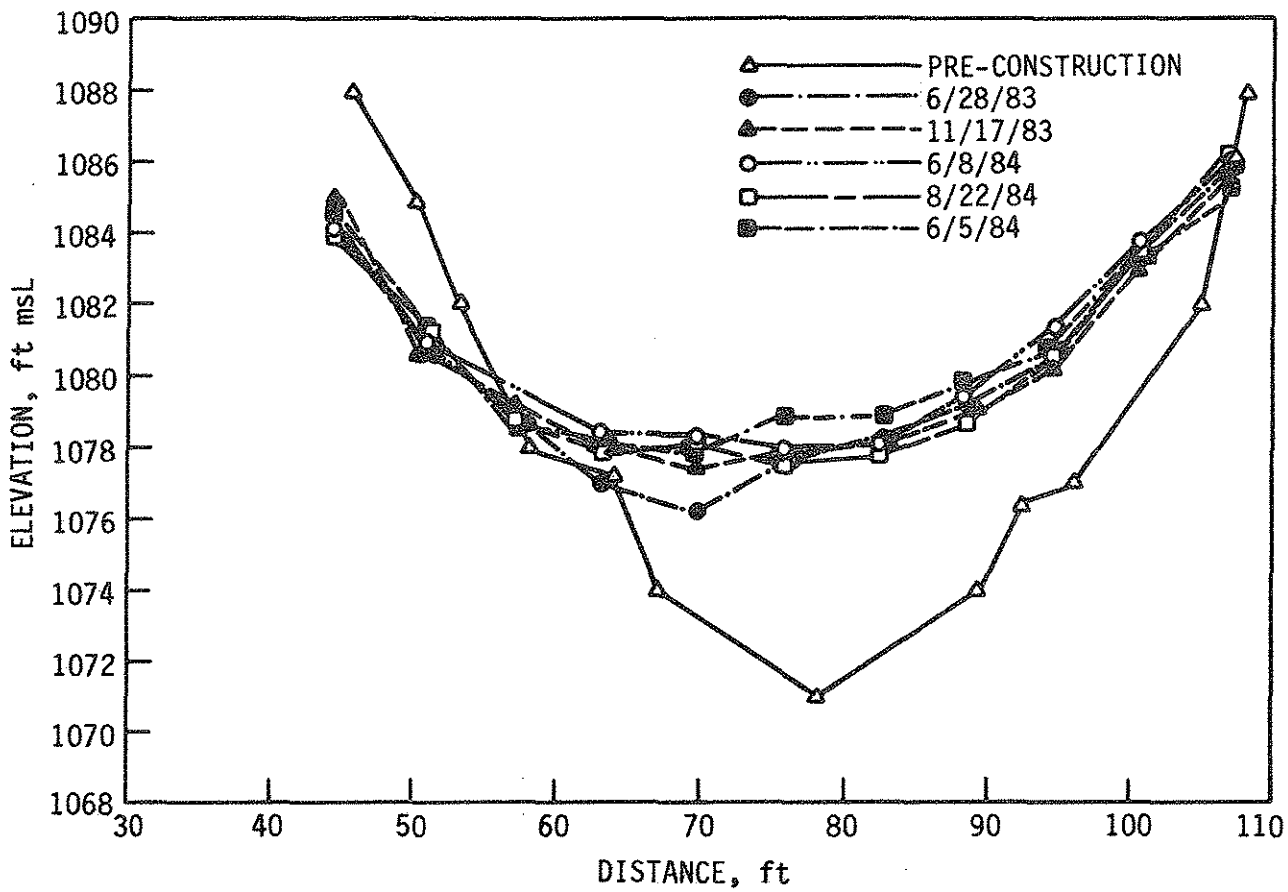


Figure 6. Transverse cross sections of channel bottom for various dates.

This indicates that the major amount of sedimentation above the crest occurred during construction and that the 1,200 cfs event since construction has had little effect on deposition.

A longitudinal profile surveyed on 8/23/84 is shown in Fig. 7. The water surface extends upstream from the crest of the structure to a distance of 5,500 ft upstream, and the sediment surface extends approximately 4,000 ft upstream from the structure. It is expected that this sediment will extend further upstream in the future. A conservative estimate is that it will continue to the point where the water surface profile intersects the stream bed profile, i.e., about 5,500 ft.

A less conservative estimate of the ultimate upstream extent of the sediment is calculated from the method suggested by Maccaferri [1984]. The stable slope of a channel can be estimated from the following equation:

$$i = \frac{(vu_\ell)^{10/3} B^{4/3} n^2}{Q^{4/3}}$$

where i is the stable slope; u is the maximum permissible velocity (which depends upon the size of bed material at which bed erosion starts); v is the ratio between mean water velocity and the corresponding velocity at the channel bottom; u_ℓ is the maximum permissible velocity developing on size of bed material; B is the wetted perimeter, n is the roughness coefficient, and Q is the design flow. This relationship is an extension of Manning's equation, and the detailed analysis with

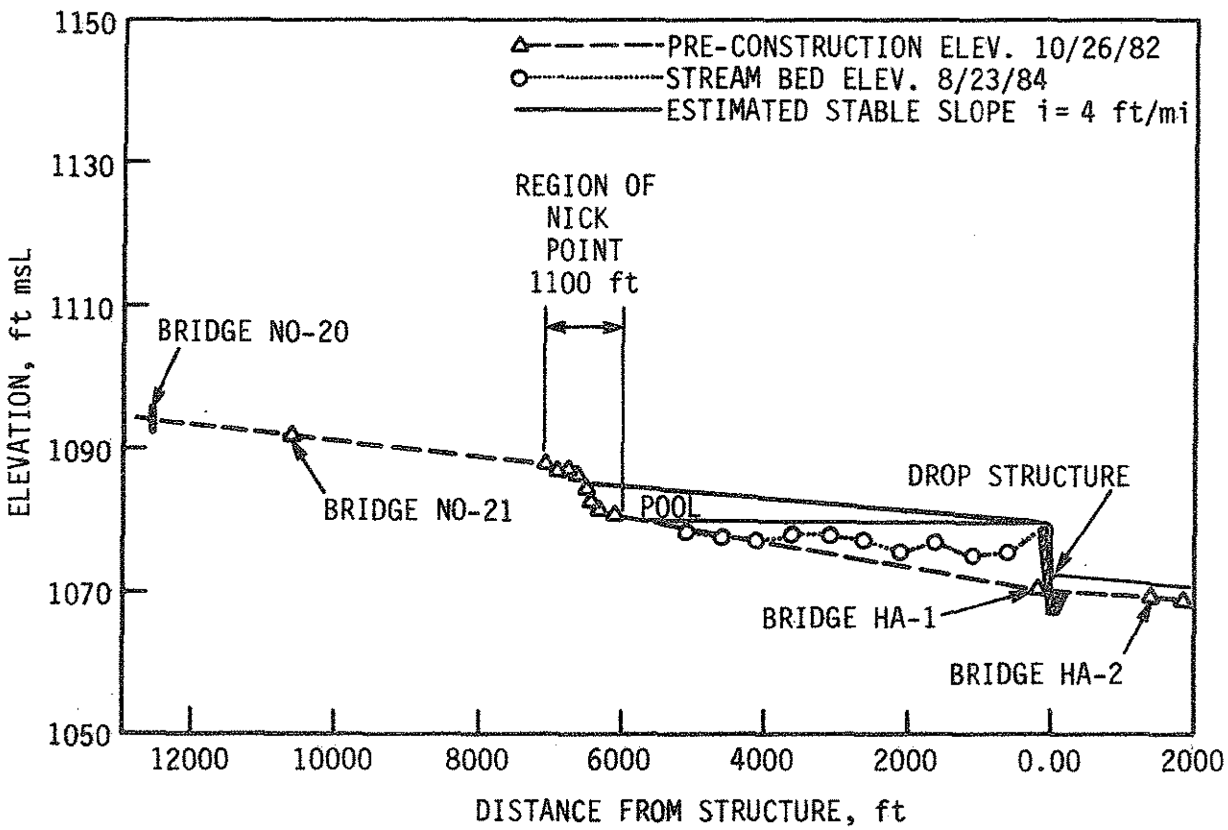


Figure 7. Longitudinal profile of channel bottom and water surface in pool upstream of structure.

application to the gabion structure is in Appendix 3. That analysis estimated a stable slope of 4 ft/mi, which would cause the sediment to extend upstream for a distance of about 6,500 ft. That slope is also plotted on Fig. 7 where it can be seen that the wedge of sediment would intersect the knickpoint at about midheight.

Even though the sedimentation effects of the structure may extend 6,500 ft upstream, there is field evidence that the channel banks are barely stable and that sloughing of the side slopes may cause further loss of land and damage to roads. Also, the upstream knickpoint was not submerged by the grade control structure, so it is likely that the knickpoint will continue to progress upstream.

DOWNSTREAM EROSION

Bank erosion is occurring downstream beyond the stilling basin to a distance of approximately 80 ft. This may be partially because of the submerged jump, which was discussed in a previous portion of this report. The submerged jump provides a relatively inefficient energy dissipation, may be cause for future concern, and may require an extension of the stilling basin to provide better energy dissipation.

ECONOMIC COMPARISON OF GABION STRUCTURE WITH CONCRETE STRUCTURES

The major objective of this research was to find low-cost alternatives for the stabilization of degrading streams. It is difficult to compare the cost of the gabion drop structure, which was built and evaluated as part of this study, with the reinforced concrete structures which have been used in the past, because of the variations in the size of the structures and the drainage areas that contribute flow to them. Table 1 shows the costs of four reinforced concrete drop structures, which were constructed in western Iowa within the last seven years, and the gabion structure. These data are the 1982 costs based on Iowa DOT's construction index; the calculations of these costs can be found in Appendix 4. Although the gabion structure is less than one-third the cost of the least expensive concrete structure, it also has the smallest drop. On the other hand, it does have the second largest drainage area and the largest design flow of the structures listed. The following analysis is an attempt to normalize the cost of the drop structures with design flow, drainage area, channel slope, and structural dimensions.

Design discharge and structure width can be combined to provide a flow area at the crest based on the assumption that critical flow occurs at the crest of the structure [Henderson 1966]:

$$A = \left(\frac{Q^2 B}{g} \right)^3$$

where A is the area of the wetted section at the critical depth, Q is the design flow, B is the corresponding width of the water surface, and g is the acceleration due to gravity. The wetted section, A, has been calculated for the five drop structures and the data shown in Table 2.

Table 1. Geometry, design discharge, and cost of Iowa grade control structure.

County	Creek	Drainage Area (sq mi)	Slope (ft/mi)	Drop (ft)	Length (ft)	Width (ft)	Design Q (cfs)	Cost ($)
Harrison	Willow	67.2	9.3	38.4	142	67.5	5,800	376,022
Monona	Willow	32	19.5	36.6	142	67.5	7,500	372,447
Harrison	Willow	100.2	8.3	24.0	115	80.0	7,240	434,562
Harrison	Pigeon	56.5	8.0	18.6	110	80.0	8,100	345,147
Pott.	Keg	90	8.0	12.6	131	*	9,930	101,000

*Stilling basin is trapezoidal with an average width of 51 ft.

Table 2. Dimensional analysis and size factor for Iowa grade control structures.

County	Creek	A* (ft^2)	a† (mi^2/ft^2)	Sc‡ (ft/mi)	Ss§ (ft/ft)	aScSs** (mi/ft)
Harrison	Willow	413.1	0.163	9.38	3.70	5.63
Monona	Willow	490.7	0.065	19.5	3.88	4.92
Harrison	Willow	507.3	0.198	8.33	4.79	7.90
Harrison	Pigeon	546.3	0.103	8.00	5.89	4.85
Pott.	Keg	602.0	0.100	8.00	10.35	12.43

*A = area of wetted section at critical depth in ft^2.

†a = drainage area in mi^2 divided by area of wetted section in ft^2.

‡Sc = channel slope in ft/mi.

§Ss = structure length divided by drop in ft/ft.

**AScSs = product of the three numbers.

The wetted area can be combined with drainage area, D.A., to form a semi-dimensionless term:

$$a = D.A./A$$

The term is semi-dimensionless because the drainage area is in sq mi and the wetted area is in sq ft. The values of "a" for all five structures are also shown in Table 2. The channel slope, Sc, is a semi-dimensionless term in ft/mi, and a dimensionless term, Ss, can be generated by dividing the overall length of the structure by its drop. These terms, along with their combined values, are shown in Table 2. This combined term describes the structure according to size, design flow, and drainage area and is defined here as the size factor. The cost of each structure is plotted versus the size factor in Fig. 8; it can be seen that the cost of the concrete structures increases linearly with increasing size factor. Note that the cost of the plotted gabion structure versus its size factor falls considerably below the projection of the line for the concrete structures. This analysis suggests that the gabion structure may be about 20% of the cost of an equivalent reinforced concrete structure.

CONCLUSIONS

The gabion grade stabilization structure has shown satisfactory structural performance throughout the two-year observation period, with minimal differential settling and no evidence of side slope

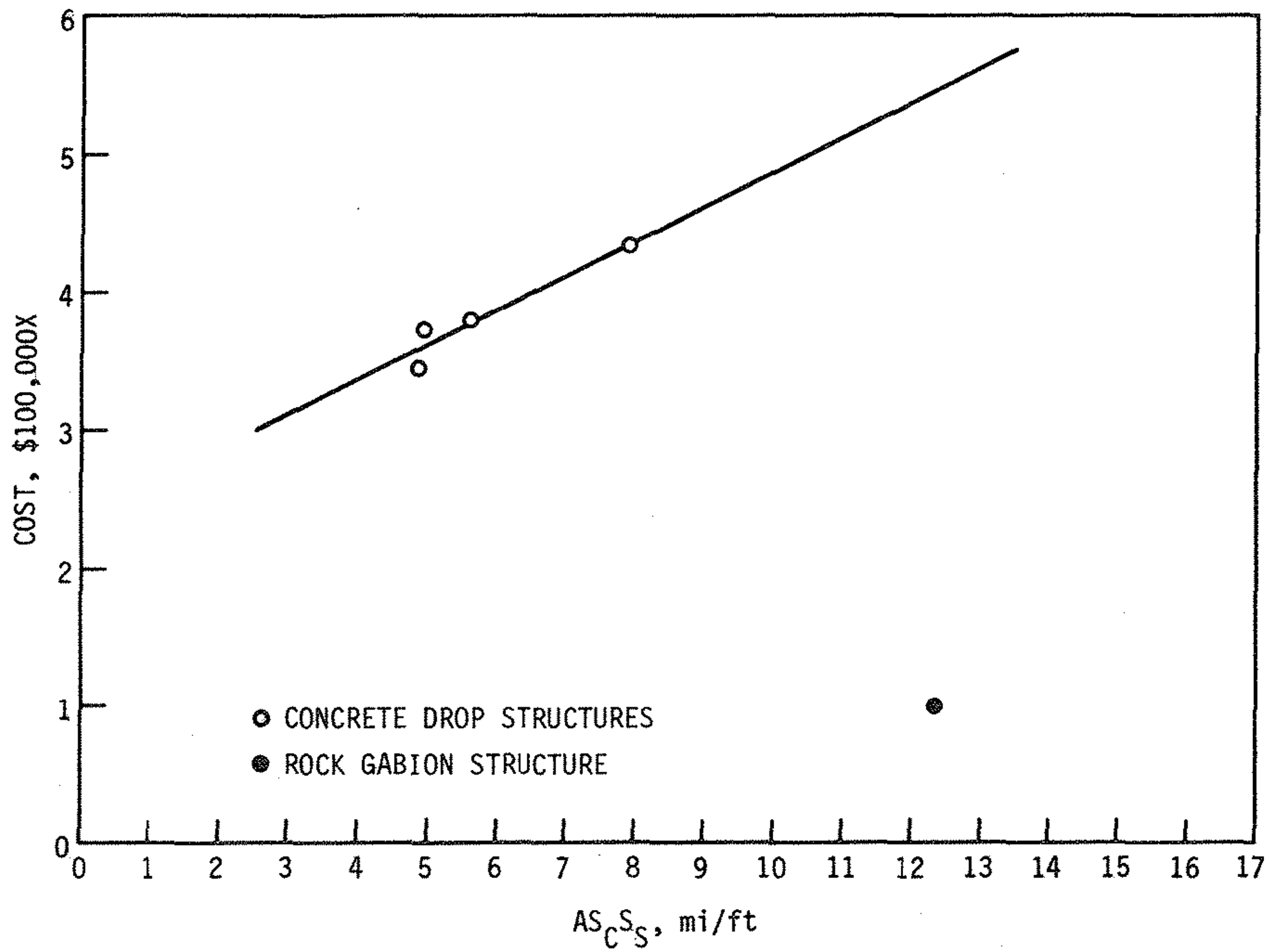


Figure 8. Plot of dimensionless size factor versus construction costs for four concrete drop structures and one gabion drop structure.

instability since construction was finished. It should be recognized that the maximum flow to date has been less than 15% of the design flow.

The major amount of sedimentation occurred during construction and is likely to extend at least 5,500 ft upstream of the structure. A more optimistic estimate is that the depositional wedge will extend 6,500 ft upstream. In any event the sedimentation effects of the structure will not submerge the knickpoint that exists upstream, so continued upstream erosion problems are likely upstream of the sedimentation area.

The sedimentation beneath the bridge has been sufficient to cover the piles to their original depth of soil cover and to stabilize the slope beneath the abutment. This is illustrated by comparing Photoplates 4 and 5. Photoplate 4 was photographed in 1979 when erosion had exposed about 6 ft of previously buried piling and pulled soil away from the abutment. Photoplate 5 was taken in June, 1985 and shows that the piles have been covered to their original depth.

Erosion downstream of the structure could be a problem, especially if it undermines the stilling basin. On the other hand, the gabions are deformable and may collapse into any scour hole that forms, thereby becoming somewhat self protecting. This downstream erosion is the result of inefficient energy dissipation by the stilling basin.

An analysis of the cost of the gabion structure as compared with costs of four concrete structures included the size, drainage area, and design flow of each of the structures. This analysis suggests that

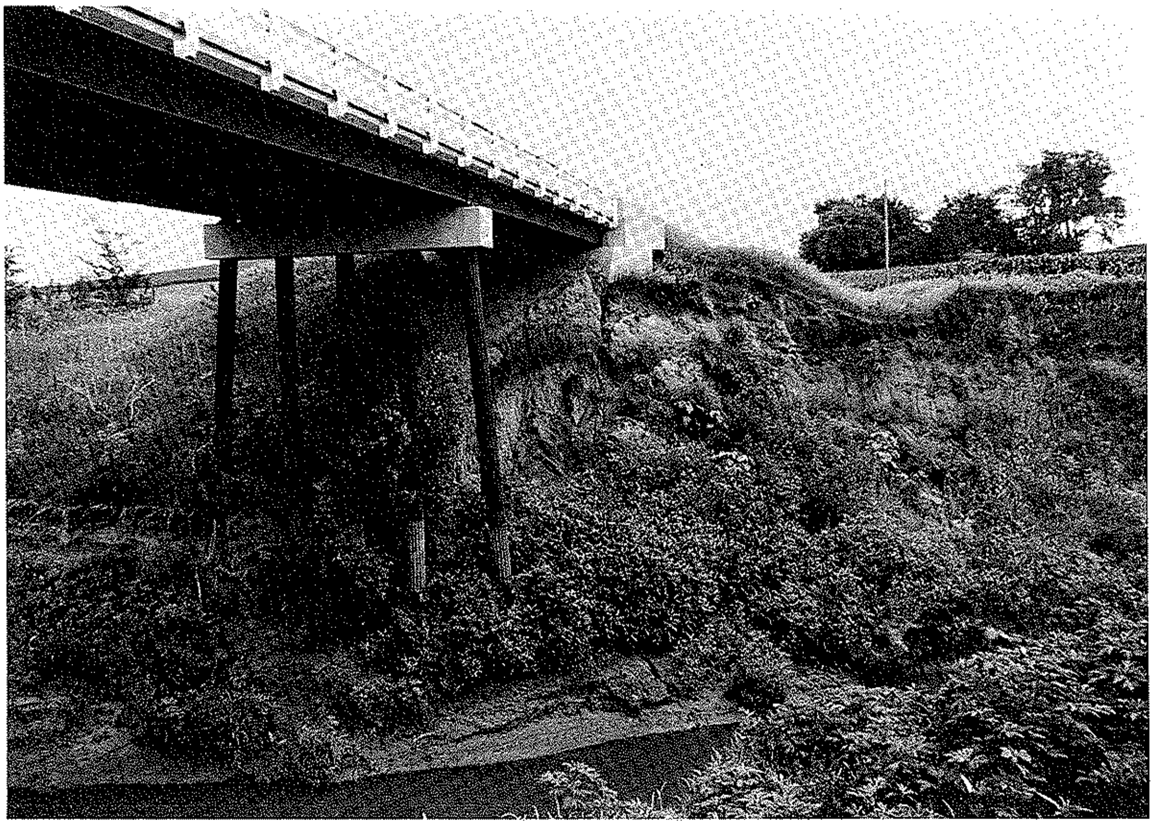


Plate 4. East end of bridge showing exposed piling and soil slide away from abutment. Light colored portion of the bottom of piles indicates where soil formerly covered piles, July 1979.

Plate 5. East end of bridge showing piling once again covered by soil and stable soil slope beneath abutment, June 1985.

the gabion structure cost about 20% of what an equivalent concrete structure would have cost.

ACKNOWLEDGMENTS

The authors gratefully acknowledge the financial support provided for this study by the Highway Research Board and the Highway Division, Iowa DOT; by ISWRRI, and by Pottawattami County. The support and technical expertise of Gene Hales, Pottawattami County Engineer; Brian Hunter, Assistant Pottawattami County Engineer; and Eldo Schornhorst, Shelby County Engineer, were most valuable in advancing the project. T. Al Austin, Director of ISWRRI, provided important guidance in the hydraulic design. Former ISU graduate students G. Tom Wade and Lester Litton collected the data for Phase I of this research, and many of their ideas and suggestions were carried into the design phase.

The Engineering Research Institute administered this project, and The Office of Editorial Services prepared this report; their support is also gratefully acknowledged.

REFERENCES

Brice, J. C. et al., 1978, "Countermeasures for Hydraulic Problems at Bridges," Federal Highway Administration Report No. FHWA-RD-78-163, Vols. 1 and 2, National Technical Information Service, Springfield, Va.

Henderson, F. M., 1966, Open Channel Flow, MacMillan Pub. Co., New York, 352 pp.

Litton, L. L. and R. A. Lohnes, 1982, "Soil-cement for Use in Stream Channel Grade Stabilization Structures," Transportation Research Record 839:33-40.

Lohnes, R. A., F. W. Klaiber, and M. D. Dougal, 1980, "Alternate Methods of Stabilizing Degrading Stream Channels in Western Iowa, Final Report on Phase I," ISU-ERI-Ames-81047, Engineering Research Institute, Iowa State Univ., Ames, Ia.

Maccaferri, 1984, Flexible Weirs for River Training, Maccaferri Gabion Co., Rome, Italy, 56 pp.

APPENDIX 1

CONSTRUCTION OF GABION STRUCTURES

Plate A-1. Excavation and grading of side slopes, Dec. 18,1982.

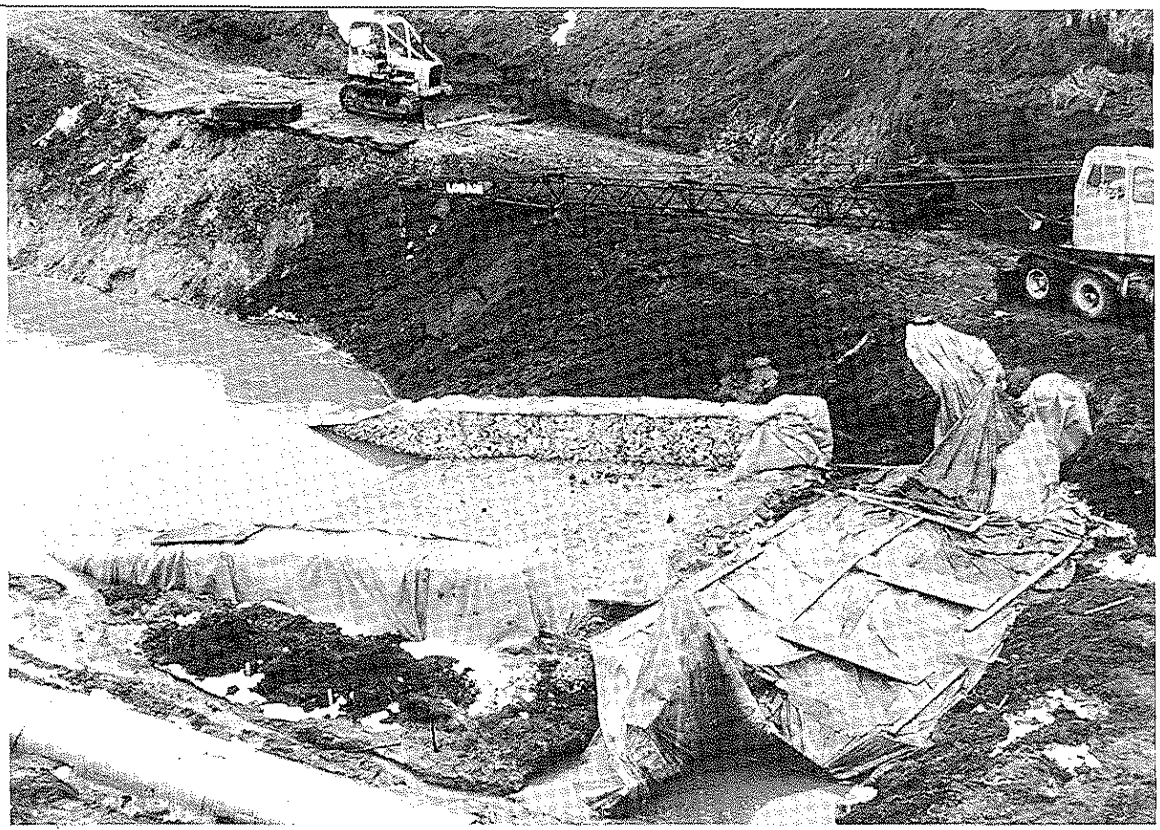


Plate A-2. Gabion spillway in place, Feb. 10,1983.

Plate A-3. Diversion channel excavated on west side of natural channel, Apr. 9, 1983.

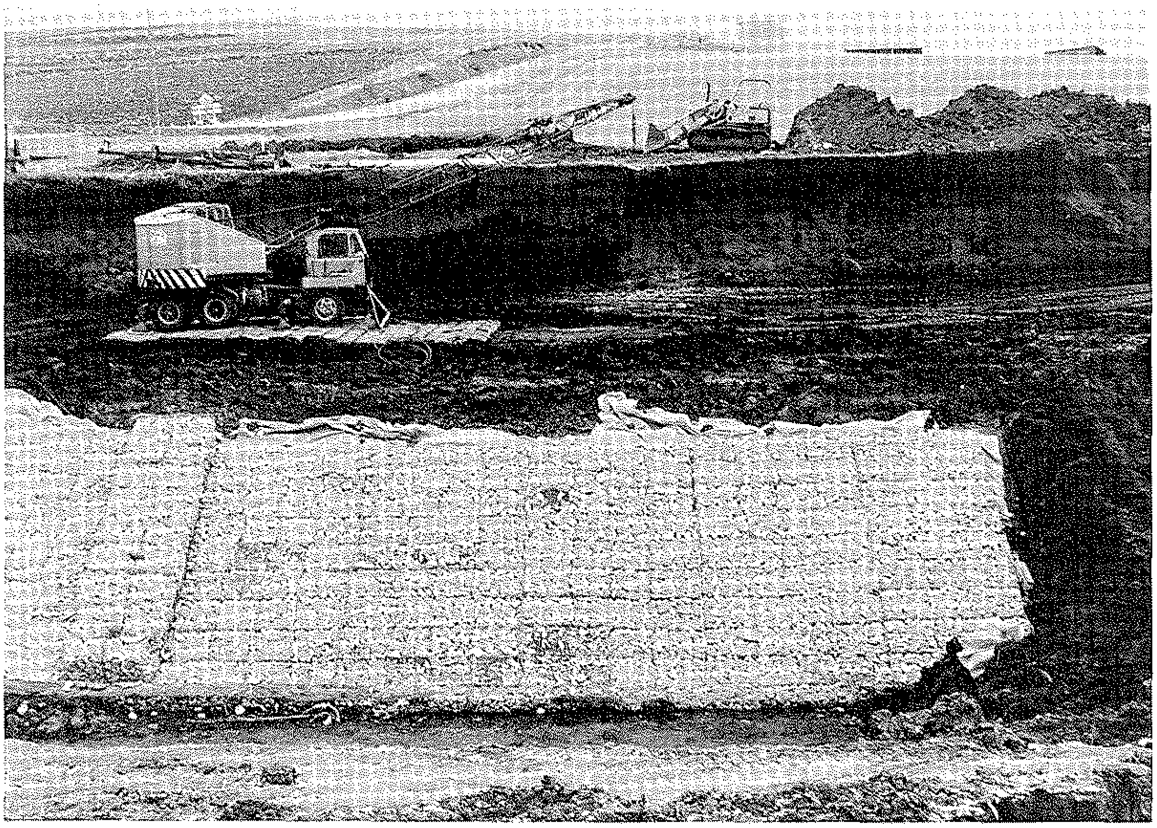


Plate A-4. Gabions in place on east side of spillway, Apr. 27, 1983.

Plate A-5. Concrete cutoff wall at crest of structure, Apr. 27, 1983.

Plate A-6. Dewatering of stilling basin after a high flow event, May 10, 1983.

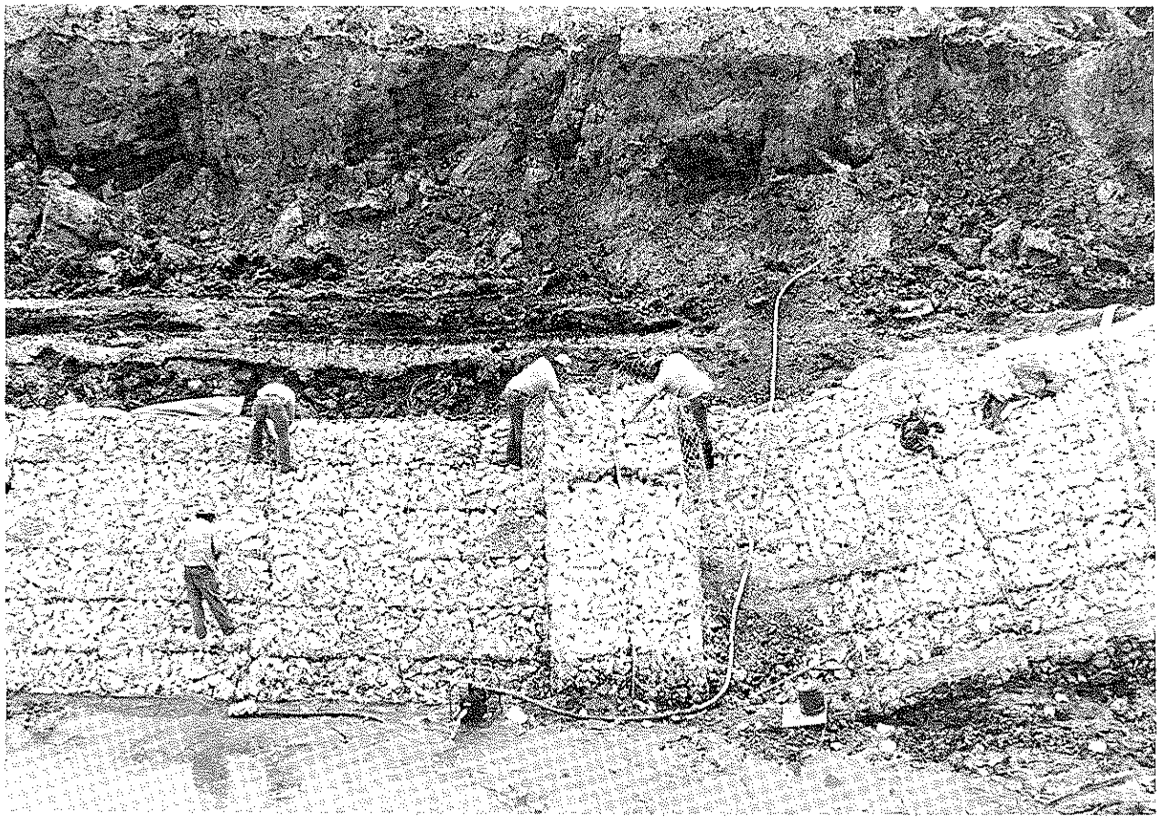


Plate A-7. Construction of evergy dissipators on west side slope, May 10, 1983.

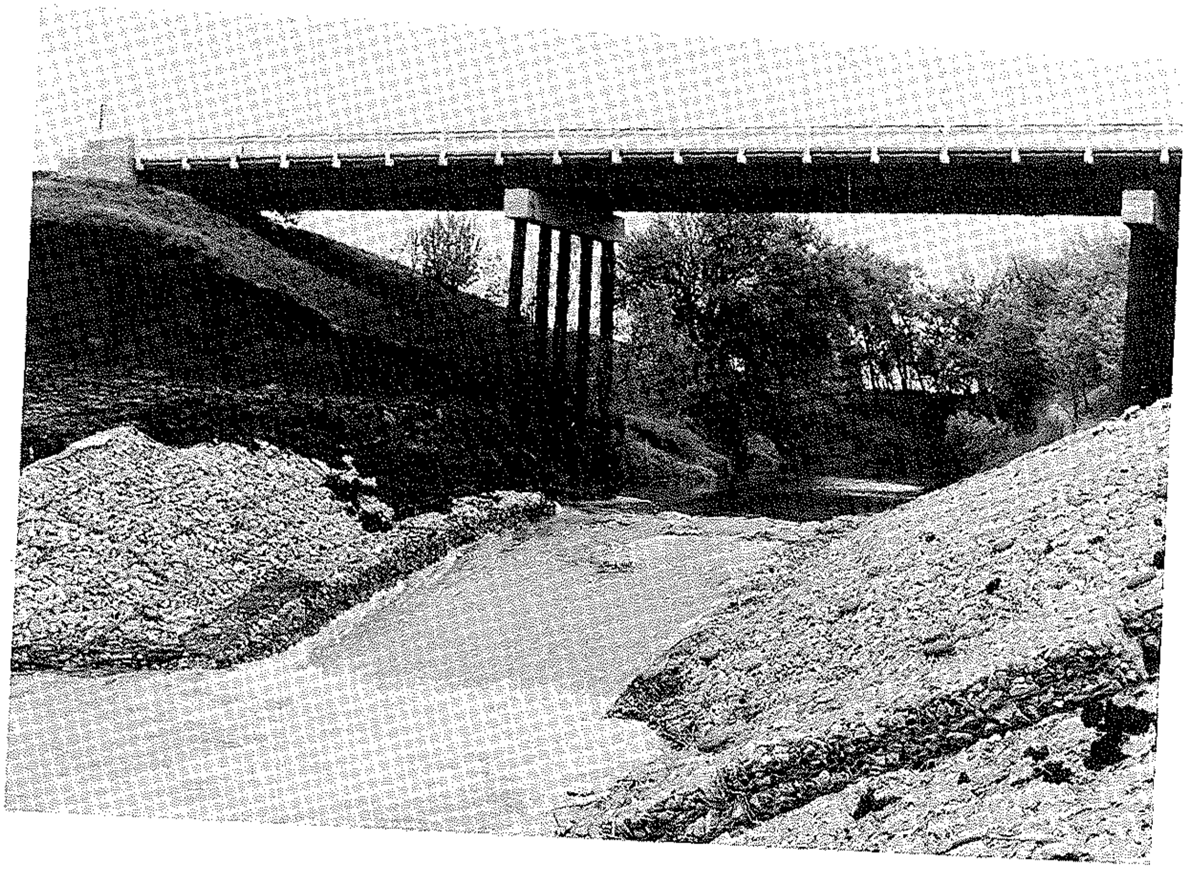


Plate A-8. Main channel flow through the structure. At this time, only work remaining is grading above side slopes, May 18, 1983.

APPENDIX 2

DISCHARGE COMPUTATIONS AND CRITICAL FLOW

Controls are certain features in a channel that tend to produce critical flow and are therefore special.

$$FR = v/\sqrt{gy}$$

where

$dz/dx = 0$, $dy/dx \neq 0$, and

$Fr = 1$ for critical flow.

"Critical" is used to describe a state at which the specific energy E is at a minimum for a given q.

The equation defining critical flow for rectangular sections is

$$E = y + \frac{q^2}{2gy^2}$$

It follows from the definition of a control that at any feature which acts as a control, the discharge can be calculated once the depth is known.

The first question to resolve is the form of the specific energy equation; if we take our datum level at the lowest point on the section and measure the depth upward from this level, we find that for every point in the cross section the sum of the pressure head and potential head is still equal to the depth y, just as for the rectangular section. In other words, the irregularity of the section does not affect the hydrostatic pressure distribution. Therefore, the specific energy equation may be written as follows:

$$E = y + \frac{v^2}{2g}$$

However, to explore the E vs. y relationship, we can no longer use the discharge per unit width (q) relationship. We must use

$$E = y + \frac{Q^2}{2gA^2}$$

where

Q = total discharge

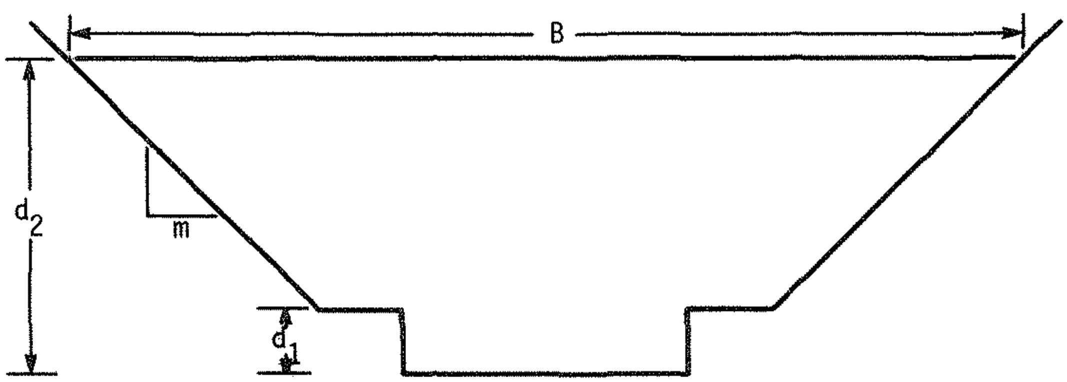
A = area of X-section

We find the condition of minimum specific energy by the differentiation

$$\frac{dE}{dy} = 1 - \frac{Q^2}{gA^3}\frac{dA}{d\gamma}$$

To assign a meaning to $dA/d\gamma$, we consider the effect of the area, A, on small changes in depth.

$$\frac{dA}{d\gamma} = \begin{cases} \frac{2B - 2d\gamma(m)}{2} = (B - md\gamma) \text{ for } d_1 < \gamma < d_2 \\ \\ Bdy \text{ for } 0 < y < d_1 \end{cases}$$



$$\frac{dE}{d\gamma} = 1 - \frac{Q^2(B - md\gamma)}{gA^3}$$

$$Q^2(B - md\gamma) = gA^3 \text{ as } d\gamma \to 0 \; Q^2B = gA^3$$

$$v_c^2(B - md\gamma) = gA$$

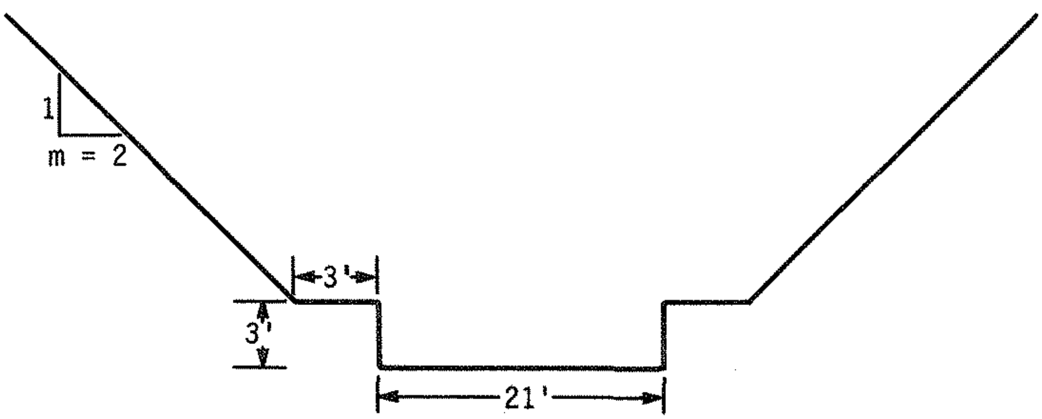
$$v_c^2 = \frac{gA}{(B - md\gamma)} \text{ as } d\gamma \to 0 \; v_c^2 = g\,\frac{A}{B}$$

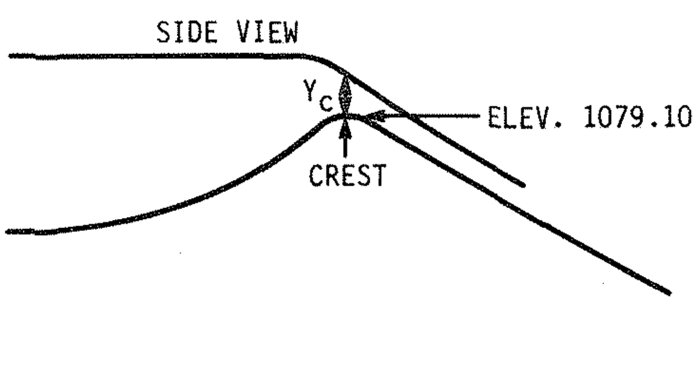
$$Q = A\sqrt{\frac{gA}{B}}$$

A = is the area of wetted section (sq ft)

B = is the corresponding width of the water surface (ft)

γ = acceleration due to gravity (32.2 ft/sec)





The following table shows estimates of flow at Keg Creek control structure x-section at the crest of spillway.

Elev.	y_c(Depth) ft	A(ft^2)	B(ft)	v_c ft/sec	Q ft^3/sec
1079.10	0	0	21	0	0
1080.10	1	21 ft^2	21	5.67	119.2
1081.10	2	42	21	8.02	337.1
1082.10	3	63	21	9.83	619.2
1083.10	4	92	31	9.78	899.4
1084.10	5	125	35	10.72	1340.5
1086.10	7	203	43	12.33	2502.9
1088.10	9	297	51	13.69	4067.0
1090.10	11	407	59	14.90	6065.9
1092.10	13	533	67	16.0	8530.6
1084.10	15	675	75	17.0	11490.9

APPENDIX 3

SEDIMENTATION ANALYSIS

DETERMINATION OF EFFECT OF ROCK GABION DROP STRUCTURE ON RIVER BED UPSTREAM

The stable slope of sediment deposited can be calculated using formula (1)

$$i_e = \frac{(vu_\ell)^{10/3} B^{4/3} n^2}{Q^{4/3}} \tag{1}$$

where

i_e = stable slope

u_ℓ(m/sec) = maximum permissible velocity, depending on the size of bed materials at which the erosion of river bed starts. The suggested value of u_ℓ for alluvial silts is 1.52 m/sec.

v = the ratio between the mean velocity of water and the corresponding velocity at the river bottom: This ratio is nearly equal to 1.3 - 1.5.

B(m) = wetted perimeter, which can be generally considered equal to the width of the river.

n($m^{-1/3}$ sec) = coefficient of roughness of the river. Manning's n. n is chosen to be 0.040 for Keg Creek.

Q(cumecs) = flood discharge according to which the river training is designed; the rock gabion structure was designed for 50-year return period.

Equation (1) is derived from Manning's resistance formula.

$$v = \frac{R^{2/3}S^{1/2}}{n}$$

$$\therefore\ s^{1/2} = \frac{vn}{R^{2/3}}$$

$$s = \frac{v^2n^2}{R^{4/3}} = \frac{v^2n^2}{\frac{A}{B}^{4/3}} = \frac{v^2n^2B^{4/3}}{A^{4/3}}$$

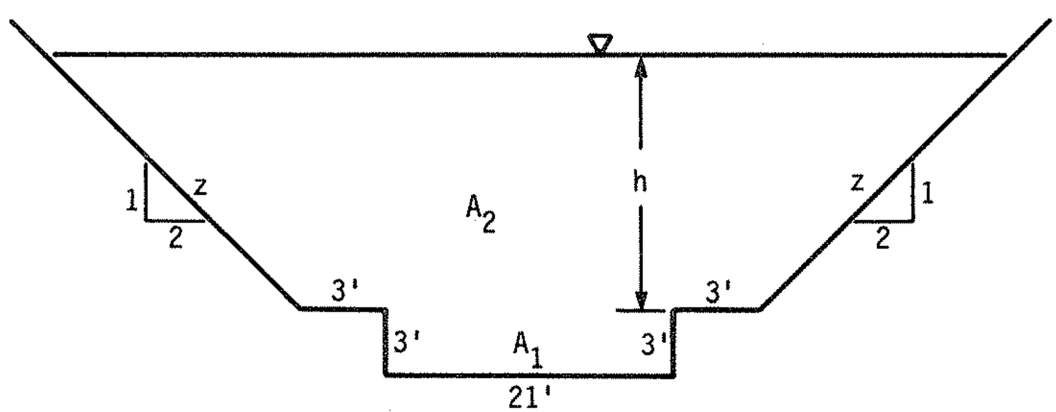
$$s = \frac{v^{4/3}}{v^{4/3}} \cdot \frac{v^2n^2B^{4/3}}{A^{4/3}} = \frac{(v)^{10/3}B^{4/3}n^2}{Q^{4/3}}$$

$$v = V \cdot u_\ell = (r0.22)1.4 \cdot (1.52 \text{ m/sec}) = 1.66 \text{ m/sec}$$

$$Q_{50} = 9930 \text{ cfs} = 281.2 \text{ m}^3\text{/sec}$$

$$A = \frac{Q}{V} = \frac{281.2 \text{ m}^2\text{/sec}}{1.00} = 169.4 \text{ m}^2 \text{ or } 1822.72 \text{ ft}^2$$

Calculations for @ A = 1822.72 ft^2.



$$A_1 = 3' \times 21' = 63'\ ft^2$$

$$A_2 = 1760/ft^2$$

$$A_T = \frac{B + T}{2} \times h$$

$$T = B + 2(Zh)$$

$$A_T = \frac{B + B + 2(Zh)}{2} \propto h = (B + Z \times g) \times h$$

$$A_T = Bh + Z \times h^2 = Bh + 2.0 \times h^2$$

$$h = 23.7\ ft$$

$$B = 1390\ ft\ or\ 42.34\ m$$

h	A
2.0	62
10.0	470
20.0	1340.0
30.0	2610.0
24.0	1800
23.7	1763.3

$$s = \frac{(1.66\ m/s)^{10/3}(42.34\ m)^{4/3}(0.040)^2}{(281.2\ m^3/sec)^{4/3}} = 0.00069\ m/m$$

$$= 0.0069\ ft/ft = 0.069\ ft/100\ ft = 3.66\ ft/mi \sim 4\ ft/mi$$

When the above parameters along the whole length of the reach in question are known, the stable slope is calculated, after which the position and height of the structures must be determined.

If a stretch of a river, having natural slope i, is to be trained to a slope i_e by means of a series of weirs at equidistant points, the height H and distance l between two weirs are connected by the relation: $H = H_1 - H_2 = (i - i_e)l$. Consequently, the number n of weirs necessary for the training of the considered length L is

$$n = \frac{L}{l} = \frac{L(i - i_e)}{H}$$

In general, it is preferable to build small and closely separated structures instead of high ones, particularly where the soil is subject to erosion, in order to disturb the natural watercourse as little as possible.

Recently, in addition to traditional forms of weirs constructed for the prevention of river bed erosion, other types of structures have been developed for this particular purpose, but they have different characteristics.

APPENDIX 4

COST ANALYSIS OF DROP STRUCTURES IN WESTERN IOWA

Design No. 371
File No. 23464
S-15(4)--50-43

Harrison County

Description: 67'-6" × 28' I. BM. Cr. & Flume

Location: Sec 20/29, T81, R42W; on North Line N.E. 1/4 Lincoln Twp.

River: Willow Creek

D.A.: 67.2 sq mi

Design Discharge: 5700 cfs Q_{20} = 6600 cfs Q_{25} = 5800 cfs

Slope: 10.1 ft/mile

Size of Drop: 987.71 - 949.28 = 38.43 ft

Length of Drop Structure: 142 ft

Width of Structure: 67.5 ft

Actual Letting Date: 7/20/71

Est. Cost.: $329,000.00

Actual Original Contract: $175,529.09

Est. Price of Bridge @ 1982 prices = (67.5' × 28') × $40/sq ft = $75,600

Est. Price of Bridge @ 1971 prices = 57.6/148.2 · $75,600 = $29,383.00

Cost of Flume @ 1971 prices = $175,529.09 - $29,383.00 = $146,146.09

Cost of Flume @ 1982 prices = 148.2/57.6 · $146,146.09 = $376,021.71

1982 Index costs were used because that is the year the rock gabion structure was let.

W/D = 67.5/38.43 = 1.76 L/W = 2.10 L/D = 3.70

W · D · L = 368351

W · D = 2594

DA/W · D = 722,216

Design No. 368
File No. 23336
SN-752( )--51-67

Monona County

Description: 67'6" × 28' PCBMBr with flume

Location: T82N; R42W; Sec 22: Civil Twp Willow

River: Willow Creek

D.A.: 32 sq mi

Design Discharge: 7500 cfs; Ave. Velocity

Slope: 19.5 ft/miles

Wood & Concrete Flume & Invert

Size of Drop: 1205.23 - 1168.65 = 36.58 ft

Length of Drop Structure: 142 ft

Width of Structure: 67.5 ft

Year of Estimate: 3/27/68 (Est $130,000)

Est. Cost of Bridge @ 1982 prices = (67.5' × 28') × $40/sq ft = $75,600

Est. Cost of Bridge @ 1968 prices = 43.0/148.2 × $75,600.00 = $21,935.22

Est. Cost of Flume @ 1968 prices = $130,000 - $21,935.22 = $108,064.78

Est. Cost of Flume @ 1982 prices = $108,064.78 × 148.2/43.0 - $372,446.50

W/D = 67.5/36.58 = 1.85 L/W = 2.10 L/D = 3.88

W · D · L = 350619.3

W · D = 2469

D.A./W · D = 3,611,323

Design No. 372
File No. 23464
S-212(1)--50-43

Harrison County

Description: 80' × 28' I-BM.Br with Conc & Wood Flume

Location: Section 23, T80N, R43W; Magnolia Twp. (on North Line of S.W. 1/4)

River: Willow Creek

D.A.: 100.2 sq mi

Design Discharge: 7250 cfs

Slope: 17.6 ft/mi

Wood & Concrete Flume & Invert.

Total Low Bid Price Was: $213,626.12

Size of Drop: 982.69 ft - 958.73 ft = 23.96 ft

Length of Drop Structure: 115 ft

Width of Structure: 80 ft

Actual Letting Date: 9/12/72

Est. Price of Bridge @ 1982 prices = $(80' \times 28') \times \$40/ft^2 = \$89{,}600$

Est. Price of Bridge @ 1972 prices = $60.4/148.2 \cdot \$89{,}600 = \$36{,}517.14$

Est. Price of Flume @ 1972 prices = $213{,}626.12 - \$36{,}517.14 = \$177{,}108.98$

Est. Price of Flume @ 1982 prices = $148.2/60.4 \cdot \$177{,}108.98 = \$434{,}562.10$

$W \cdot D = 80'/23.96 = 3.34$ $L/W = 1.44$ $L/D = 4.80$

$W \cdot D = 1916.8$

$DA/W \cdot D = 1{,}457{,}332.9$

Design No. 378
File No. 5110S
FM-43(4)--55-43

Harrison County

Description: 80' × 30' I-Beam Br. & Flume

Spans: 80' 0° skew

Sec 33 Twp 78N

Range: 42W (on secondary road in the NE 1/4 SE 1/4)

River: Pigeon Creek; Union Twp.

D.A.: 56.5 mi^2; Slope could not be determined but estimated @ 8.0 ft/mile
$\therefore Q_{50} = C_T(A)X_T = 8100$ cfs

Wood & Concrete Flume & Invert

Total Low Bid Price Was: $417,929.30

Size of Drop: 987.20 - 968.56 - 18.64 ft

Length of Drop Structure: 109.50 ft

Width of Drop Structure: 80.0 ft

Design Discharge = 8100 cfs

Ave. Vel.: 6.1 F.P.S.

Est. Cost of Bridge @ 1982 prices = (80' × 30') × $40/$ft^2$ = $96,000

Est. Cost of Bridge @ 1979 prices = 140.4/148.2 · 96,000 = $90947.37

Est. Cost of Flume @ 1979 prices = $417,928,30 - $90,947.37 = $326,981

Est. Cost of Flume @ 1982 prices = 148.2/140.4 · $326,981 = $345,146.54

W/D = 80'/18.64 = 4.29 L/W = 1.37 L/D = 5.87

W · D · L = 163286.4

W · D = 1491

DA/W · D = 1,056,424

Contract payments for Research Project HR-236:

Original Contract	$ 97,287.03
Extra work order #1 ($2,375.04 - $500.00)	1,875.04
Extra work order #2	5,507.28
Additional contract dirt	6,307.20
Borrow for additional dirt	99.85
Rock on borrow drive	139.50
Total amount paid to date	$ 64,162.94
Settlement owed to contractor	4,500.00
TOTAL COST	$108,080.30

Cost Estimate

1984 Cost of Bridge 40 per sq ft or also $100/linear ft

Highway Construction Costs

Calendar Year		Iowa Cost Index
1961		36.3
1962		39.6
1963		42.1
1964		41.0
1965		42.1
1966		48.8
1967		46.2
1968		43.0
1969		51.0
1970		56.6
1971		57.6
1972		60.4
1973		75.5
1974		104.5
1975		104.0
1976		95.1
1977	- Base -	100.0
1978		115.0
1979		140.4
1980		148.0
1981		150.5
1982		148.2
1983		166.6