TECHNICAL REPORT H-75-19

FOURMILE RUN LOCAL FLOOD-CONTROL PROJECT, ALEXANDRIA AND ARLINGTON COUNTY, VIRGINIA

Hydraulic Model Investigation

by

Noel R. Oswalt, John F. George, and Glenn A. Pickering

Hydraulics Laboratory
U. S. Army Engineer Waterways Experiment Station
P. O. Box 631, Vicksburg, Miss. 39180

December 1975

Final Report

Approved For Public Release; Distr. Unlimited

Prepared for U. S. Army Engineer District, Baltimore
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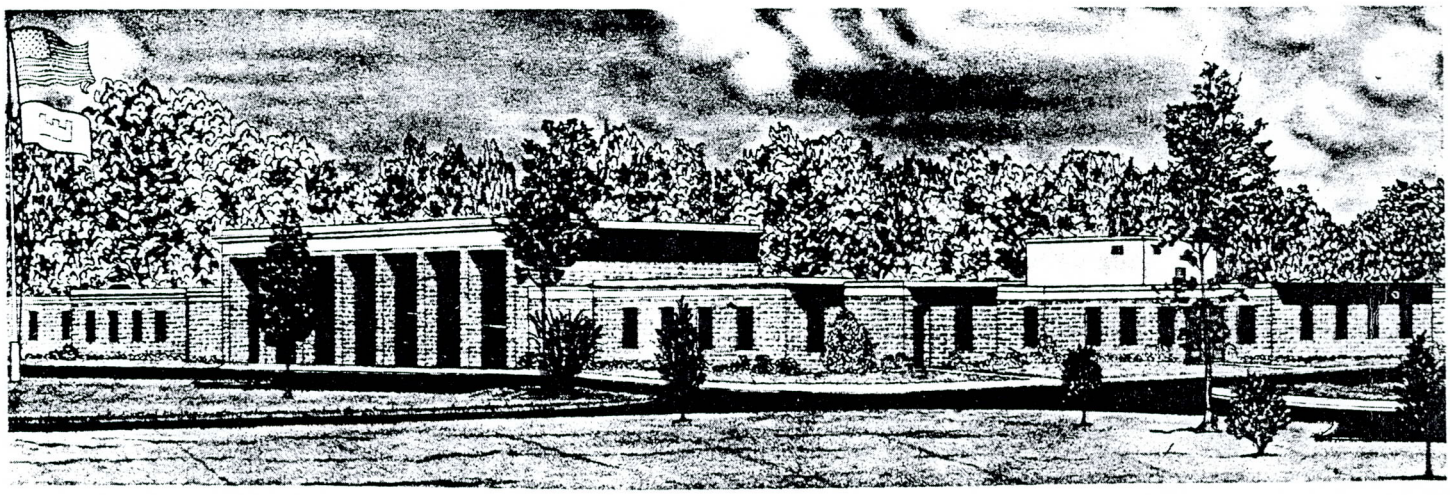

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The Fourmile Run flood-control project will provide protection for the city of Alexandria and Arlington County, Va., from flooding of the Fourmile Run channel. The proposed plan for containing flood flows consists of eliminating existing constrictions in the channel, excavating and widening the channel, providing adequate bank-slope protection, and constructing energy dissipators, grade control structures, and flow dividers. The investigation was conducted

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with a 1:30-scale model that reproduced approximately 5700 ft of the Fourmile Run channel and 400 ft of the Long Branch tributary. The existing bridges and vertical walls, as well as the proposed improved channel with bank-slope protection and hydraulic structures, were reproduced in the model. Tests encompassed flow conditions at the bridges and at the confluence of Long Branch, water-surface elevations, riprap and gabion stability, and performance of the hydraulic structures.

Flow conditions through the hydraulic structures of the original design protection plan from the I-95 Bridge to just downstream of West Glebe Road Bridge were unsatisfactory for design discharges. Efforts to improve the hydraulic performance of these structures were not made because the sponsor considered development of the natural, soft-bottom channel a more feasible solution to problems.

With design discharges in the improved natural soft-bottom channel, surface waves between I-95 and West Glebe Road Bridge caused failure of the riprap on the bank slopes. Failure of the riprap also occurred on the left bank slope downstream of West Glebe Road Bridge and along the left bank slope at the Long Branch confluence with Fourmile Run. Larger riprap was not considered feasible so gabion protection was tested in these areas and found to provide the optimum protection. Proposed gabion and riprap proctection plans were developed for the remaining reach of Fourmile Run and Long Branch.

Discharges exceeding 8000 cfs overtopped portions of the left slope between West Glebe Road Bridge and Long Branch and both bank slopes downstream from Mount Vernon Avenue Bridge. A vertical wall was placed 2 to 5 ft from the crest of the left slope between West Glebe Road and Long Branch. This addition provided the necessary protection for the low topography. Additional protection was unwarranted for the low topography downstream of Mount Vernon Avenue Bridge based on a feasibility study by the sponsor.

The proposed drop structure in Long Branch will lower the water-surface elevation to coincide with that in Fourmile Run and will reduce erosive velocities in the channel. The structure performed satisfactorily for low discharges; however, it was ineffective during peak discharges in Long Branch due to the tailwater effects from the higher flows in Fourmile Run.

PREFACE

The model investigation reported herein was authorized by the Office, Chief of Engineers (OCE), U. S. Army, on 12 April 1973 at the request of the U. S. Army Engineer District, Baltimore. The studies were conducted by personnel of the Hydraulics Laboratory (HL), U. S. Army Engineer Waterways Experiment Station (WES), during the period June 1973 to May 1974, under the general supervision of Messrs. H. B. Simmons, Chief of HL, and J. L. Grace, Jr., Chief of the Structures Division, HL. The tests were conducted by Messrs. N. R. Oswalt, J. F. George, and H. H. Allen under the supervision of Mr. G. A. Pickering, Chief of the Locks and Conduits Branch. This report was prepared by Messrs. George, Oswalt, and Pickering.

Mr. T. Robinson of OCE; Messrs. E. Lally and W. D. Stockman of the U. S. Army Engineer Division, North Atlantic; and COL R. S. McGarry, District Engineer, LTC S. J. Norton, and Messrs. C. R. Strong, C. Kendrew, B. T. Cortright, J. Hemler, E. Veskimets, and M. A. Kolessar of the Baltimore District visited WES during the study to discuss test results and to correlate these results with concurrent design work.

Director of WES during the study and the preparation and publication of this report was COL G. H. Hilt, CE. Technical Director was Mr. F. R. Brown.

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CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
inches	2.54	centimetres
feet	0.3048	metres
miles (U. S. statute)	1.609344	kilometres
square miles	2.589988	square kilometres
pounds (mass)	0.4535924	kilograms
tons (2000 lb mass)	907.1847	kilograms
cubic feet per second	0.02831685	cubic metres per second

FOURMILE RUN LOCAL FLOOD-CONTROL PROJECT,
ALEXANDRIA AND ARLINGTON COUNTY, VIRGINIA

Hydraulic Model Investigation

PART I: INTRODUCTION

The Prototype

1. The Fourmile Run flood-control project is designed to provide fluvial flood protection for the city of Alexandria and Arlington County, Va. The present channel commences in the Brilyn Park area in Fairfax County and flows southeasterly for 9.3 miles* to its confluence with the Potomac River. The proposed improvements of the flood-control project will begin where the Fourmile Run channel enters the Potomac River and extend upstream to the I-95 Bridge (see Figure 1). Other proposed

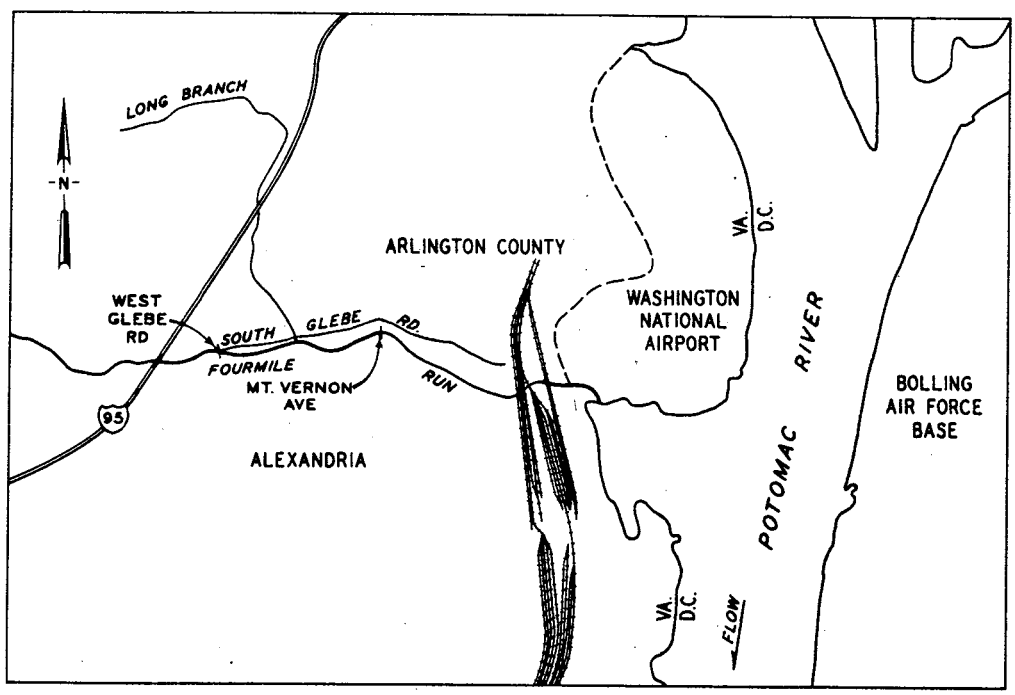

Figure 1. Vicinity map

* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 3.

improvements will be initiated in Arlington County on the Long Branch tributary immediately upstream from its junction with Fourmile Run.

2. The Fourmile Run drainage basin contains an area of 19.7 square miles and is located in a heavily urbanized area. Elevations in the watershed vary from 450* at the northwestern edge to sea level where Fourmile Run enters the Potomac River. The average stream slope for the first 2.2 miles of reach from the confluence of Fourmile Run with the Potomac River is 12 ft per mile. The remaining 7.1-mile reach of Fourmile Run has an average stream slope of 45 ft per mile. The principal tributaries of Fourmile Run are Long Branch, Luck Run, Doctors Run, and Lubber Run. These tributaries have steeper slopes that vary from 67 ft per mile in Long Branch to 117 ft per mile in Luck Run.

3. The existing Fourmile Run channel is a well-defined stream consisting of erodible material in the channel invert, as well as in the side slopes. The channel is typically an irregular trapezoidal shape with the bottom width varying from 70 ft at the I-95 Bridge to 180 ft at sta 454+00. The estimated average annual sediment load in Fourmile Run is 1000 tons per square mile, comprised of equal parts of sand, clay, and gravel. The majority of the gravel is deposited in the channel while the major portion of the clay and sand is carried to the Potomac River.

4. The Long Branch tributary is a well-defined stream 3.3 miles long and is comprised of the same erodible material as Fourmile Run. The drainage basin is highly developed and covers an area of 2.7 square miles. The bottom width of the stream varies from 30 ft to almost 50 ft in the area covered by the flood-control project.

5. Due to urbanization in the Fourmile Run drainage basin, the improved drainage and increased areas of imperviousness result in a high percent of runoff into the Fourmile Run channel. Flood conditions are produced by intense rainfall of short duration due to the characteristics of the watershed and the inability of the Fourmile Run channel to handle a high degree of runoff.

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* All elevations (el) cited herein are in feet referred to mean sea level.

Existing Problems

6. Flood flows are highly erosive, resulting in major damage to bridge abutments, retaining walls, and commercial and residential property adjacent to the channel. Constrictions in the channel, poor alignment, and deposition around bridges have greatly reduced the discharge capacity of Fourmile Run. Upstream from the confluence with the Potomac River, a constriction is present as a result of a series of highway and railroad culverts and bridges. Extensive backwater flooding in the city of Alexandria and Arlington County occurs because of this situation. A poor hydraulic condition also exists in the vicinity upstream of Mount Vernon Avenue with the intrusion of the left bank into the channel. This intrusion reduces the effective width of the Mount Vernon Avenue Bridge opening to almost 50 percent. Realignment of the left bank is restricted because of the urbanization of that area. Further constrictions are present in the vicinity of West Glebe Road Bridge where observations after flood conditions indicated several bays were probably rendered ineffective by sediment deposited prior to peak discharges. Scour patterns consistently develop along the left vertical wall upstream of the West Glebe Road Bridge, resulting in damage to the footing and even failure of the vertical wall.

7. The discharge capacity of Long Branch is limited by an inadequate-size opening under the South Glebe Road Bridge. Backwater flooding results from this constriction. Attention should be called to the fact that the water surface where Long Branch enters Fourmile Run is approximately 2 ft higher than the water surface in Fourmile Run. Under the flood-control project, a drop structure will be constructed to lower the invert of Long Branch so that the water surfaces at the confluence of Long Branch and Fourmile Run coincide.

8. The proposed flood-control project is designed to increase the discharge capacity of the Fourmile Run channel, which will aid in the containment of design floods of 100-yr frequency. Some of the proposed improvements involve elimination of the existing constriction at the bridges near the Potomac River and excavation of a 180- to 200-ft-wide

channel upstream to Mount Vernon Avenue Bridge. Other proposed improvements involve protecting the bank slopes against erosion, constructing vertical walls along particular reaches, and widening and improving the alignment of the channel.

Project Design Floods

9. The proposed flood-control project will be designed for peak discharges resulting from a 100-yr frequency flood. The protective works on Fourmile Run from I-95 to its confluence with Long Branch are designed for a peak discharge of 22,500 cfs. The protective works are designed for an increased peak discharge of 26,000 cfs from the Long Branch junction to a right bank side drain upstream from U. S. Rt. 1, with the additional flow entering from Long Branch, and for 27,000 cfs from the side drain to the Potomac River. The protective works on Long Branch are designed for a peak discharge of 4500 cfs. The project hydrographs for both channels are shown in Plate 1.

Purpose of Model Investigation

10. Design criteria for hydraulic losses at junctions, expansions, contractions, and bridges were not fully defined and could not be reliably computed. In order to ensure an optimum and economical design, a hydraulic model study was recommended. Specifically, the model study was to determine:

- a. Flow conditions resulting from junctions, expansions, contractions, and bridges.
- b. The optimum bank slope protection.
- c. Areas of critical velocity and flow distribution.
- d. Water-surface elevations at various areas throughout the project.

PART II: THE MODEL

Description

11. The model, constructed to a scale of 1:30, reproduced approximately 5700 ft of the Fourmile Run channel, beginning at the I-95 Bridge and terminating approximately 600 ft downstream of Mount Vernon Avenue Bridge, and 400 ft of the lower reach of Long Branch (Figure 2). All proposed hydraulic structures, riprapped slopes, and improved alignment of the channel were reproduced in the model. The proposed stilling basins and vertical walls were reproduced in plastic-coated plywood; simulated concreted sections in the channel were constructed of concrete with a smooth finish. All bridges, piers, and channel dividers were constructed of transparent plastic. The channel was molded in sand with either scaled riprap or scaled gabions placed on filter cloth on the side slopes. The gabions consisted of wired baskets filled with rock.

Model Appurtenances

12. Water used in the operation of the model was supplied by a circulating system. Discharges were measured by means of Venturi meters installed in the flow lines and were baffled when entering the model. Steel rails graded to specific elevations were placed along both sides of the model to serve as supports for measuring devices and to provide a convenient means of establishing stations and elevations in the model. Velocities were measured with Pitot tubes which were mounted to permit measurement of flow from any direction and at any depth. Water-surface elevations were measured with point gages. Different designs before and after tests along with several different flow conditions were recorded photographically.

Scale Relations

13. The accepted equations of hydraulic similitude, based on

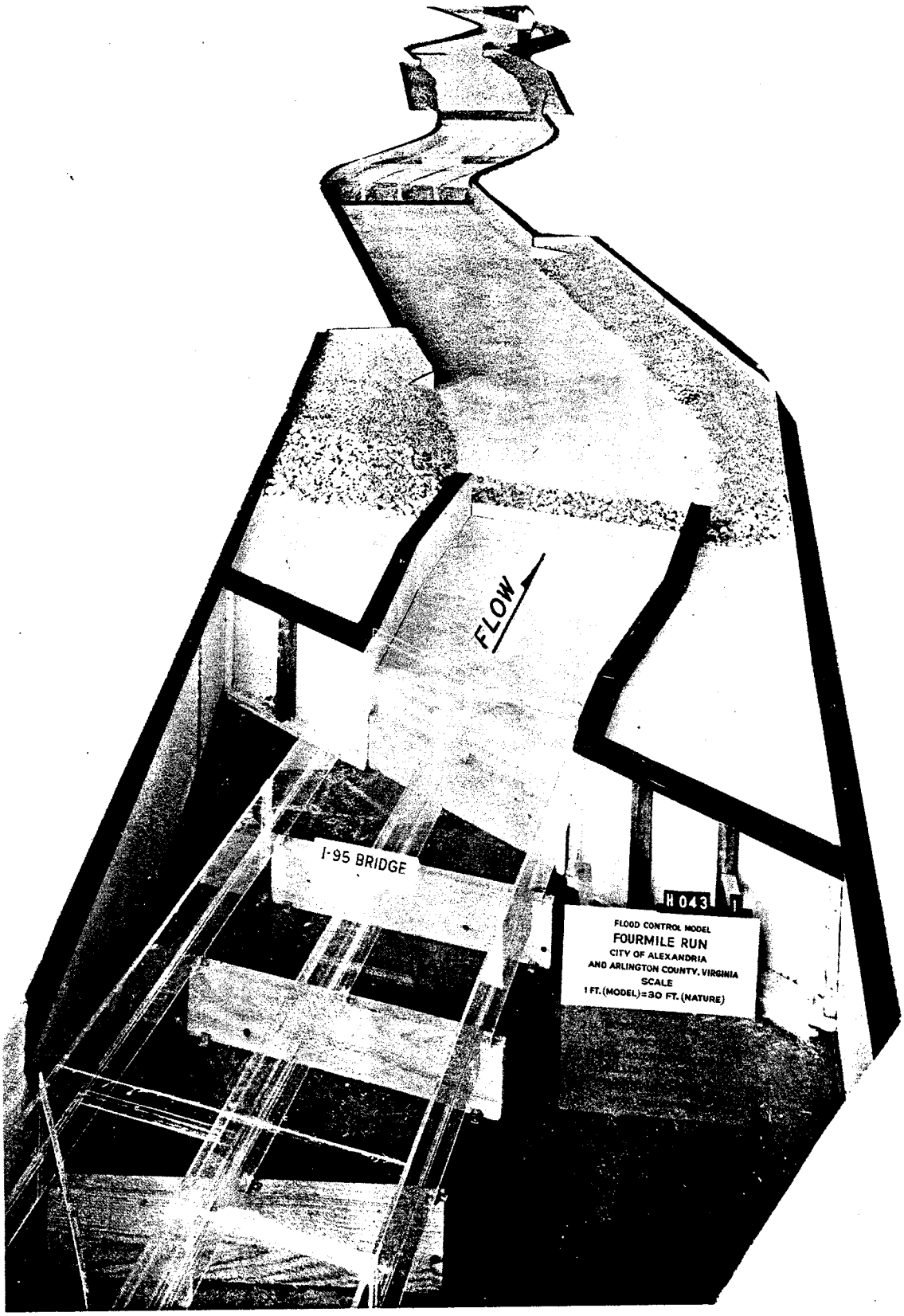

Figure 2. General view of model

Froudian relations, were used to express mathematical relations between the dimensions and hydraulic quantities of the model and prototype. General relations for transference of model data to prototype equivalents are as follows:

Characteristic	Dimension*	Model:Prototype
Length	L	$L_R = 1:30$
Area	L_R^2	$A_R = 1:900$
Velocity	$L_R^{1/2}$	$V_R = 1:5.477$
Time	$L_R^{1/2}$	$T_R = 1:5.477$
Discharge	$L_R^{5/2}$	$Q_R = 1:4,929.50$
Weight	L_R^3	$W_R = 1:27,000$

* Dimensions are in terms of length.

14. Model measurements of discharge, water-surface elevations, and velocities can be transferred quantitatively to prototype equivalents by means of the preceding scale relations. Also, the limited experimental data available indicate that the prototype-to-model scale ratio is valid for scaling riprap in the sizes used in this investigation. Evidences of sand scour, however, are considered only qualitatively reliable, since it is not yet possible to reproduce quantitatively in a model the same ratio of flow depth to the diameter of bed material representative of the prototype.

PART III: TESTS AND RESULTS

Original Design

Initial tests

15. The original plan of improvement for the Fourmile Run flood-protection project consisted of realigning and widening the channels to confine peak discharges, protecting the side slopes with riprap, and constructing hydraulic structures at various locations throughout the project. These improvements and the model limits are shown in Plate 2. Initial tests were conducted to observe general flow conditions and determine the problem areas throughout the reach reproduced in the model during a simulated project hydrograph. Results of the tests pertinent to each component of the project will be discussed in order of its position, beginning with the stilling basin downstream from the I-95 Bridge and proceeding in a downstream direction.

Stilling basin downstream from I-95 Bridge

16. The original design improvement plan included a stilling basin immediately downstream from the I-95 Bridge to dissipate excess energy from the high-velocity flow underneath the bridge. This basin is shown in Photo 1a; details are shown in Plate 3. With the design flow of 22,500 cfs, a hydraulic jump was not maintained within the basin (Photo 1b); this resulted in high velocities and excessive wave action in the channel downstream. The end sill of the basin was lowered 5.37 ft to el 19.0 and a 1V-on-20H slope with riprap protection was extended 95 ft downstream in an effort to improve flow conditions; however, little improvement was realized from this modification. Testing of the stilling basin was discontinued at this preliminary stage because a more feasible plan of protection was developed which consisted of improving the soft-bottom channel without the hydraulic structures.

Debris weir, flow control vanes, and energy dissipator

17. A debris weir (Photo 2 and Plate 4), located at sta 408+50,

was designed to create an area upstream for sediment deposition and develop supercritical flow to lower the water-surface elevation under the West Glebe Road Bridge. The weir and channel invert downstream to sta 410+00 were paved. Three flow control vanes (Plate 5) were designed to equally distribute the flow through the curved reaches of channel between sta 408+56 and 414+50. An energy dissipator (Plate 6) was located at the downstream end of the flow control vanes (Photo 3) to change flow from a supercritical to subcritical state and reduce velocities to an acceptable level in the soft-bottom channel.

18. Flow conditions over the weir and through the flow control vanes were satisfactory with the design discharge (see Photo 2b). However, considerable scour developed just downstream from sta 410+00 (Photo 2c). This sediment was trapped in the stilling basin downstream (Photo 3c) causing it to be ineffective as an energy dissipator. The basin with a flow of 22,500 cfs is shown in Photo 3b. Completely paving the channel invert from the debris weir to the energy dissipator (Photo 4) greatly reduced the sediment deposition in the energy dissipator and improved flow conditions. However, no additional tests were conducted with these hydraulic structures in place due to the reasons discussed in paragraph 16.

19. Further testing to determine the minimum size of riprap for bank-slope protection and observation of the performance of the drop structure in Long Branch (Plate 7) with the proposed hydraulic structures incorporated in the Fourmile Run channel were terminated. All efforts were redirected to modifying the model in order to study the improved natural soft-bottom channel with riprap- or gabion- (wire basket filled with riprap) protected side slopes.

Improved Natural Channel Design

20. After the original design improvement plan had been constructed in the model and initial tests conducted, a plan of improvement which eliminated the hydraulic structures in the Fourmile Run channel was found by the sponsoring agency to be more feasible. As shown in

Plate 8, this plan consisted of widening the existing channel and protecting the bank slopes against erosion. The drop structure in Long Branch remained basically the same as originally proposed. Model tests were concerned with developing the optimum bank-slope protection and determining water-surface elevations in critical areas. The results of tests will be discussed as problem areas were encountered and solved in a downstream direction.

I-95 Bridge to
West Glebe Road Bridge

21. The proposed improvements in this reach of channel consisted of widening the channel invert from 90 ft at the I-95 Bridge to 120 ft at West Glebe Road Bridge and protecting the bank slopes (1V on 2H) with 36-in. riprap (Plate 8 and Photo 5a). An existing vertical retaining wall on the left side of the channel from sta 409+00 to West Glebe Road Bridge and the existing pier alignment of the West Glebe Road Bridge were reproduced in the model (Photo 6a) and were not altered in the proposed improvements.

22. During a simulated project design hydrograph, failure of the 36-in. riprap occurred along both bank slopes just downstream from the I-95 Bridge (Photo 5b). The failure was primarily due to surface wave action and not the velocity of the flow or the flow condition created by the constriction under the bridge. Since riprap larger than 36 in. was not considered feasible by the sponsor, the riprap was replaced with gabion protection in this area.

23. Various types and sizes of gabions were tested along with several types of gabion toe protection (Plate 9) required to prevent sloughing of the gabions due to scour at the toe. Failure in the gabion test sections was consistent when no toe protection was installed. Figure 3 shows the type of failure that resulted with inadequate toe protection (type II, Plate 9) and the stability of the gabions when adequate protection (types I and III, Plate 9) was provided. Gabions in the prototype would not fail as shown in Figure 3 because they would be laced together to form a monolithic structure. Therefore, results of tests in the model without the lacing should be conservative. However, the

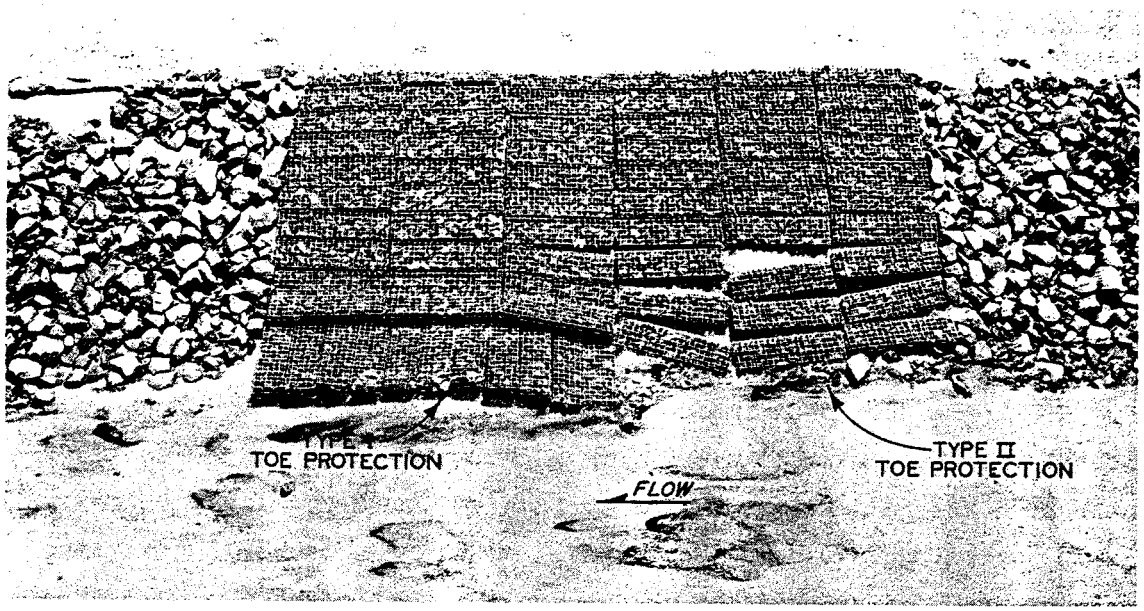

Figure 3. Gabion test section showing stability of type I toe protection and failure with type II toe protection

types I and III toe protection plans were superior to type II regardless of whether the baskets were laced together. Gabions, 12 by 3 by 1.5 ft, with either the type I or III toe protection provided the optimum protection for this reach of channel. The model with the gabions in place is shown in Figure 4. The gabions in this area were extended down 5 ft below the channel invert and backfilled to provide additional protection from the high-velocity flow under I-95 Bridge.

24. The curved alignment of the channel upstream from the West Glebe Road Bridge caused flow to be concentrated along the left bank (Photo 6). This condition resulted in considerable scour along the vertical wall on the left bank and deposition of sediment along the right bank causing the right bay of West Glebe Road Bridge to be ineffective in passing flow during the project hydrograph. Efforts were made to reduce the deposition of sediment by directing more flow through the right bay with various deflector vanes upstream from the bridge. However, no feasible solution was found to keep the total cross section of the bridge open during flows greater than 10,000 cfs. A plan for protecting the footing of the wall along the left bank against scour

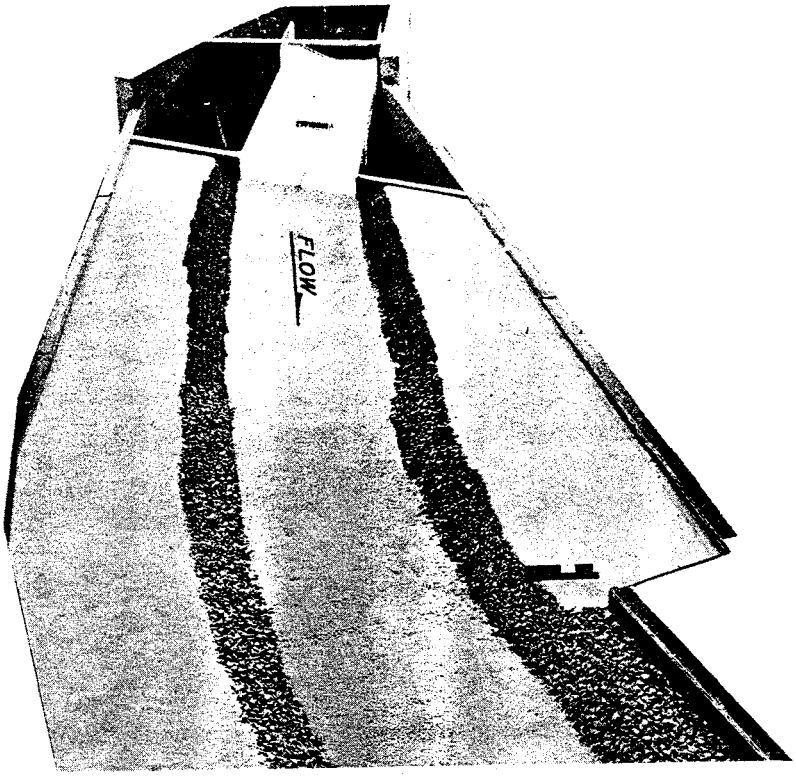

Figure 4. A 12- by 3- by 1-ft gabion pattern downstream of I-95 Bridge

was developed using a pattern of gabions (Plate 10).

West Glebe Road
Bridge to sta 429+00

25. The proposed improvements in this reach consisted of widening the channel to 120 ft from West Glebe Road Bridge to sta 425+21, transitioning to 150 ft at sta 427+15 with this same width continued downstream to sta 429+00. The bank slopes (1V on 2H) were protected with 36-in. riprap (Plate 8 and Figure 5).

26. Failure of the riprap occurred on the left bank just downstream from West Glebe Road Bridge (Figure 6) during a discharge of 10,000 cfs, and along the right side from sta 413+00 to sta 417+00 during peak discharges of the project hydrograph. High velocities created considerable scour along the toe of the slope in these areas, causing failure of the riprap due to sloughing. The areas in which

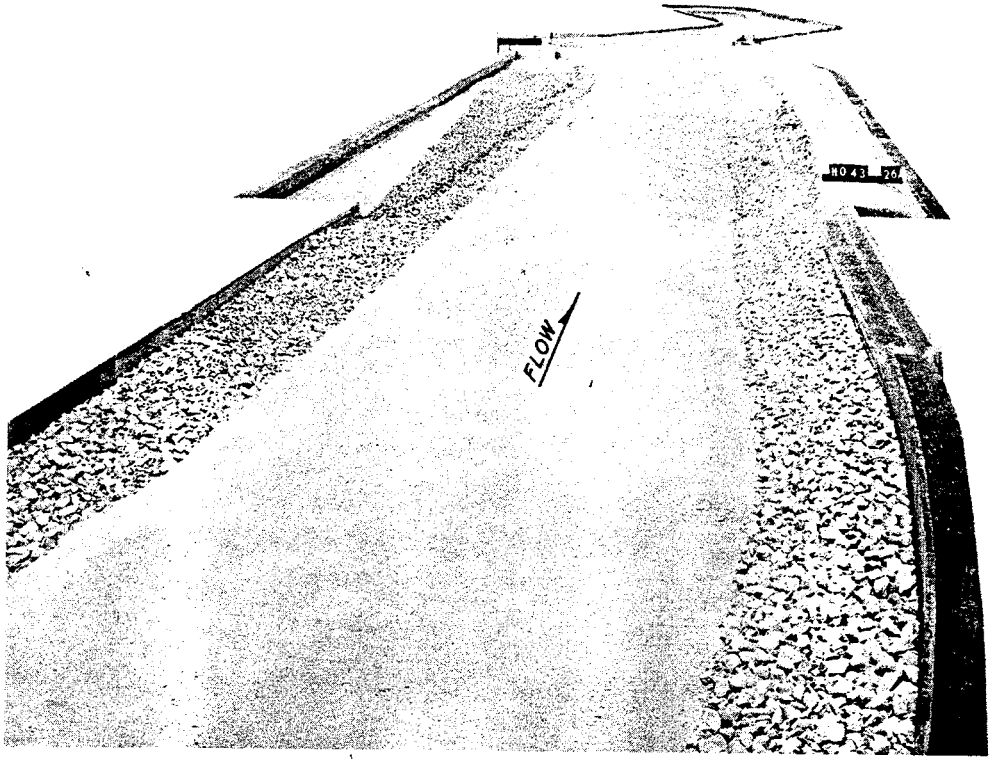

Figure 5. Improved natural channel downstream from West Glebe Road Bridge

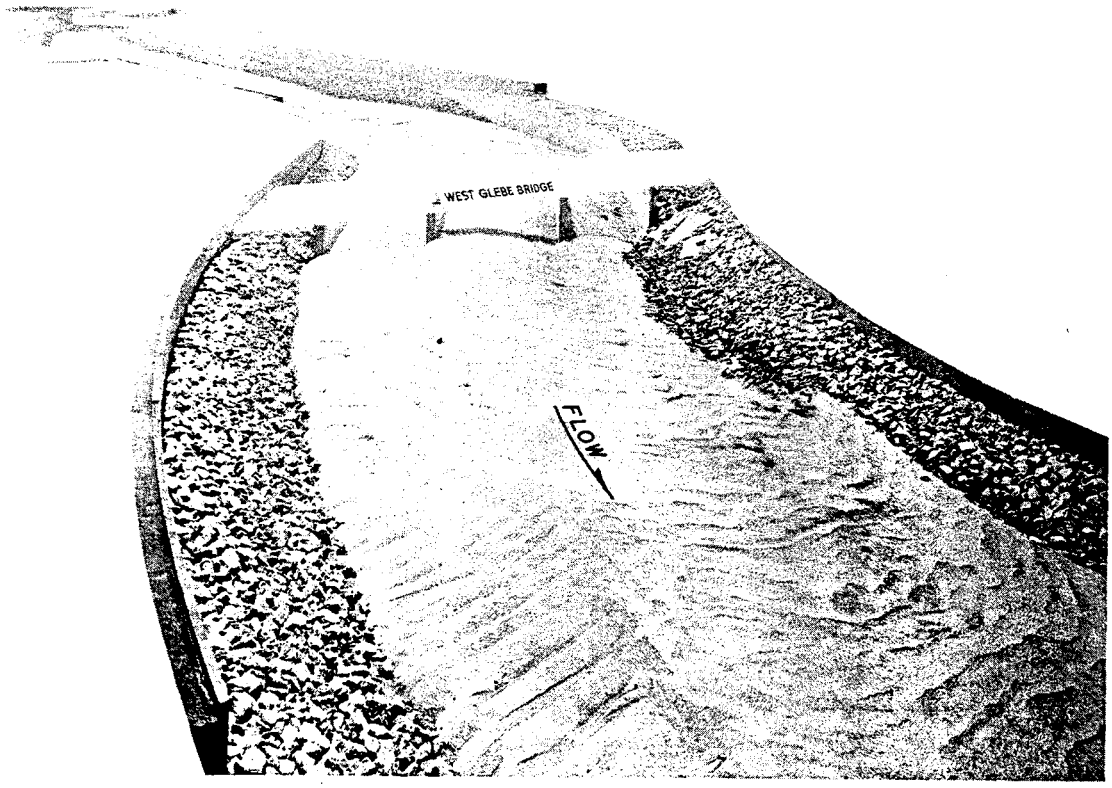

Figure 6. Upstream view of failure in the 36-in. riprap below West Glebe Road Bridge after a discharge of 10,000 cfs

failure occurred were replaced with gabions and tests were conducted to determine their minimum thickness for stability. The area on the left bank immediately downstream from the West Glebe Road Bridge required gabions 3 ft thick for a distance of 50 ft and an additional 200 ft of 1.5-ft-thick gabions. The recommended protection is shown in Figure 7

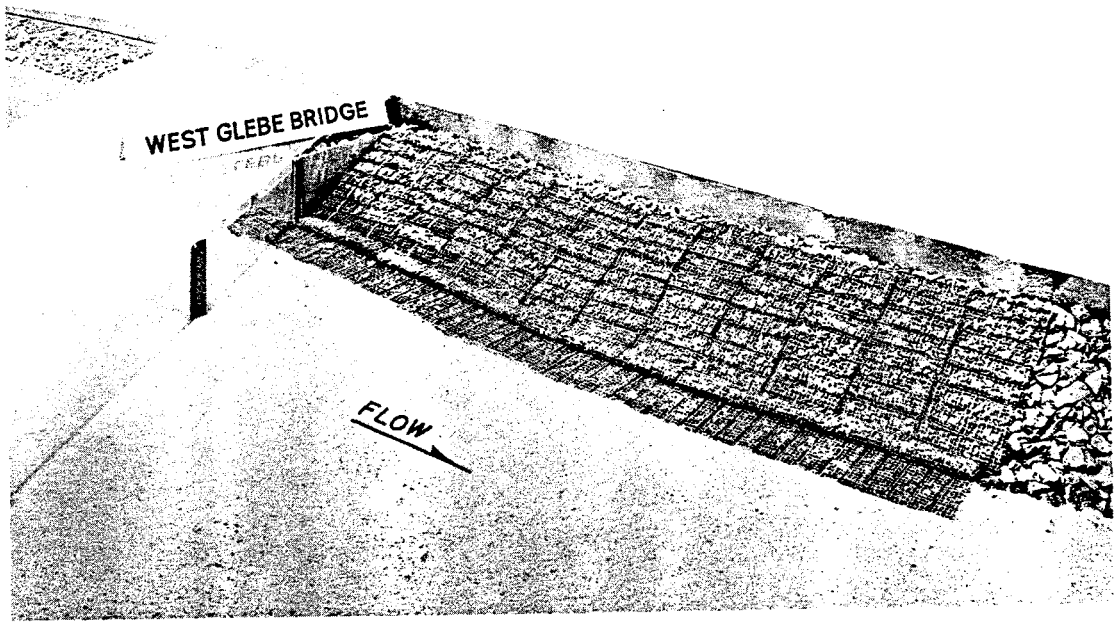

Figure 7. Recommended gabion protection on left bank downstream from West Glebe Road Bridge

and Plate 10. Gabions 1 ft thick were sufficient protection for the right bank between sta 412+00 and 429+00.

27. A discharge of 8000 cfs or greater overtopped the left slope between sta 420+00 and 421+00. To prevent flooding the area adjacent to the left slope, a vertical wall was placed 2 to 5 ft from the top of the slope, extending from 500 ft downstream of West Glebe Road Bridge to 250 ft upstream of Long Branch.

Sta 429+00 to
Mount Vernon Avenue Bridge

28. The channel in this reach will be 150 ft wide from sta 429+00 to 444+35 and transition to 200 ft wide at sta 447+45. The bank slopes (1V on 2H) were protected with 36-in. riprap from sta 429+00 to 439+62 and with 15-in. riprap from sta 439+62 to 444+50. Vertical walls are

located on both sides of the channel at sta 444+50 and are extended downstream to the Mount Vernon Avenue Bridge. Details of this reach are shown in Plate 8; the model channel is shown in Figure 8.

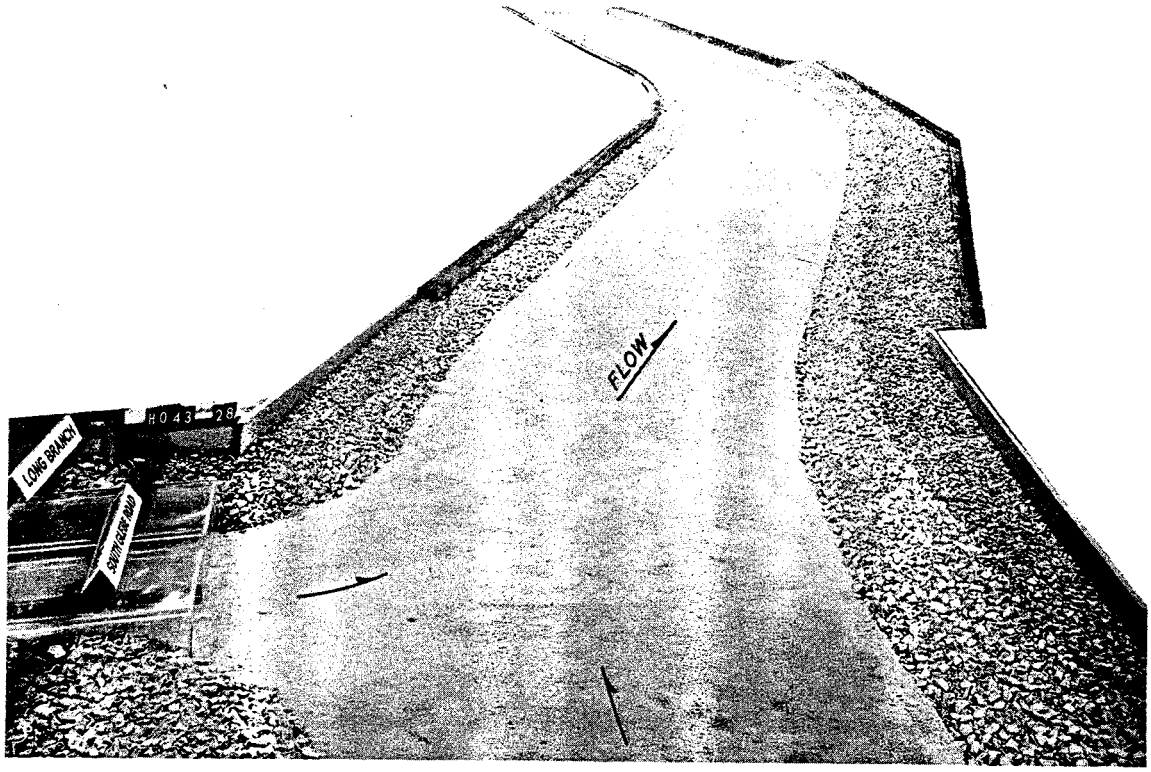

Figure 8. Improved natural channel downstream from Long Branch

29. Failure occurred in the 36-in. riprap along the left bank from the confluence of Long Branch to sta 431+00 (Figure 9). Flow from Long Branch was forced against the left slope by the higher contemporaneous discharges in Fourmile Run, causing scour at the toe of the slope, thereby undermining the riprap. A test section of 1-ft-thick gabions was placed in this area (Figure 10). These gabions were stable for all discharges tested, including the design flow.

30. The 15-in. riprap on the right bank just upstream from Mount Vernon Avenue Bridge failed during peak discharges of the project hydrograph. This failure is shown in Figure 11. Tests with various sizes of riprap indicated that 21-in. riprap would be required for slope protection on the right bank between sta 429+00 and 444+50 and on the left bank between sta 432+00 and 439+62. The 15-in. riprap on the left bank

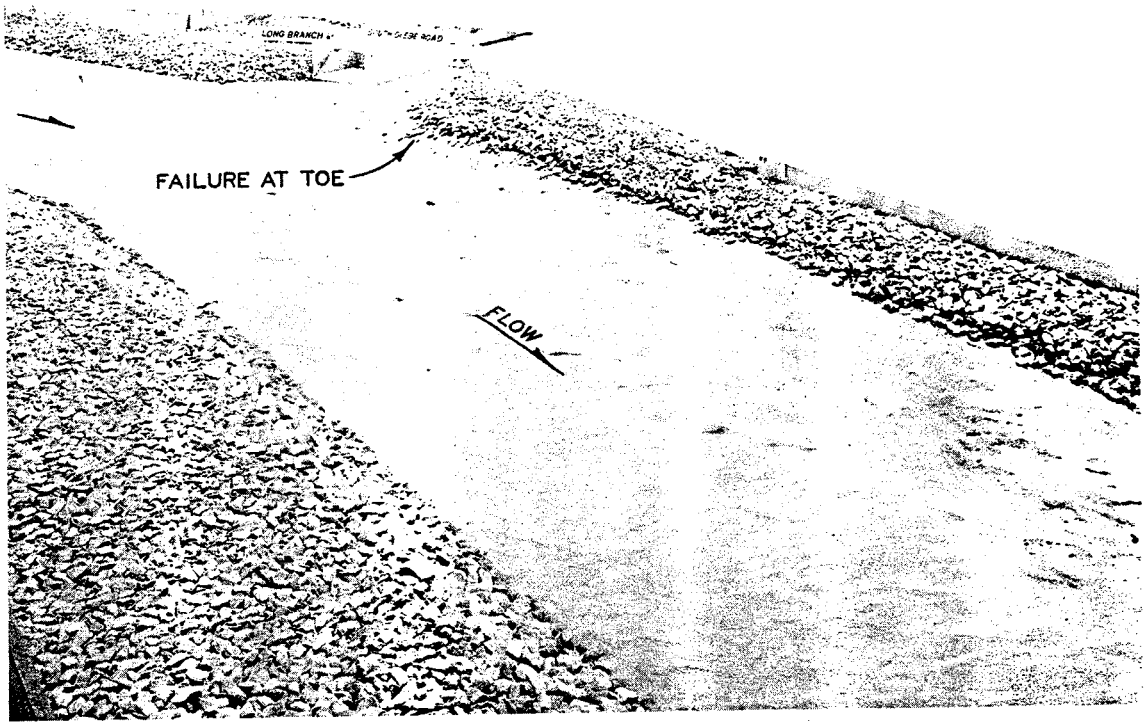

Figure 9. Failure of 36-in. riprap downstream from Long Branch confluence

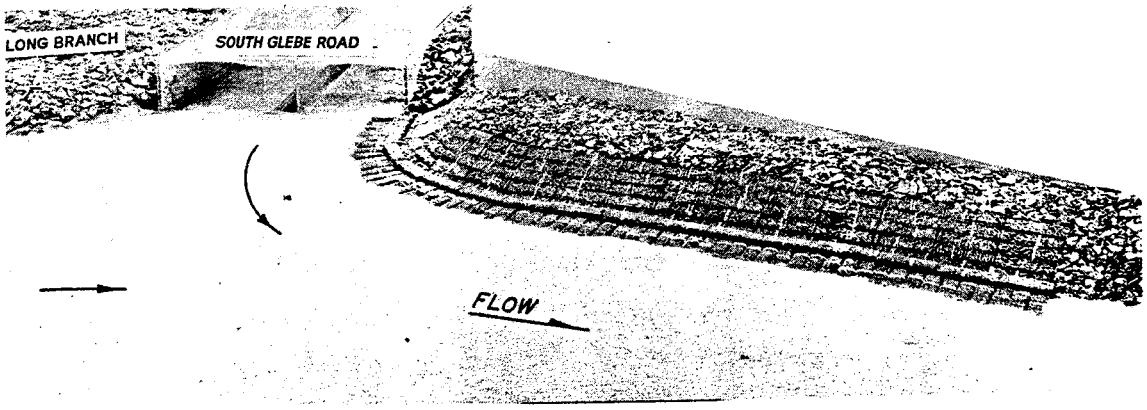

Figure 10. Recommended gabion protection downstream from Long Branch confluence

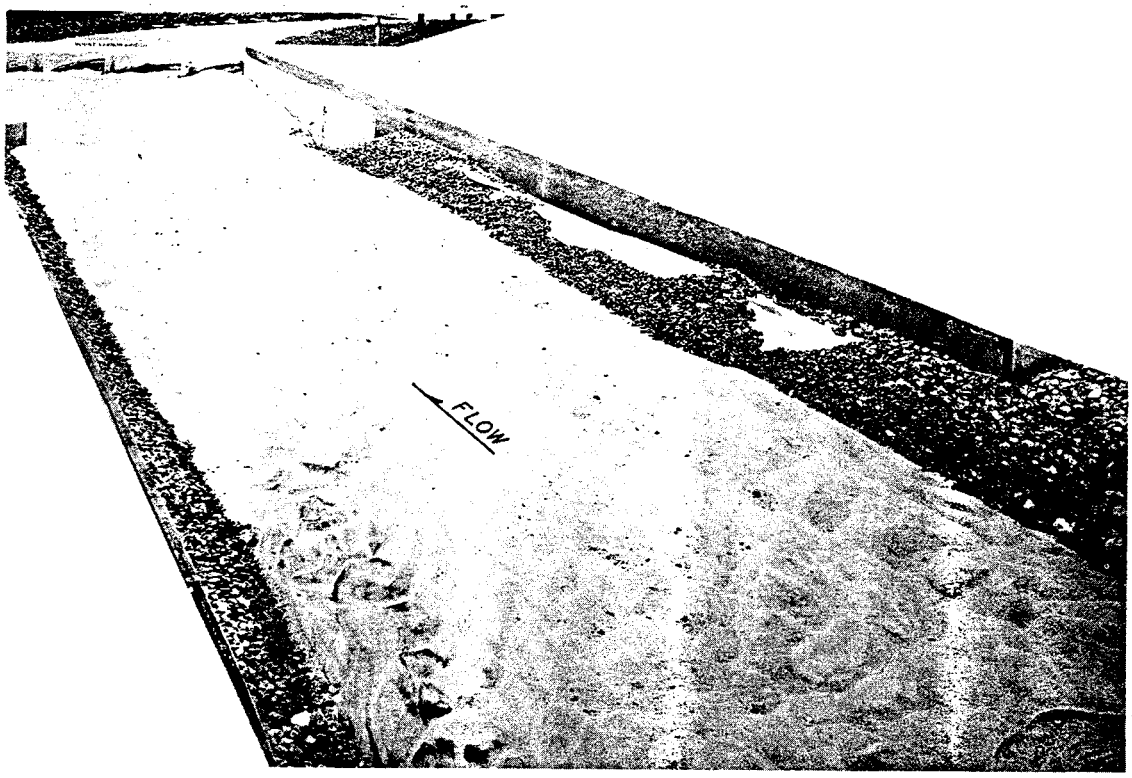

Figure 11. Failure of 15-in. riprap on right bank between Long Branch and Mount Vernon Avenue Bridge.

between sta 439+62 and 444+50 was stable during the entire simulated project hydrograph. The model also indicated that 12- by 3- by 1-ft gabions would be stable over this entire reach.

Mount Vernon Avenue
Bridge to sta 454+00

31. The proposed improvements in this reach consisted of improving the channel alignment at Mount Vernon Avenue Bridge by abandoning the northernmost span and widening the channel to 200 ft. The bank slopes (1V on 4H) were protected with 21-in. riprap on both sides. No failure occurred in the riprap during the project hydrograph, but flow did overtop the bank slopes at el 8.0 during peak discharges. Protective measures to prevent flooding this area were not feasible according to the sponsor, so no flood walls will be provided in this area. Further testing resulted in the reduction in the minimum required riprap size from 21 to 18 in. Tests conducted with gabions placed in the

control section of this reach indicated that 12- by 3- by 1-ft gabions would be stable in this entire reach.

Long Branch

32. The proposed improvements on Long Branch start at the junction with Fourmile Run and consist of adding an additional 30-ft span under South Glebe Road Bridge and widening the channel upstream from the bridge to provide the proper approach flow conditions under the bridge. The channel improvements consist of a trapezoidal channel with a 60-ft bottom width and 1V-on-2H bank slopes protected with 36-in. riprap. The channel will narrow to meet the existing channel approximately 180 ft upstream from South Glebe Road Bridge. The improvements also include a drop structure (Photo 7a, Plate 11) located at sta 13+25. This structure is designed to reduce the erosive velocities in the channel and to lower the water surface to coincide with that of Fourmile Run.

33. Discharges up to 4,500 cfs were observed in Long Branch, with various flows ranging from 1,000 cfs to 22,500 cfs in Fourmile Run. The drop structure in Long Branch was submerged with flows greater than 10,000 cfs in Fourmile Run because of the tailwater effects. The 36-in. riprap was stable during the complete series of tests on Long Branch.

34. Additional tests were conducted on Long Branch to determine the optimum gabion protection plan. These tests were conducted with the project hydrograph and a low tailwater in the Fourmile Run channel, since this condition would produce the most critical design condition for the gabions. The peak discharge of 4500 cfs is shown in Photo 7b. These tests indicated that 12- by 3- by 1-ft gabions would be the optimum size for the entire reach of Long Branch. The proposed gabion protection plan is shown in Plate 12.

Recommended Channel Design

Bank-slope protection

35. Two separate plans of bank-slope protection were developed from the model tests. One plan used riprap where 36-in. stone or less was stable, and gabions in the areas where the 36-in. riprap was

unstable; this plan is shown in Plate 13. The other plan used gabion protection for the entire reach (Plate 10). This plan was developed using typical gabion test sections in the model since it was not practical to cover the entire model reach with gabions because of the expense and time required for their construction.

Water-surface profiles

36. Water-surface profiles were obtained over the entire model reach during steady flow conditions with discharges of 10,000, 15,000, and 22,500 cfs in Fourmile Run with concurrent discharges of 3,000, 4,000, and 3,500 cfs, respectively, in Long Branch. These profiles are shown in Plates 14-16; water-surface elevations are tabulated in Table 1.

Channel bed configuration

37. A final test was conducted to determine the approximate configuration of the channel bed after a project design hydrograph was reproduced. Measurements were made to determine the areas of scour and deposition. These data are shown in Plates 17-20 for various stations and should be considered as only qualitatively reliable. These data cannot be used to predict prototype depths of scour and deposition but do indicate areas of scour and deposition. In areas where scour appears to undermine the side slopes on the cross sections, the gabion toe and bank protection conformed and protected the altered slope.

PART IV: DISCUSSION OF RESULTS

38. The initial plan of improvements for the flood-control project incorporated hydraulic structures from the I-95 Bridge to sta 416+50, and a drop structure in Long Branch. Testing of this proposed plan was halted in the preliminary stage of the testing program because the development and protection of the natural, soft-bottom channel was considered more feasible.

39. With the design discharge in the improved natural soft-bottom channel, surface wave action between the I-95 Bridge and West Glebe Road Bridge caused failure in the 36-in. riprap along both bank slopes. Riprap larger than 36 in. was not considered feasible so gabions were tested and found to provide better protection.

40. The curved alignment of the channel upstream from West Glebe Road Bridge caused undesirable flow conditions along the left bank and under the bridge as anticipated. During high flows, considerable scour occurred along the left retaining wall upstream from West Glebe Road Bridge and a pattern of gabions was placed along the wall. The gabions reduced the scour considerably and provided the necessary protection.

41. Deposition of sediment in the right bay of West Glebe Road Bridge occurred during the project hydrograph, rendering it ineffective (see Photo 8b). Efforts were made to reduce the deposition of sediment by directing more flow through the right bay, but no feasible solution was found.

42. Failure occurred in the 36-in. riprap on the left bank just downstream of West Glebe Road Bridge with a discharge of only 10,000 cfs. Various size gabions and patterns were tested for higher discharges and an adequate slope protection plan for the 22,500-cfs, 100-yr flood was developed.

43. Flow conditions from West Glebe Road Bridge to sta 429+00 were satisfactory. However, with discharges of 8000 cfs or greater, flow overtopped portions of the left slope. A 5-ft-high vertical wall was placed 2 to 5 ft back from the top of the slope to provide the

necessary protection during the peak flows. Riprap and gabion protection plans for the bank slopes were also developed for this reach of channel.

44. Flow from Long Branch at its confluence with Fourmile Run was consistently kept against the left bank by the higher contemporaneous discharges in Fourmile Run. This flow condition resulted in failure of the 36-in. riprap originally proposed for this area, and the necessary protection along this slope was found to be 12- by 3- by 1-ft gabions.

45. Flow conditions in Fourmile Run between Long Branch and Mount Vernon Avenue Bridge were satisfactory. During the peak discharges, flow overtopped both bank slopes downstream of Mount Vernon Avenue Bridge; however, additional protection to this area was not considered feasible. Riprap and gabion protection plans for the bank slopes were developed for this reach of channel.

46. The drop structure in Long Branch performed satisfactorily for low discharges. During peak discharges the drop structure was ineffective due to the high tailwater resulting from the higher discharges in Fourmile Run. No other problems were encountered in Long Branch. An optimum gabion bank-slope protection plan was developed for the 400-ft reach of Long Branch reproduced in the model.

47. After the optimum bank-slope protection had been determined for the entire reach reproduced in the model, a final test was conducted to determine the configuration of the movable channel bed after a project design hydrograph was reproduced. This information is only qualitatively reliable, but does indicate areas in which scour and deposition can be expected in the prototype.

48. Although no precise design rules were established for determining the minimum thickness of gabions required for stability against flow (it was impractical to construct gabions of enough different thicknesses to determine the exact minimum thickness that would be stable for all of the areas), there were several areas in the model in which 36-in.-thick riprap and 12-in.-thick gabions were stable. Therefore, results of this study indicate that the required gabion thickness is no more than one-third the required riprap thickness.

necessary protection during the peak flows. Riprap and gabion protection plans for the bank slopes were also developed for this reach of channel.

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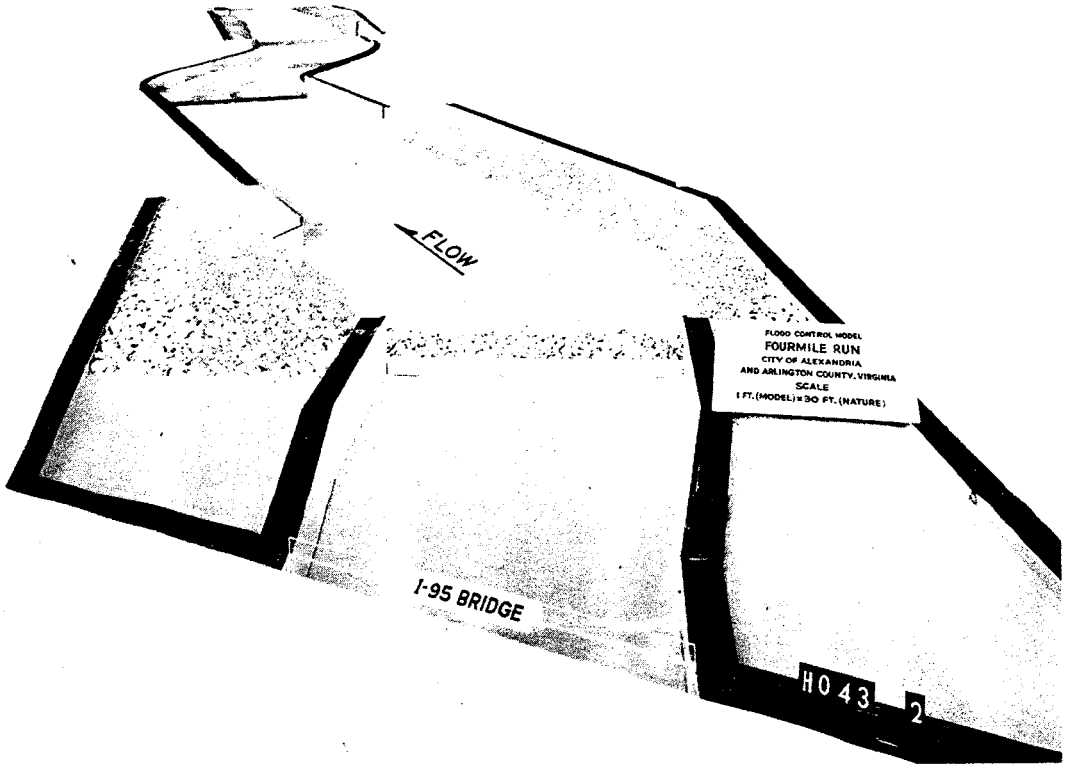
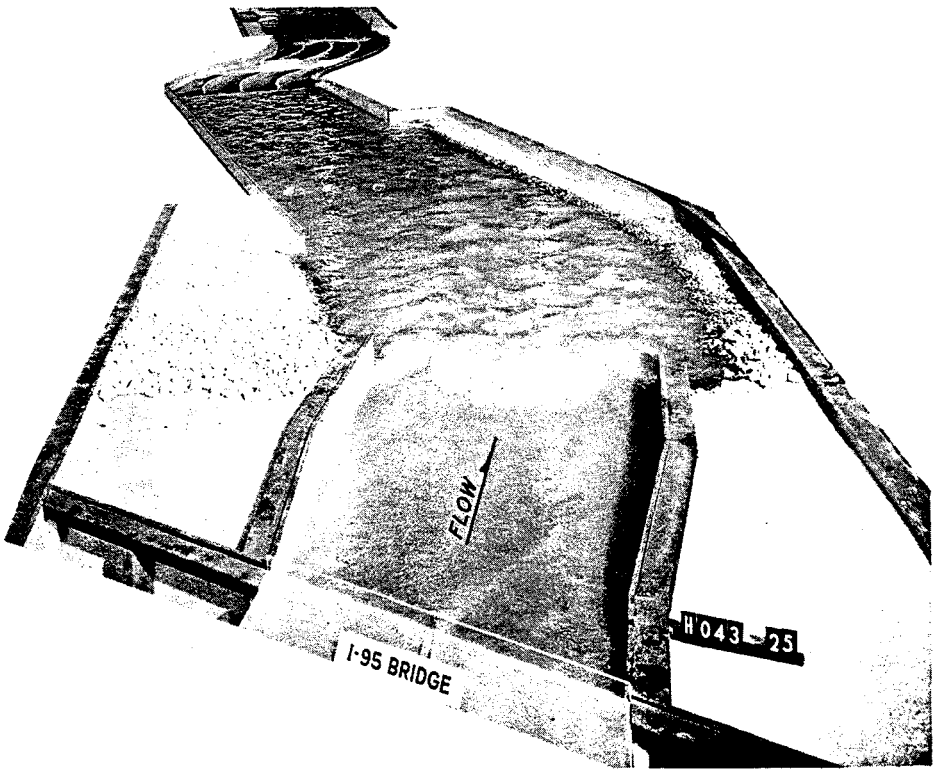
Table 1

Water-Surface Elevations

Stations	Q = 10,000 cfs (Fourmile Run) Q = 3,000 cfs (Long Branch) (Tailwater el, 8.0)		Q = 15,000 cfs (Fourmile Run) Q = 4,000 cfs (Long Branch) (Tailwater el, 10.0)		Q = 22,500 cfs (Fourmile Run) Q = 3,500 cfs (Long Branch) (Tailwater el, 12.0)	
	5 ft from Left Wall or Riprap	5 ft from Right Wall or Riprap	5 ft from Left Wall or Riprap	5 ft from Right Wall or Riprap	5 ft from Left Wall or Riprap	5 ft from Right Wall or Riprap
398	29.0	29.2	30.7	31.4		
399	28.2	28.3	29.0	29.1		
400	27.8	28.0	28.5	28.5	31.7	32.5
401	26.4	26.6	27.0	27.2	30.4	30.8
402	26.2	26.6	26.0	26.9	28.6	30.0
403	25.6	26.3	26.1	26.7	28.5	29.5
404	25.6	25.4	26.0	26.0	27.5	27.4
405	25.4	25.2	25.5	25.8	27.0	27.0
406	24.6	24.6	25.5	25.4	26.5	26.5
407	24.2	24.1	25.0	25.1	26.5	26.4
408	24.2	24.2	25.3	24.9	26.7	26.5
409	24.3	23.5	25.6	24.7	27.0	25.9
410	24.3	23.0	25.9	24.4	27.5	25.4
Upstream of West Glebe Road Bridge	23.7	22.5	25.7	24.4	27.0	25.5
Downstream of West Glebe Road Bridge	22.6	22.1	23.7	23.4	26.6	25.6
412	22.0	22.3	22.7	23.4	25.9	25.5
413	21.6	22.3	22.5	23.1	24.5	25.7
414	21.3	21.8	22.6	23.2	24.4	25.4
415	20.7	21.5	21.9	23.1	24.0	25.0
416	20.4	20.8	21.7	22.4	23.2	23.2
417	20.1	20.1	21.9	21.7	23.0	23.2
418	19.4	20.1	21.4	21.0	22.8	23.0
419	19.1	19.5	20.6	20.6	22.0	22.4
420	18.7	18.3	20.3	20.4	21.7	21.9
421	18.0	18.2	19.8	19.6	21.5	21.5
422	18.4	18.2	19.8	20.2	22.0	21.3
423	18.1	18.0	20.4	20.0	21.5	21.5
424	17.1	17.4	19.6	19.5	21.0	21.2
425	16.5	16.1	18.5	19.3	20.6	20.9
426	16.2	16.1	18.8	18.9	20.3	20.5
427	16.2	16.4	18.7	19.1	20.7	20.8
428	15.6	15.6	18.6	18.4	20.7	20.0
429	15.7	15.4	19.0	18.5	20.8	20.2
430	14.9	14.5	17.4	17.3	19.8	19.1
431	14.7	14.2	17.5	17.0	18.2	17.8
432	14.1	13.7	17.1	16.5	18.0	17.8
433	13.2	13.1	16.2	16.3	17.5	17.7
434	13.2	12.7	15.7	15.8	17.7	17.5
435	12.3	12.5	15.2	15.1	17.2	17.3
436	12.3	12.5	14.4	15.0	16.4	17.0
437	12.0	12.1	14.0	14.6	16.5	17.0
438	11.7	12.2	13.6	14.3	16.0	16.4
439	11.4	12.2	13.0	14.4	15.5	16.5
440	11.0	11.5	12.5	13.4	14.8	16.0
441	10.6	10.8	12.3	12.8	14.0	15.1
442	10.1	10.5	11.4	11.5	12.8	13.9
443	9.5	9.8	10.5	11.1	12.5	12.5
444	9.4	9.7	11.0	11.0	12.6	12.8
445	9.2	9.2	10.4	10.5	12.4	12.0
446	8.7	8.2	9.5	9.4	11.2	11.3
447	8.7	8.6	9.8	9.9	11.0	10.9
Upstream of Mt. Vernon Ave Bridge	8.8	8.5	10.0	9.4	11.9	10.5
Downstream of Mt. Vernon Ave Bridge	8.7	8.3	9.6	9.4	11.7	11.0
449	8.0	8.5	10.0	9.3	12.2	10.0
450	8.5	8.7	9.9	9.1	11.3	10.0
451	8.4	7.1	10.0	8.2	11.0	8.2
452	8.0	6.7	9.5	7.7	10.6	8.4
453	6.5	7.2	8.5	7.4	8.8	7.0

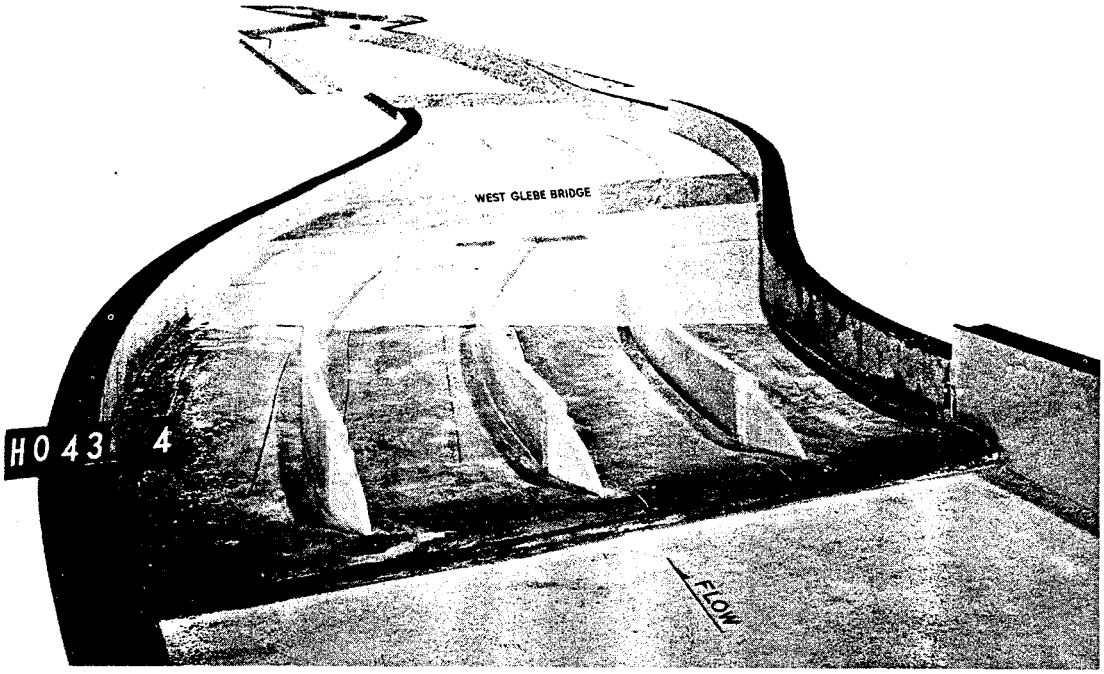


a. Dry bed downstream

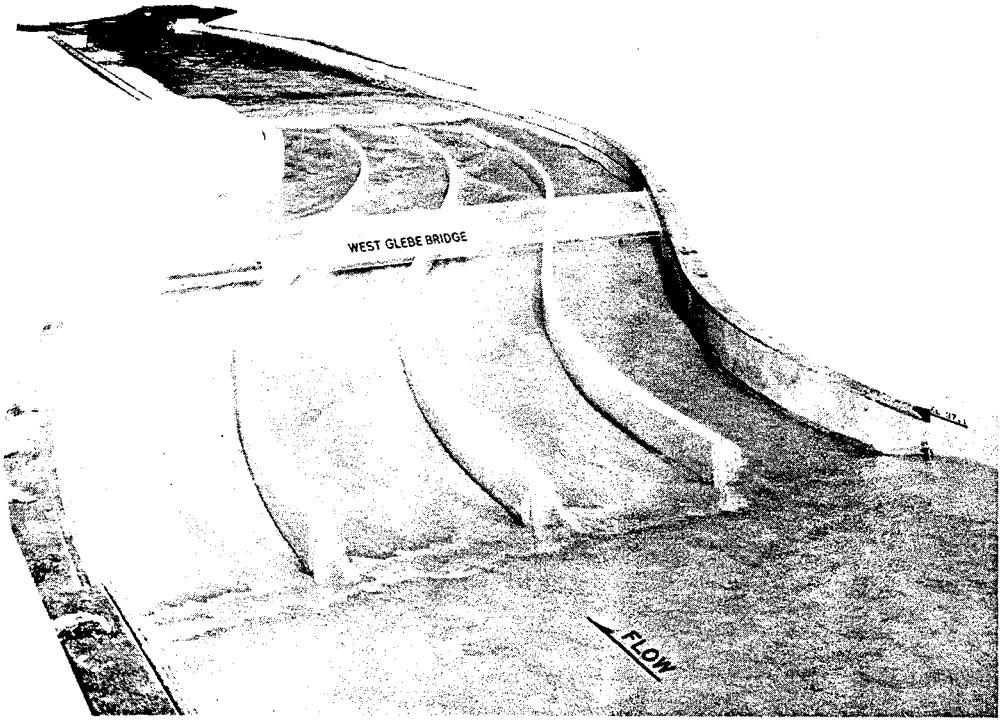


b. Discharge of 22,500 cfs; weak hydraulic jump

Photo 1. Stilling basin downstream from I-95 Bridge

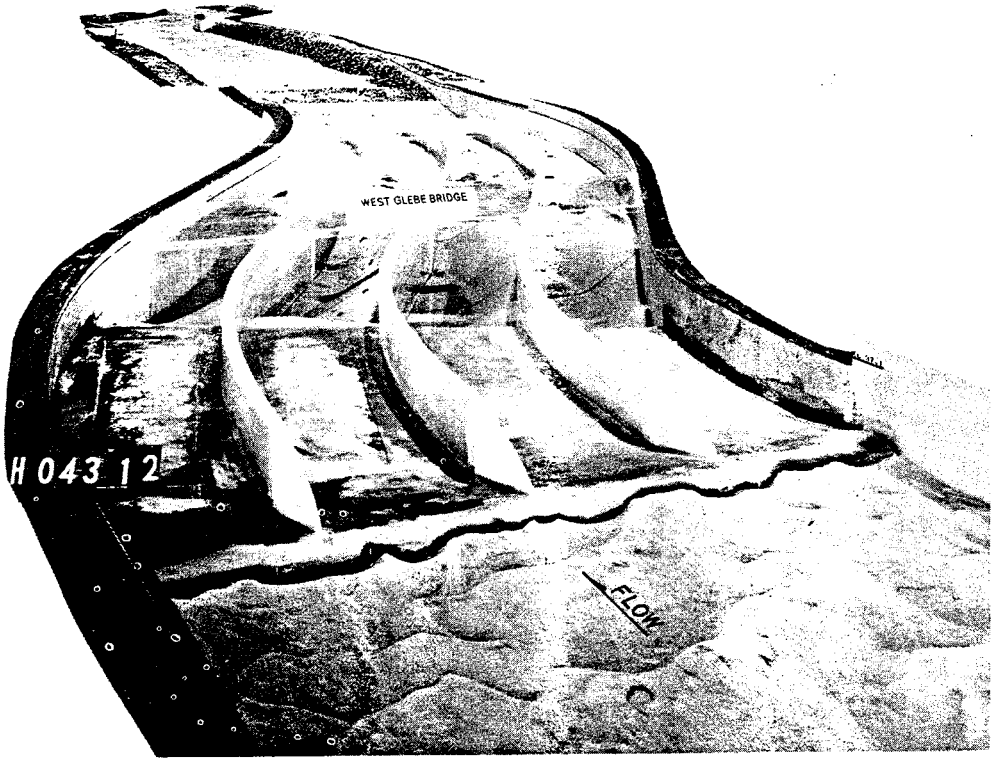


a. Dry bed downstream



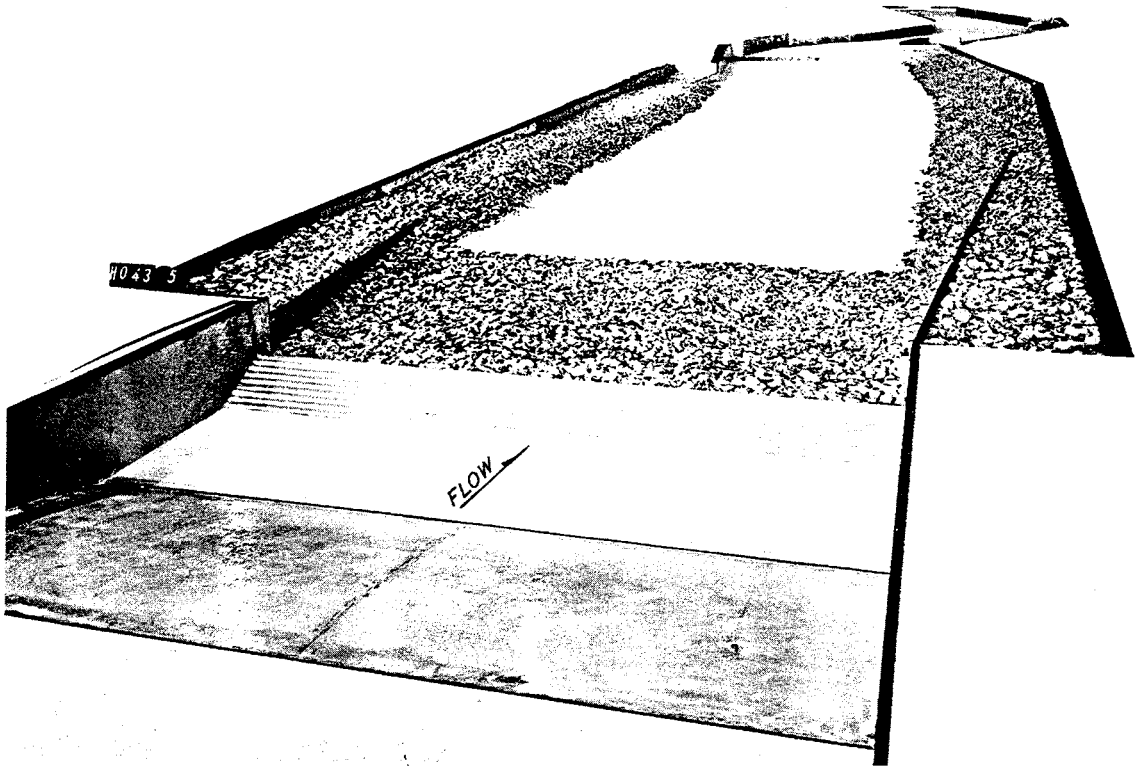
b. Discharge = 22,500 cfs

Photo 2. Debris weir and flow control vanes (sheet 1 of 2)

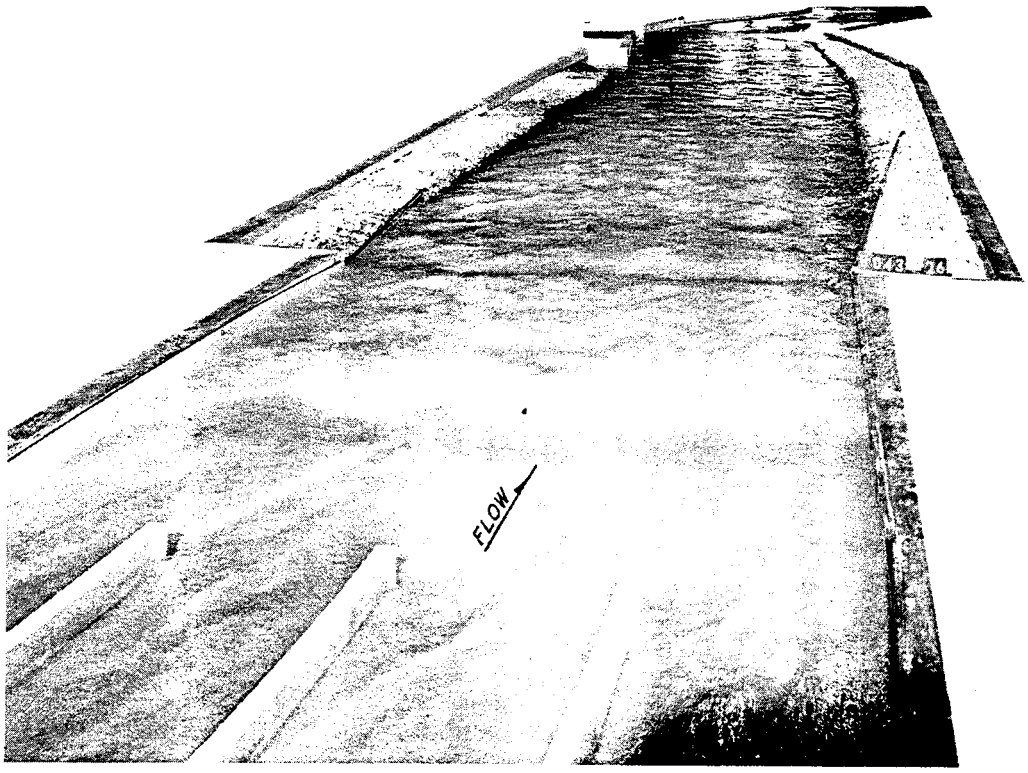


c. Scour between flow control vanes after a project design hydrograph.

Photo 2 (sheet 2 of 2)

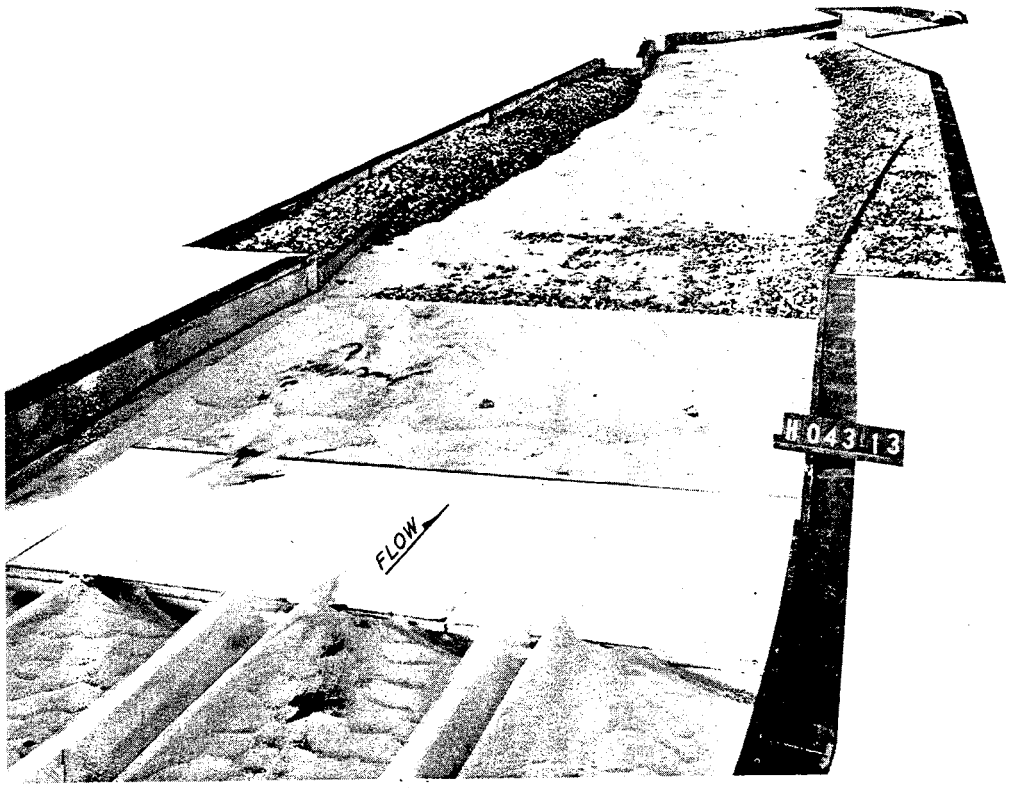


a. Dry bed downstream



b. Discharge = 22,500 cfs

Photo 3. Energy dissipator downstream from flow control vanes (sheet 1 of 2)



c. Sediment trapped in energy dissipator after a project hydrograph

Photo 3 (sheet 2 of 2)

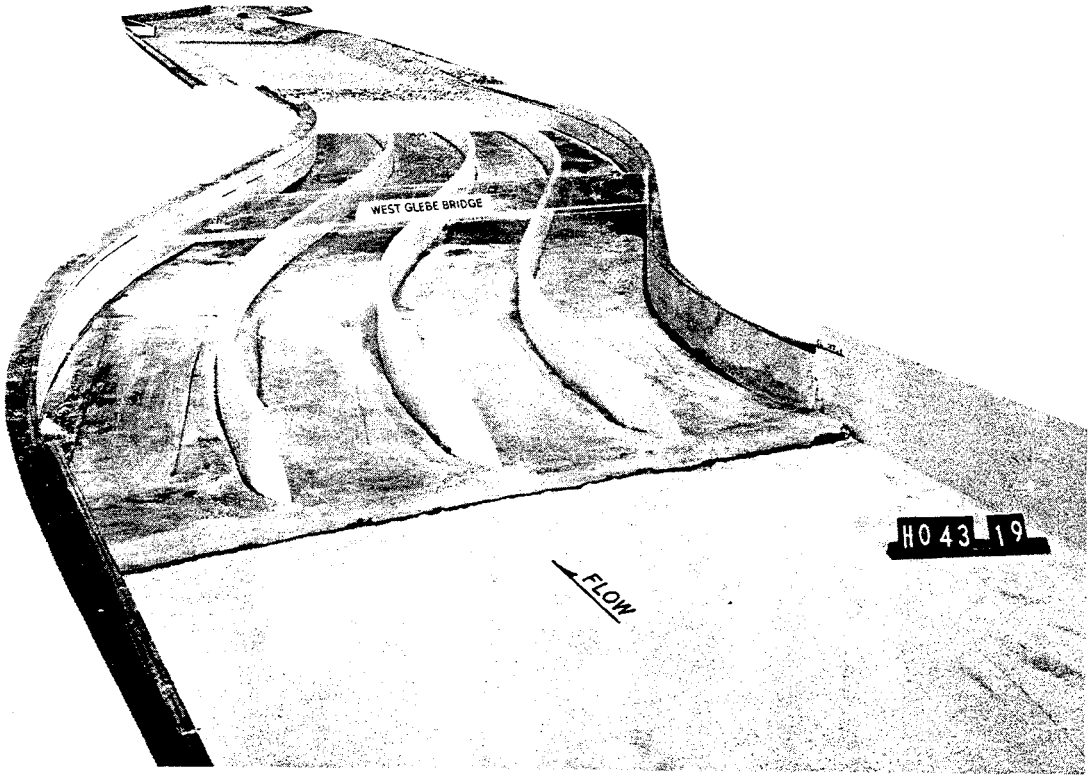
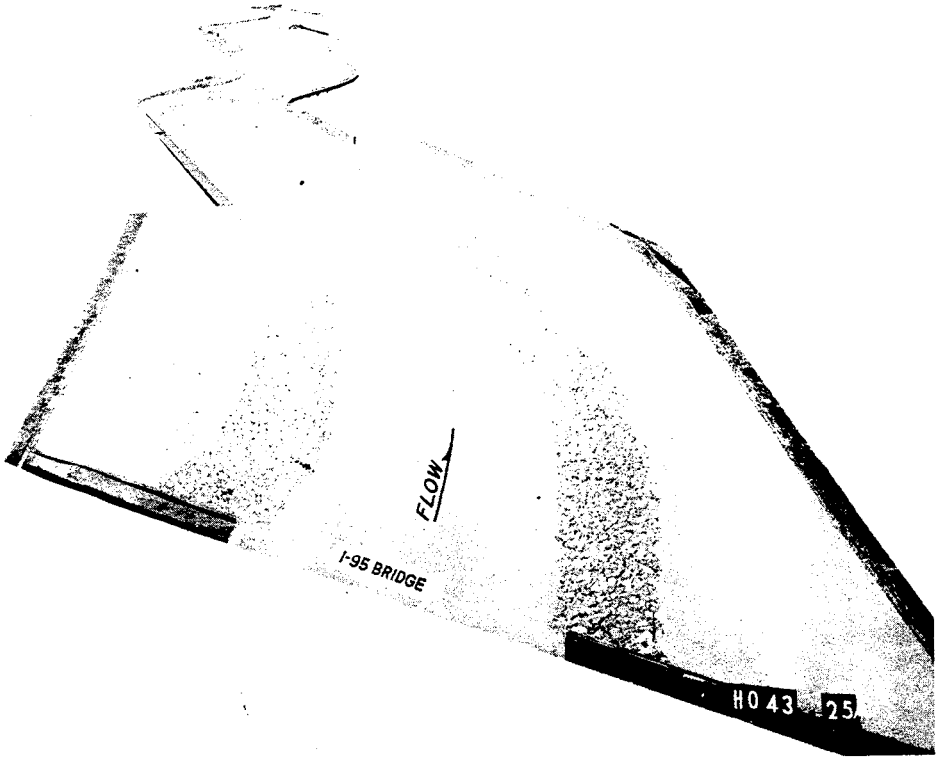
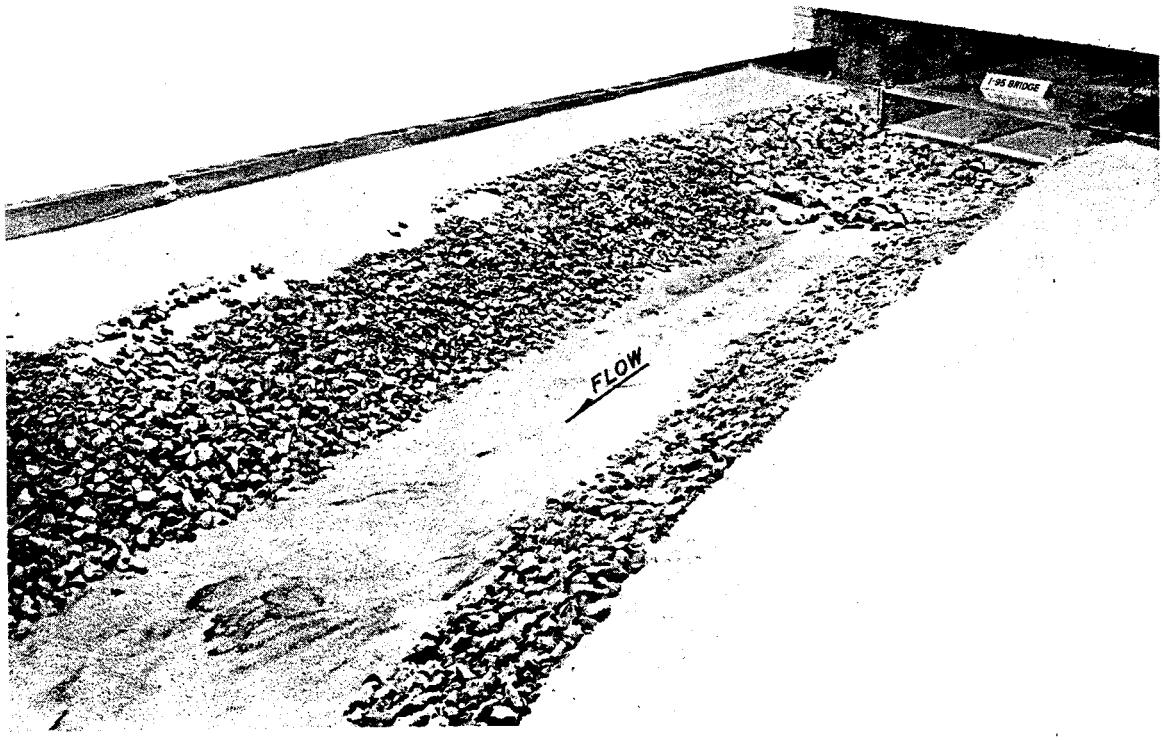


Photo 4. Channel invert paved from debris weir to energy dissipator

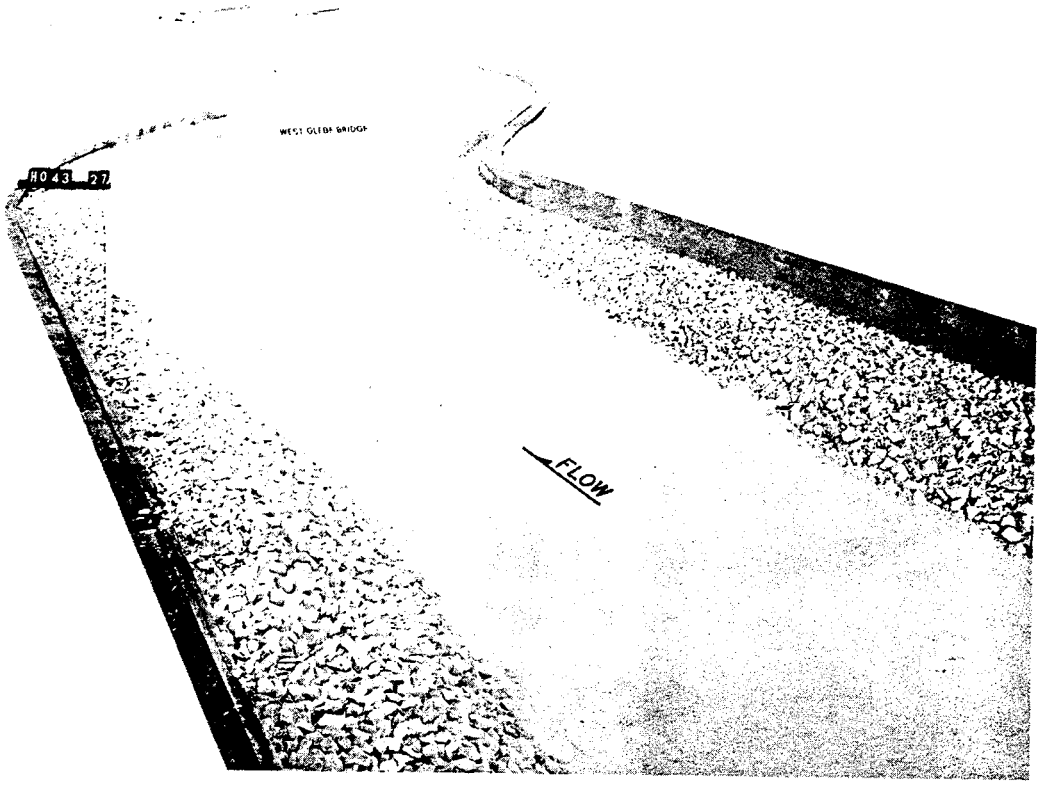


a. Before test

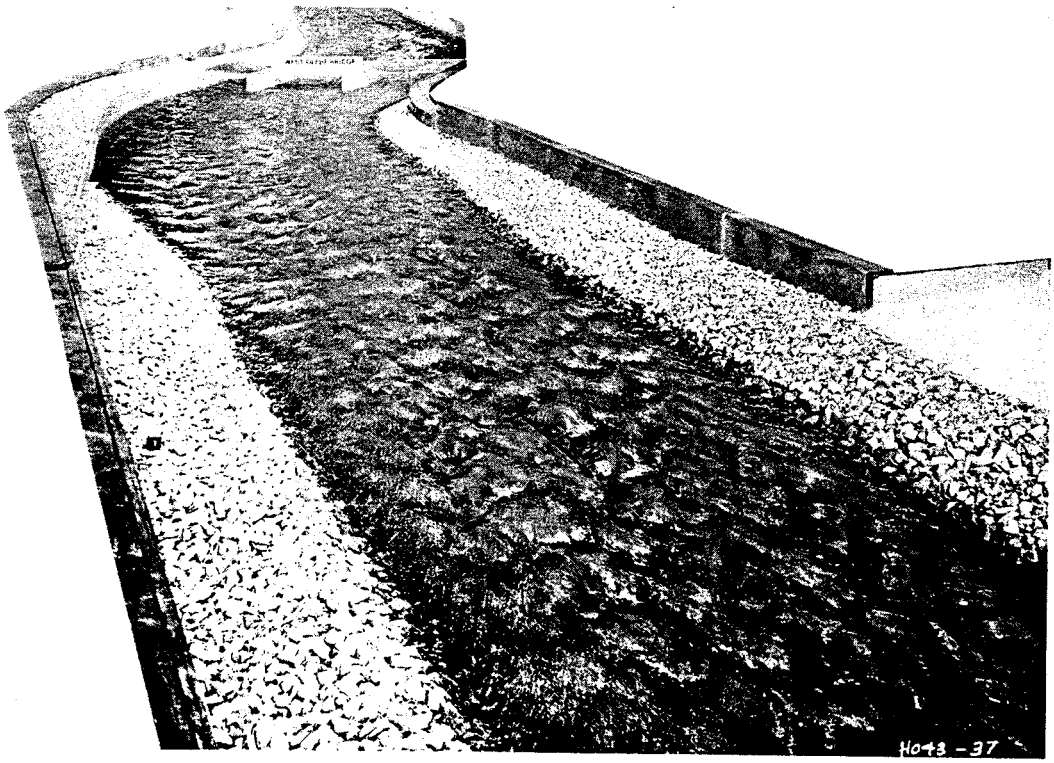


b. After simulated hydrograph. Note riprap failure on bank slopes

Photo 5. Improved natural channel downstream from I-95 Bridge



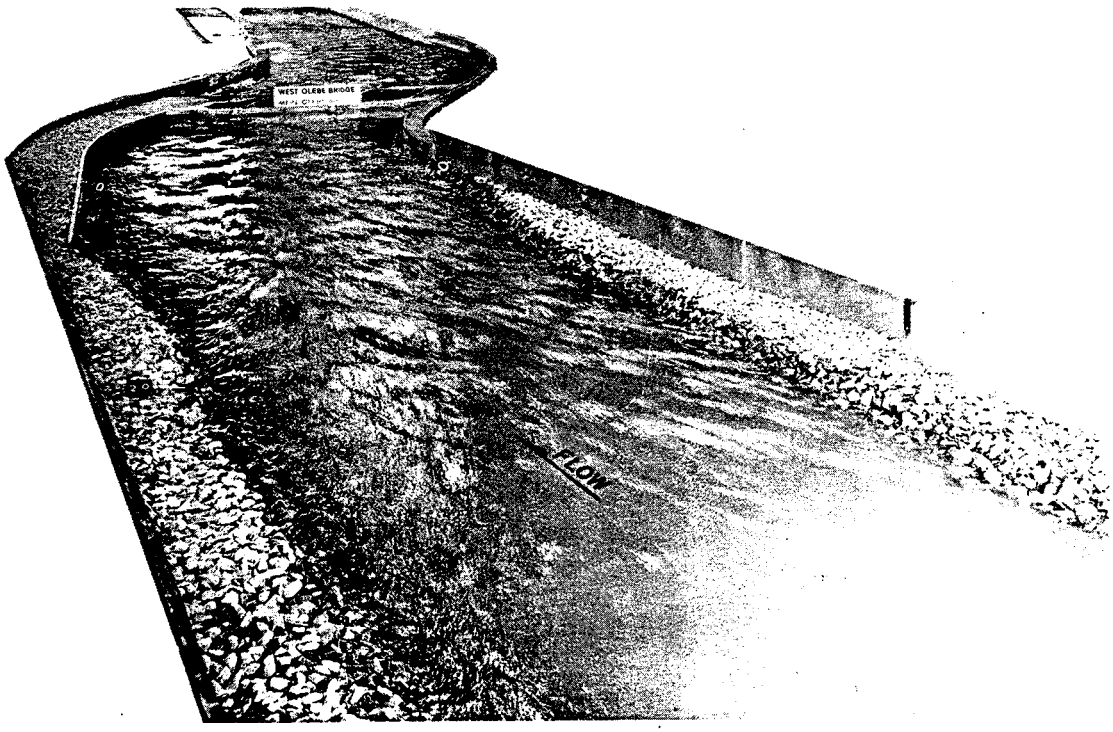
a. Dry bed before test



b. Discharge = 1000 cfs

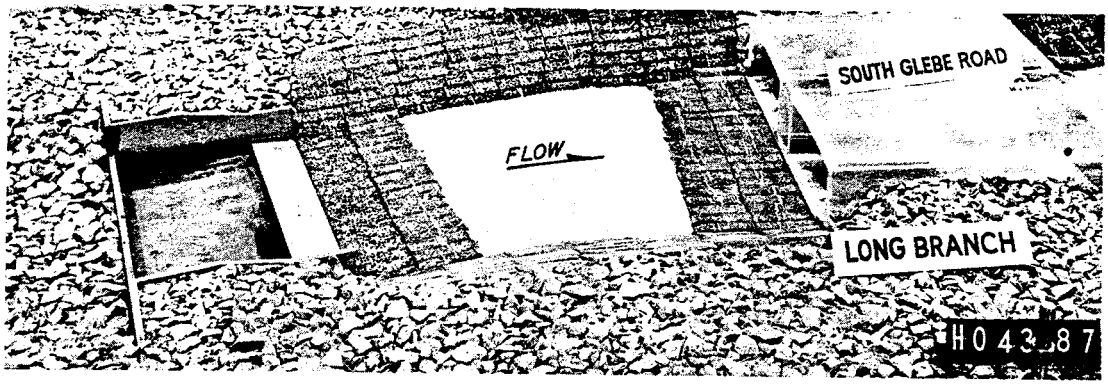
Photo 6. Improved natural channel downstream to West Glebe Road Bridge (sheet 1 of 2)

c. Discharge = 9000 cfs

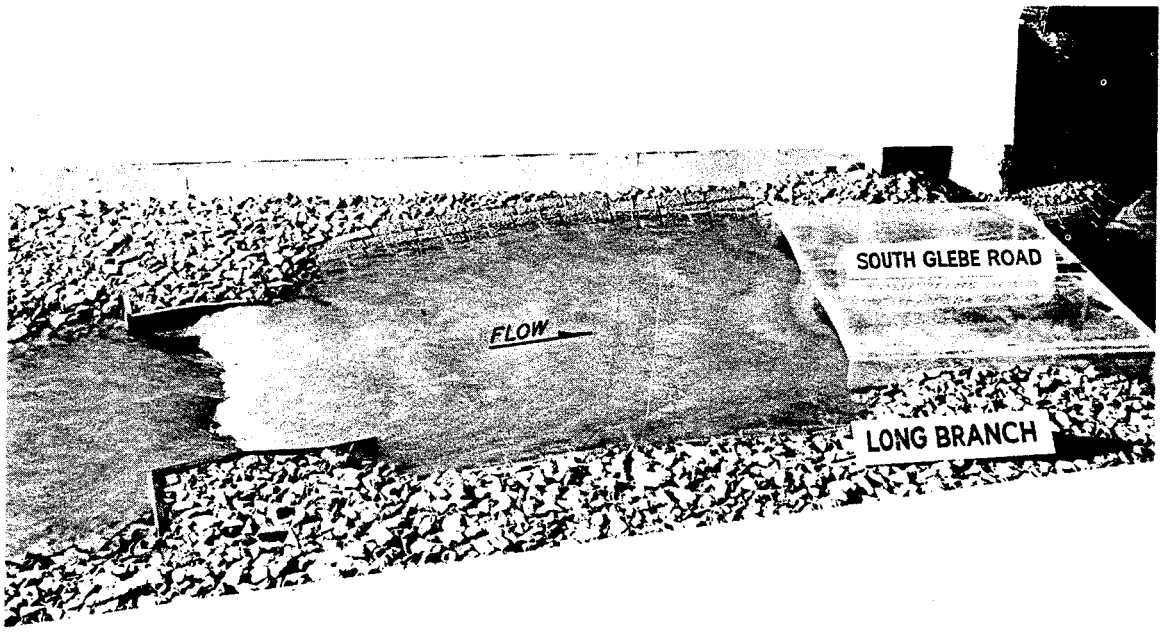


d. Discharge = 20,000 cfs

Photo 6 (sheet 2 of 2)

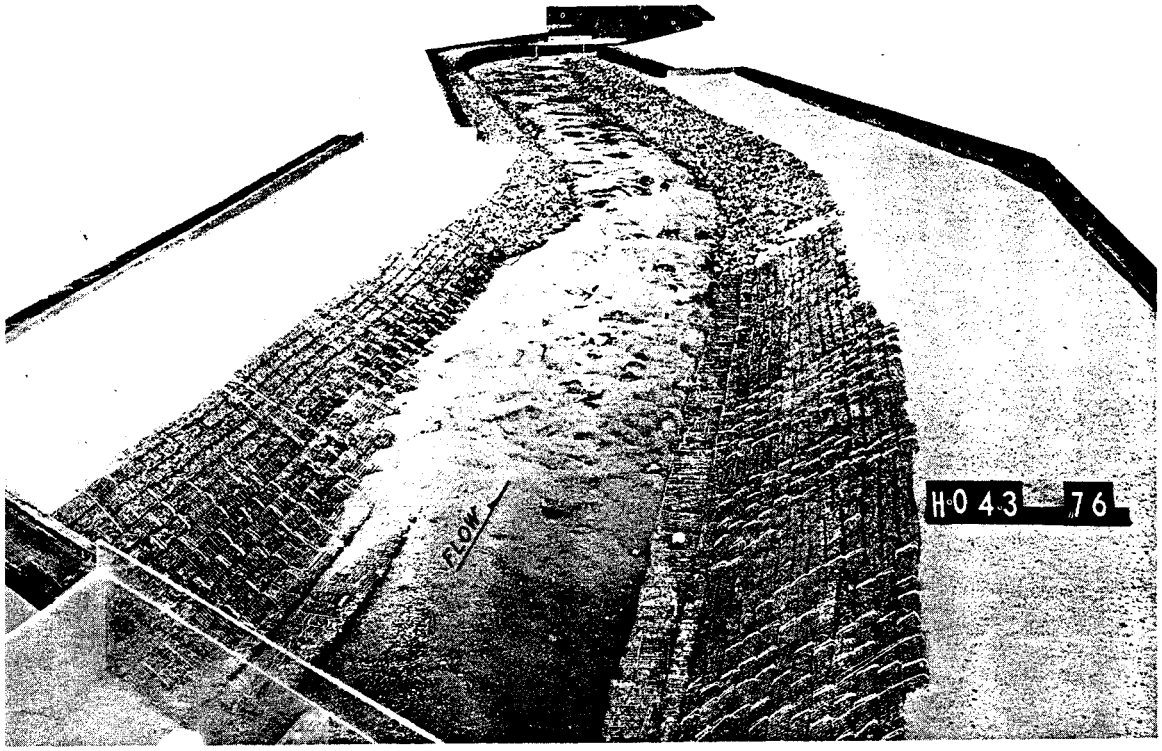


a. Dry bed showing proposed gabion protection plan downstream from drop structure



b. Discharge = 4500 cfs with below minimum tailwater

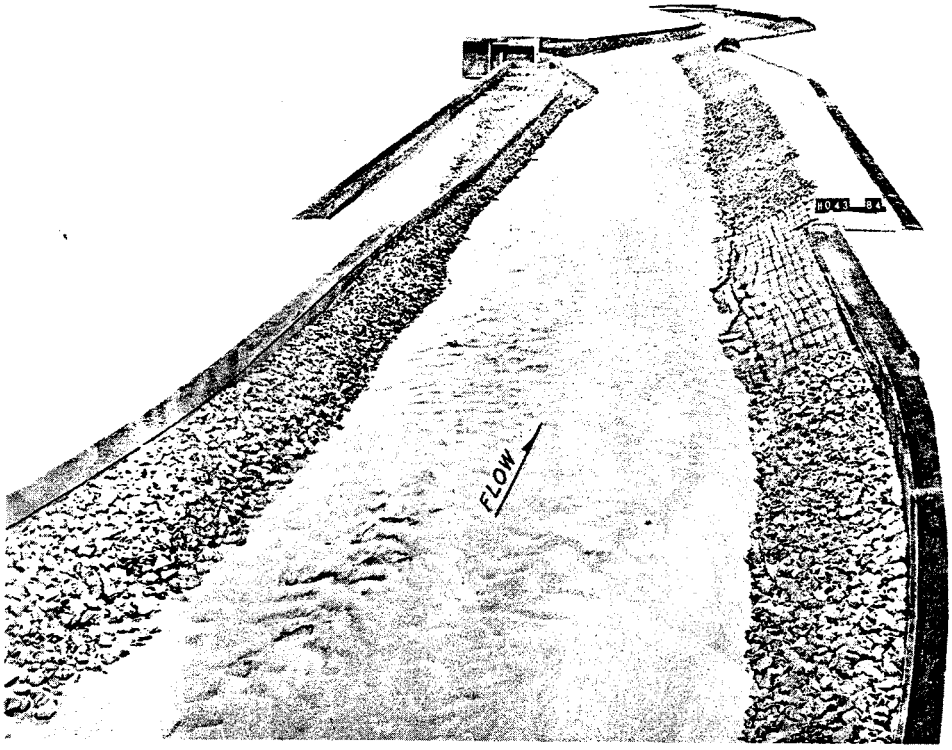
Photo 7. Long Branch improved channel



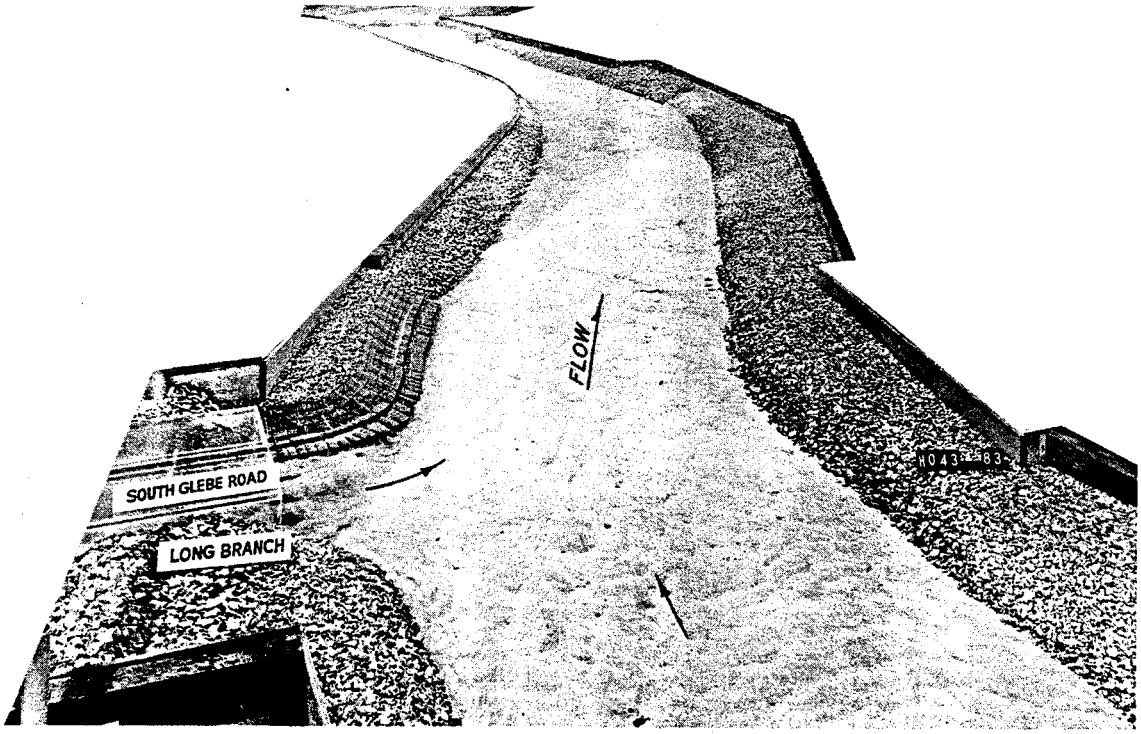
a. Downstream from I-95 Bridge

b. Downstream to West Glebe Road Bridge

Photo 8. Channel bed configuration after a project hydrograph was simulated (sheet 1 of 4)

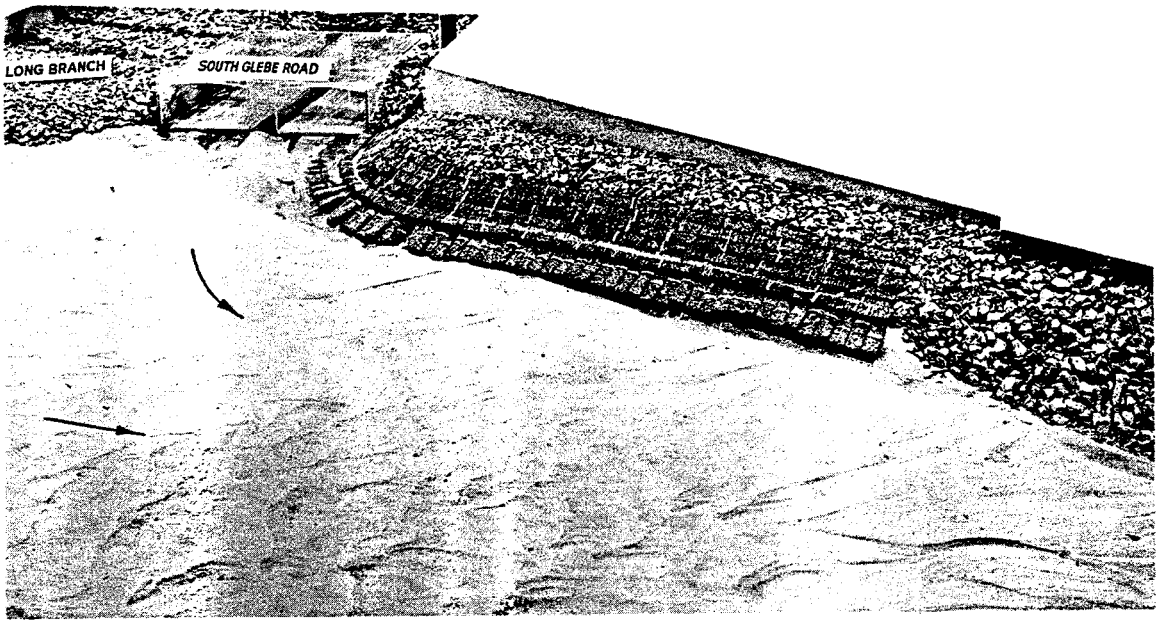


c. Downstream from West Glebe Road Bridge. Note gabions were not wired together

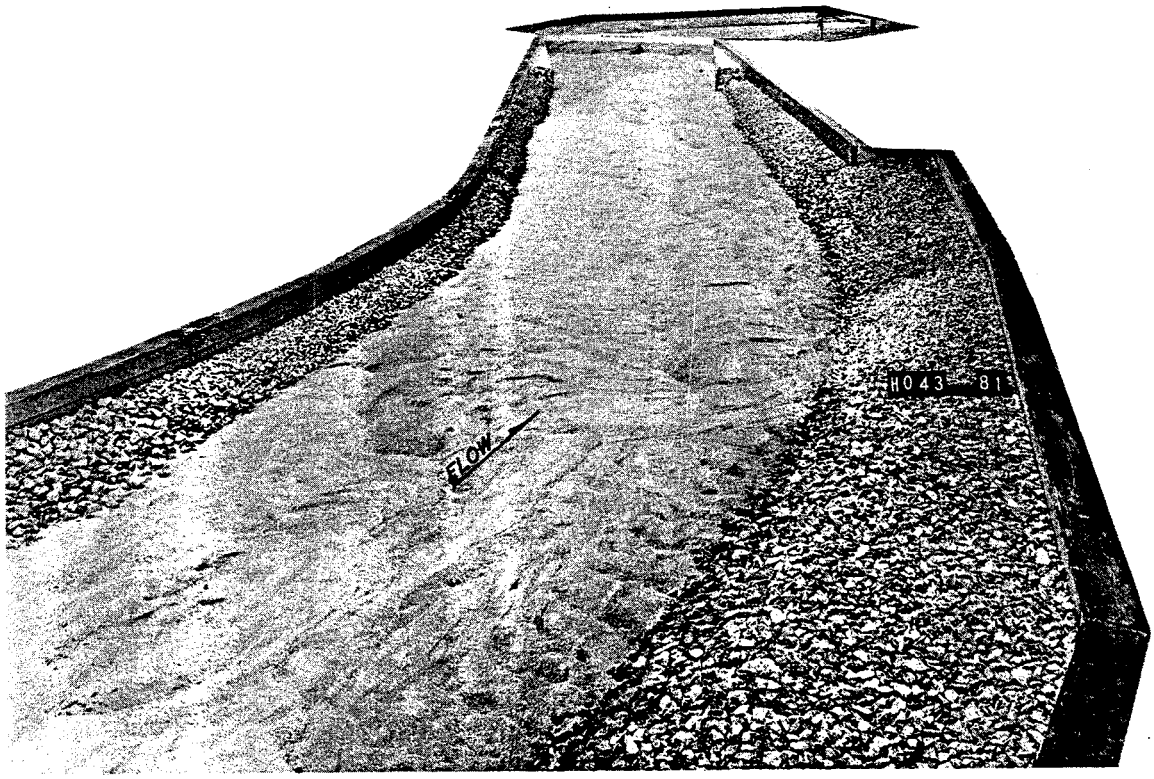


d. Downstream from Long Branch confluence

Photo 8 (sheet 2 of 4)

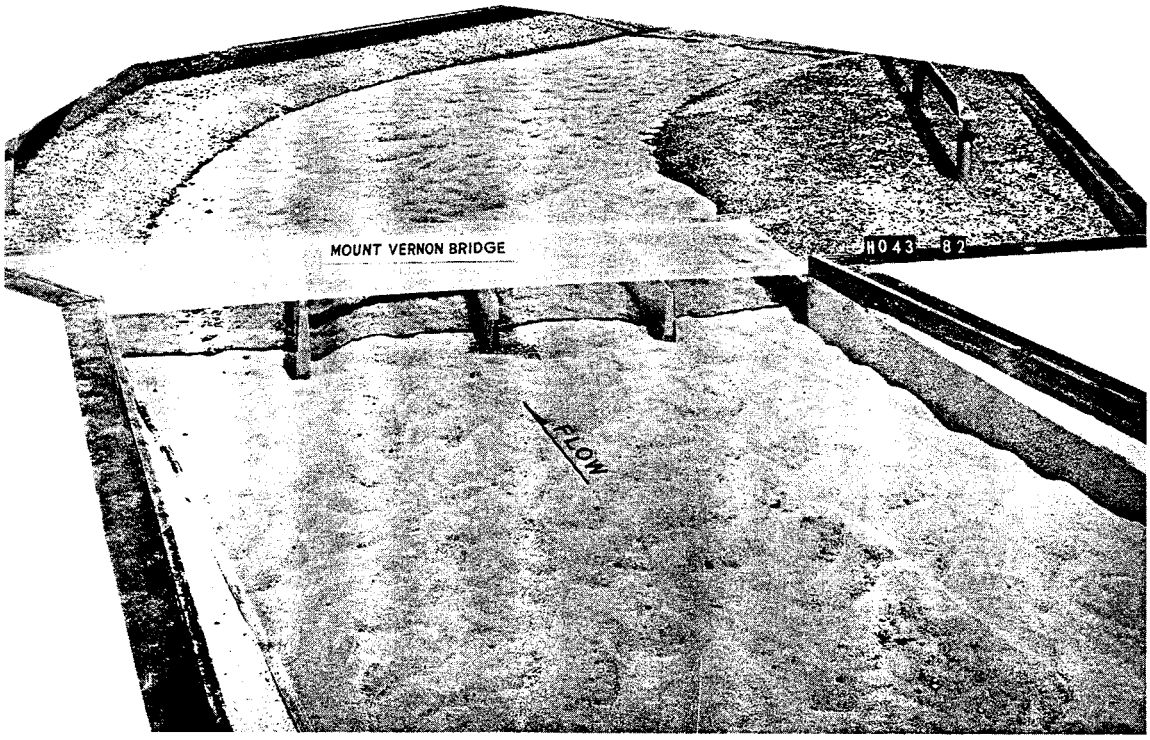


e. Close-up of Long Branch confluence



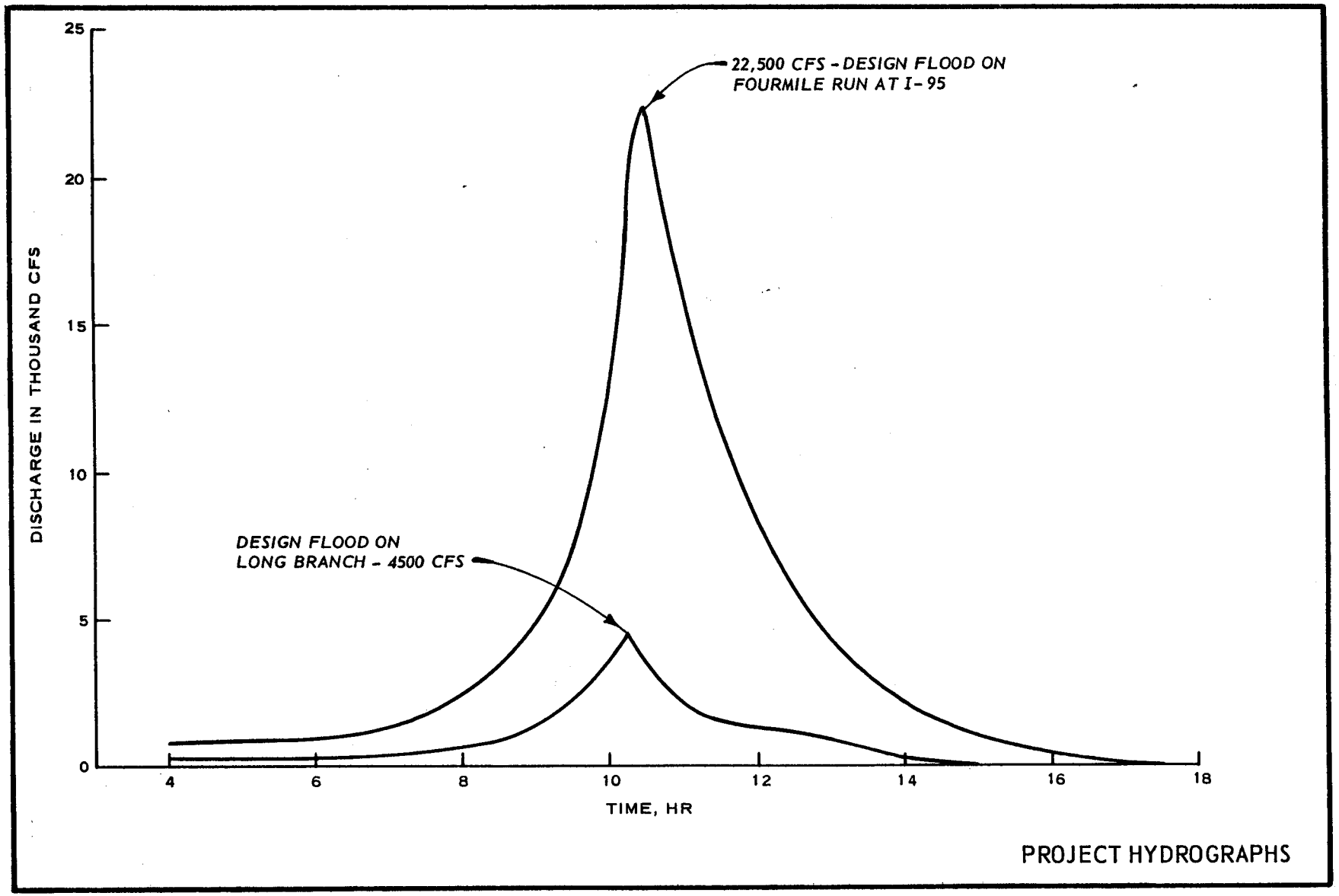
f. Downstream to Mount Vernon Avenue Bridge

Photo 8 (sheet 3 of 4)



g. Close-up of Mount Vernon Avenue Bridge

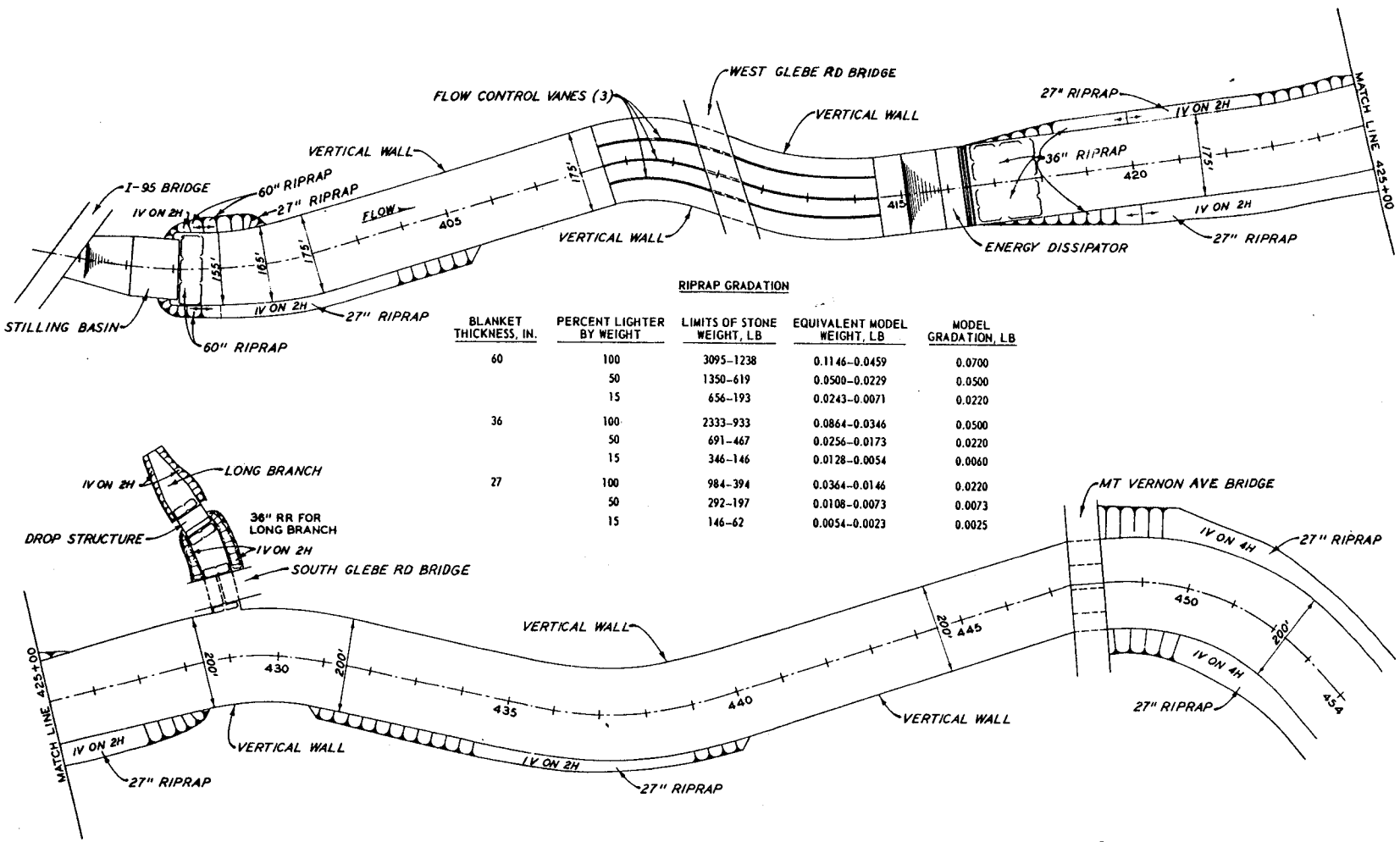
Photo 8 (sheet 4 of 4)



PROJECT HYDROGRAPHS

PLATE 1

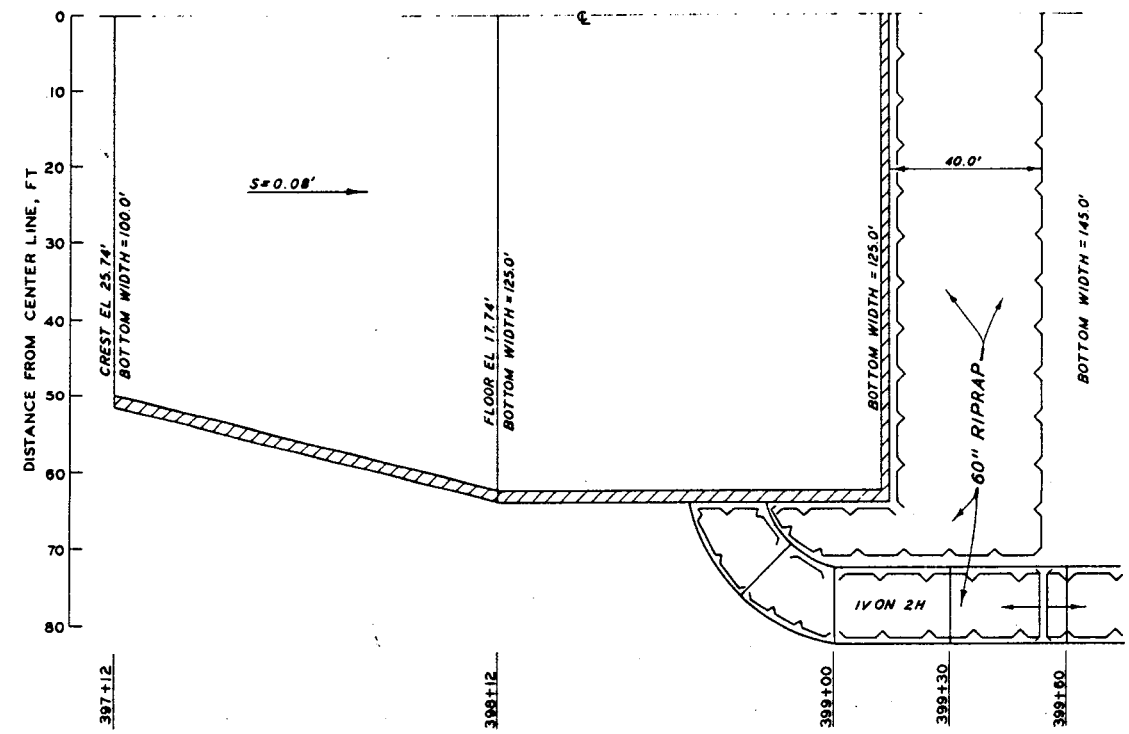
PLATE 2



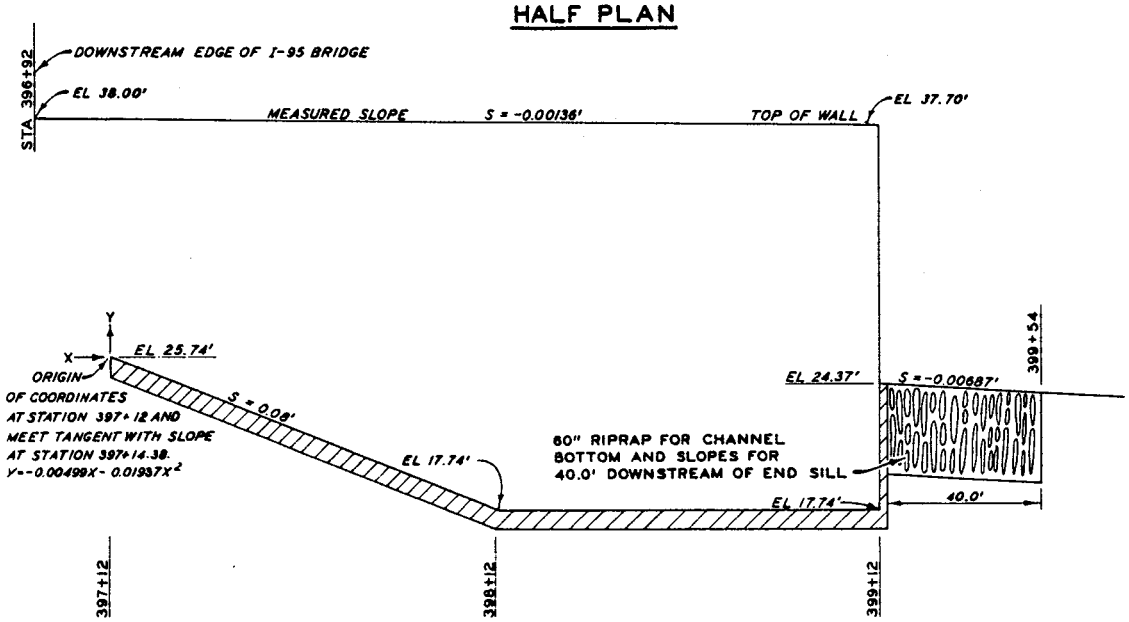
RIPRAP GRADATION

BLANKET THICKNESS, IN.	PERCENT LIGHTER BY WEIGHT	LIMITS OF STONE WEIGHT, LB	EQUIVALENT MODEL WEIGHT, LB	MODEL GRADATION, LB
60	100	3095–1238	0.1146–0.0459	0.0700
	50	1350–619	0.0500–0.0229	0.0500
	15	656–193	0.0243–0.0071	0.0220
36	100	2333–933	0.0864–0.0346	0.0500
	50	691–467	0.0256–0.0173	0.0220
	15	346–146	0.0128–0.0054	0.0060
27	100	984–394	0.0364–0.0146	0.0220
	50	292–197	0.0108–0.0073	0.0073
	15	146–62	0.0054–0.0023	0.0025

ORIGINAL DESIGN WITH HYDRAULIC STRUCTURES

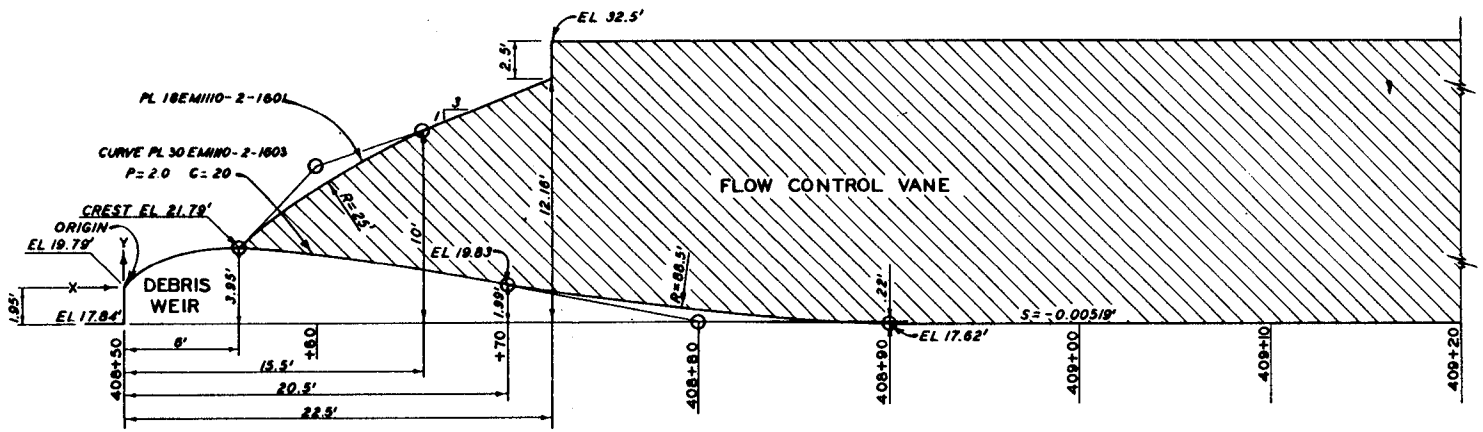


HALF PLAN



ELEVATION

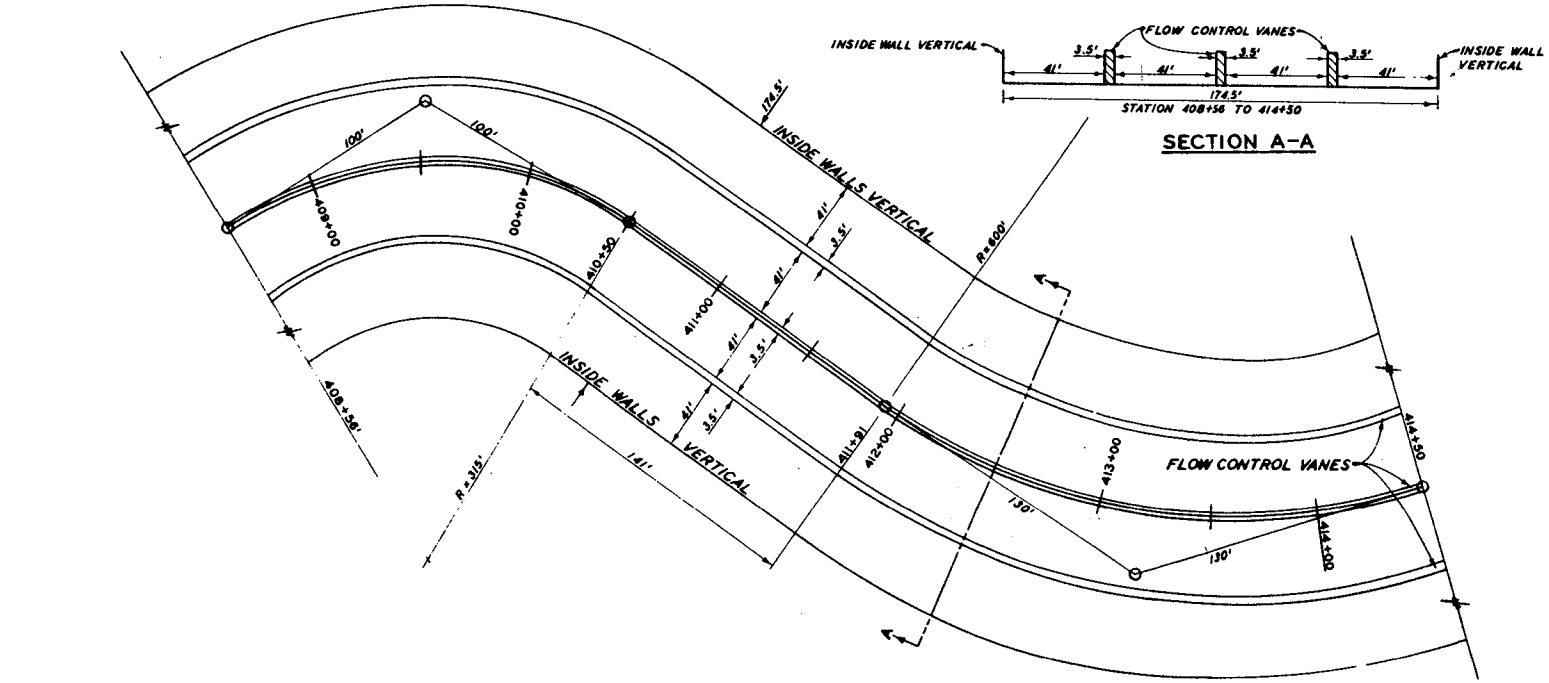
I-95 STILLING BASIN



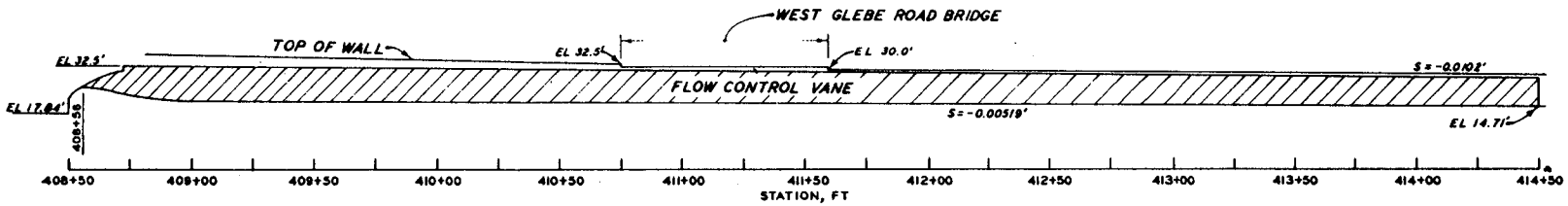
ELEVATION

DEBRIS WEIR
STATION 408+50

PLATE 4



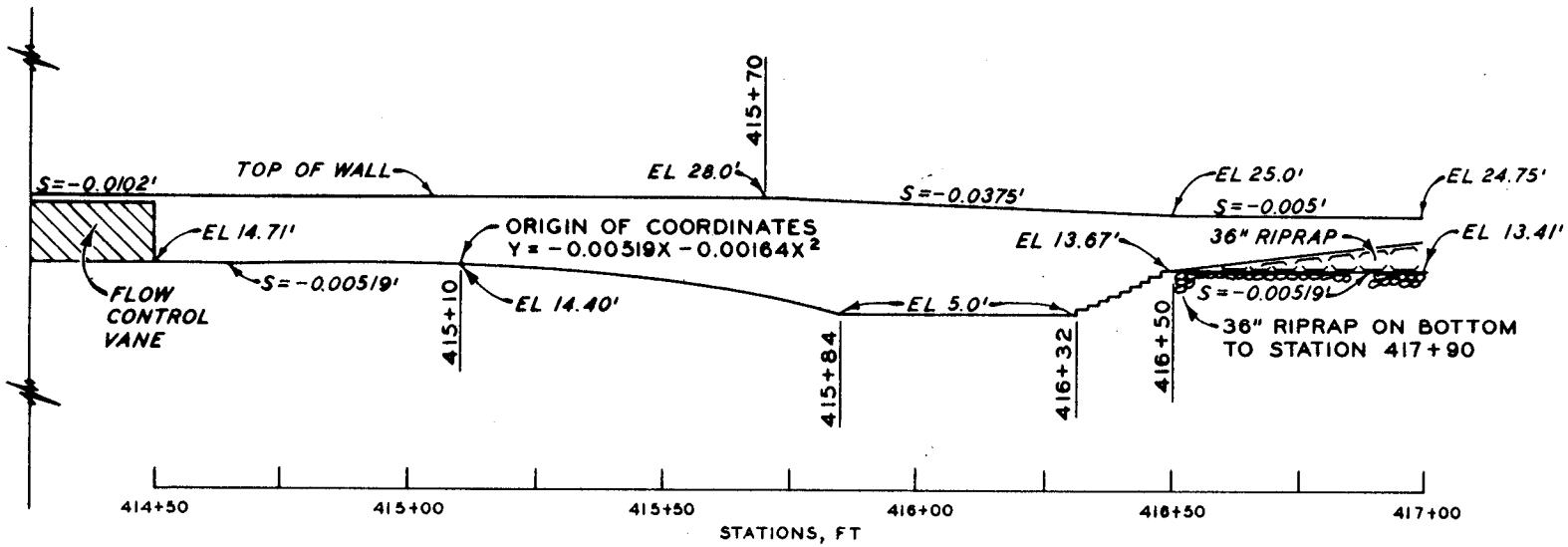
PLAN



ELEVATION

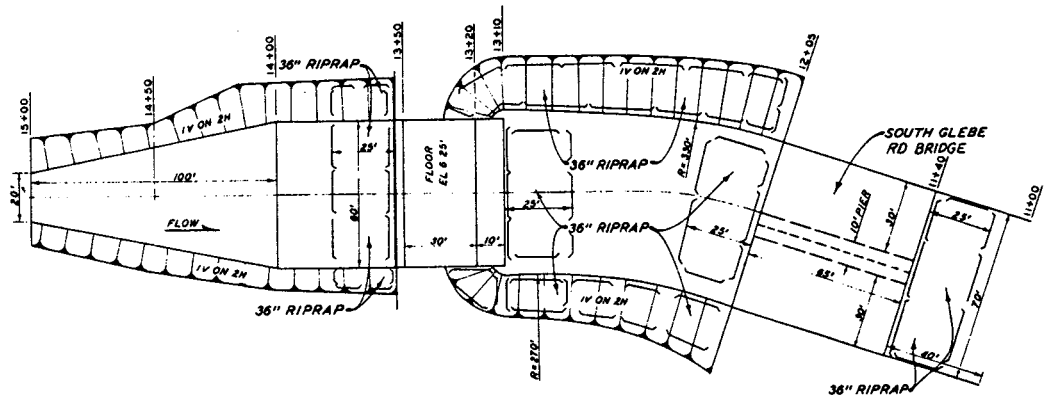
FLOW CONTROL VANES
STA 408+56 TO STA 414+50

PLATE 5



ELEVATION

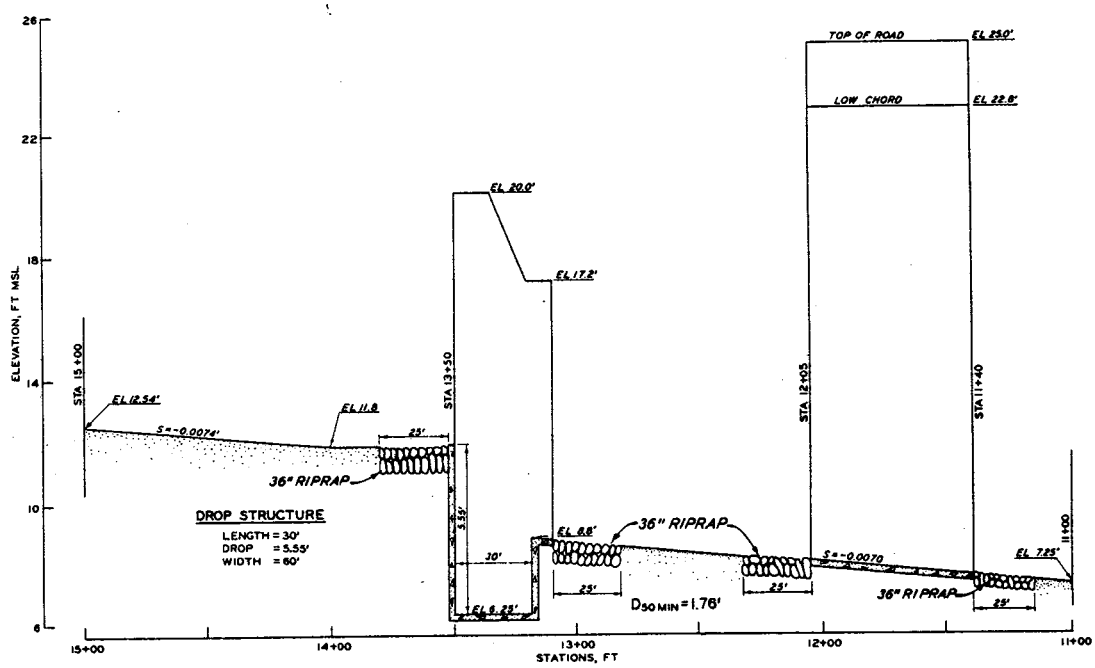
ENERGY DISSIPATOR DOWNSTREAM OF FLOW CONTROL VANES



PLAN

RIPRAP GRADATION

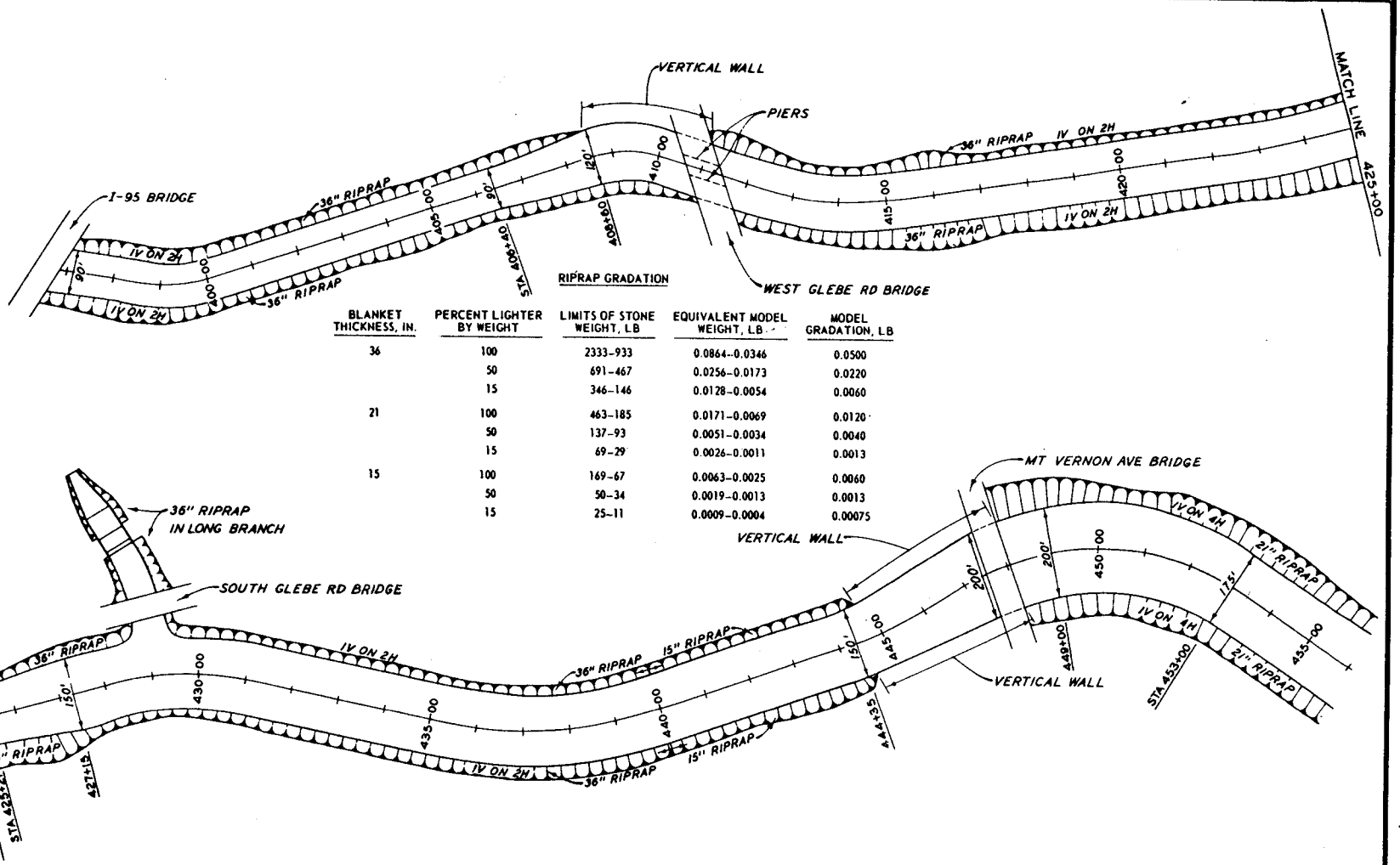
BLANKET THICKNESS, IN.	PERCENT LIGHTER BY WEIGHT	LIMITS OF STONE WEIGHT, LB	EQUIVALENT MODEL WEIGHT, LB	MODEL GRADATION, LB
36	100	2333–933	0.0864–0.0346	0.0500
	50	691–467	0.0256–0.0173	0.0220
	15	346–146	0.0128–0.0054	0.0060



PROFILE

LONG BRANCH

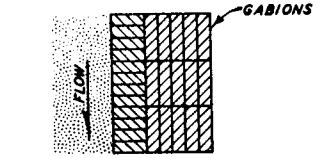
DROP STRUCTURE AND CHANNEL PROTECTION ORIGINALLY PROPOSED



RIPRAP GRADATION

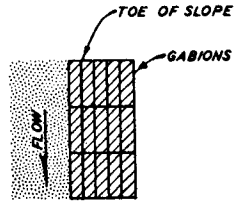
BLANKET THICKNESS, IN.	PERCENT LIGHTER BY WEIGHT	LIMITS OF STONE WEIGHT, LB	EQUIVALENT MODEL WEIGHT, LB	MODEL GRADATION, LB
36	100	2333–933	0.0864–0.0346	0.0500
	50	691–467	0.0256–0.0173	0.0220
	15	346–146	0.0128–0.0054	0.0060
21	100	463–185	0.0171–0.0069	0.0120
	50	137–93	0.0051–0.0034	0.0040
	15	69–29	0.0026–0.0011	0.0013
15	100	169–67	0.0063–0.0025	0.0060
	50	50–34	0.0019–0.0013	0.0013
	15	25–11	0.0009–0.0004	0.00075

IMPROVED NATURAL CHANNEL
ORIGINAL BANKSLOPE PROTECTION



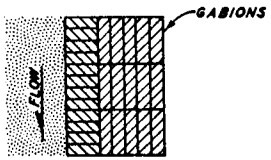
PLAN

ELEVATION
TYPE I TOE PROTECTION

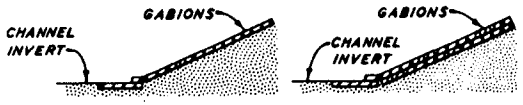


PLAN

ELEVATION
TYPE II TOE PROTECTION



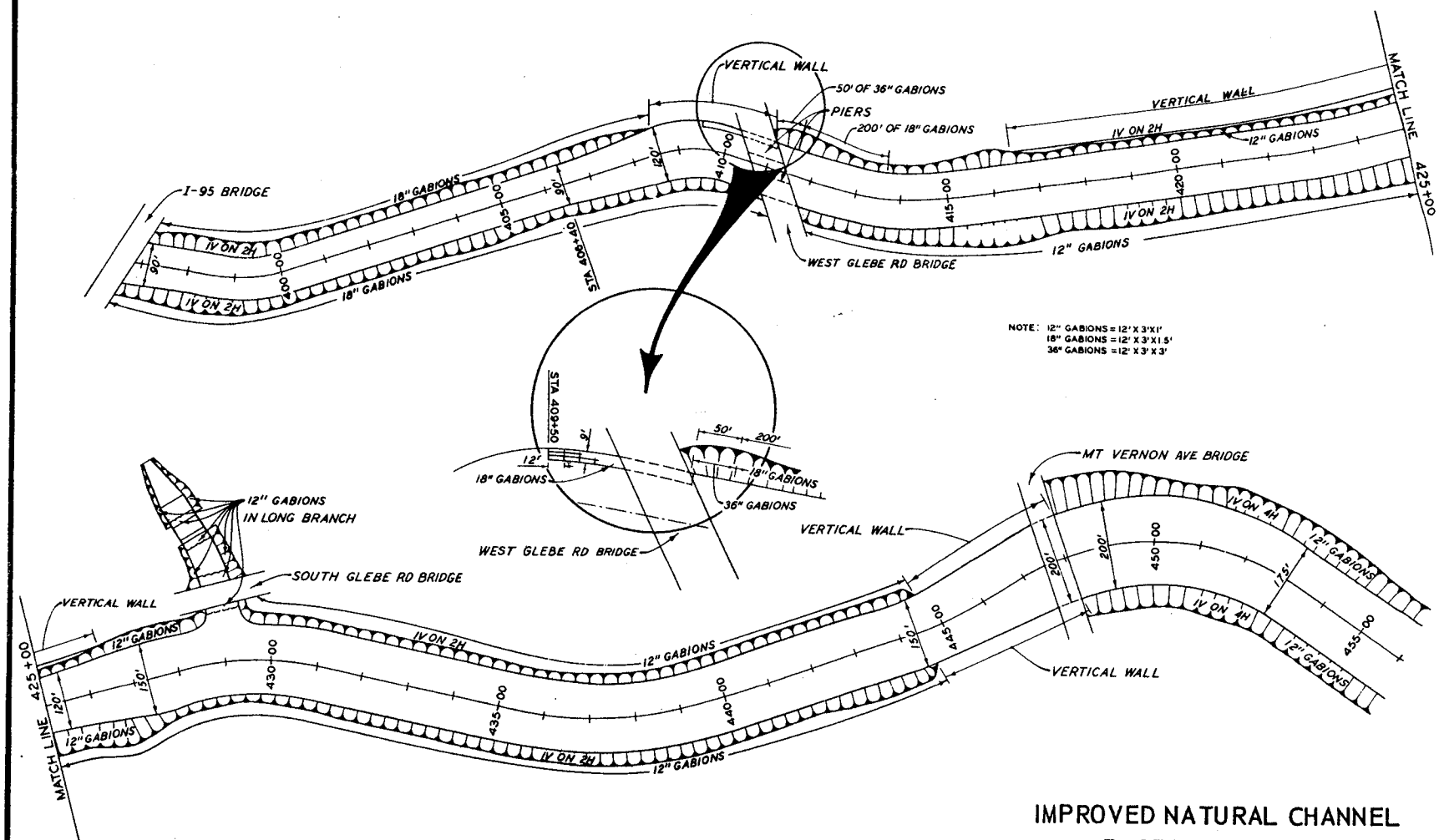
PLAN



ELEVATION
TYPE III TOE PROTECTION

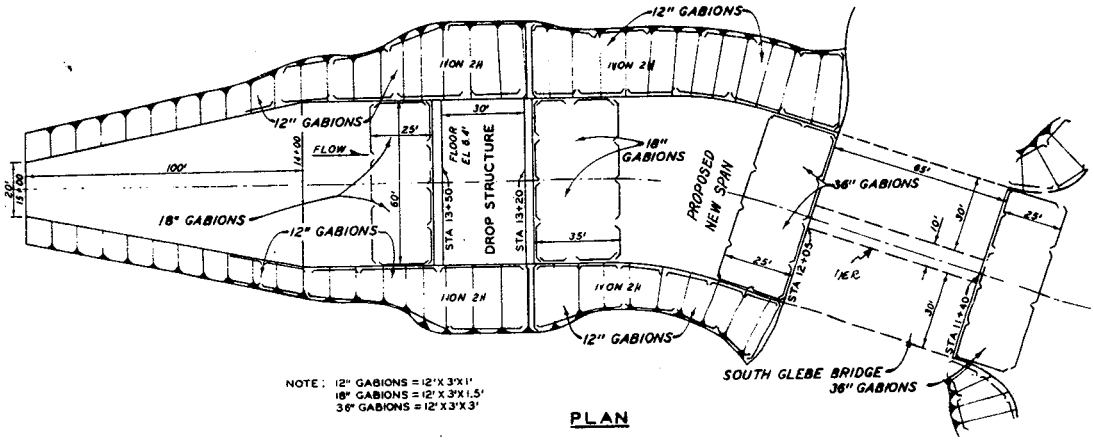
NOTE: FILTER CLOTH IS PLACED BETWEEN GABIONS AND SAND

TYPES OF TOE PROTECTION TESTED

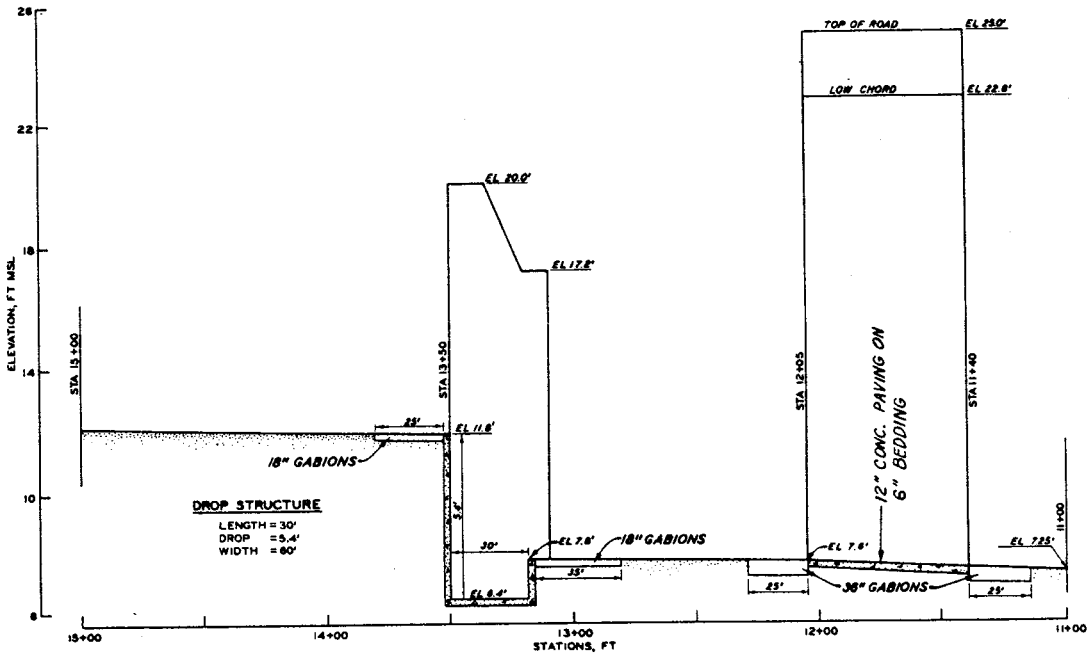


IMPROVED NATURAL CHANNEL
PROPOSED GABION
PROTECTION PLAN

PLATE 10



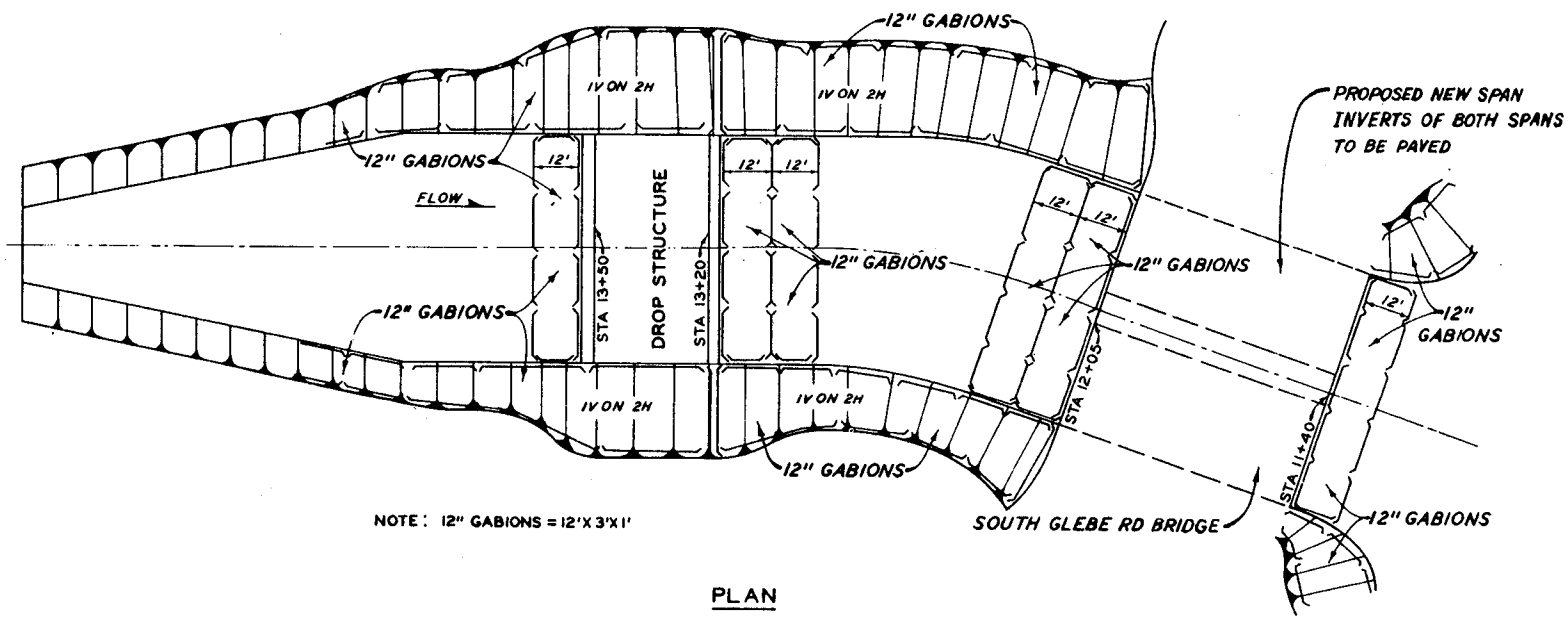
PLAN



PROFILE

LONG BRANCH

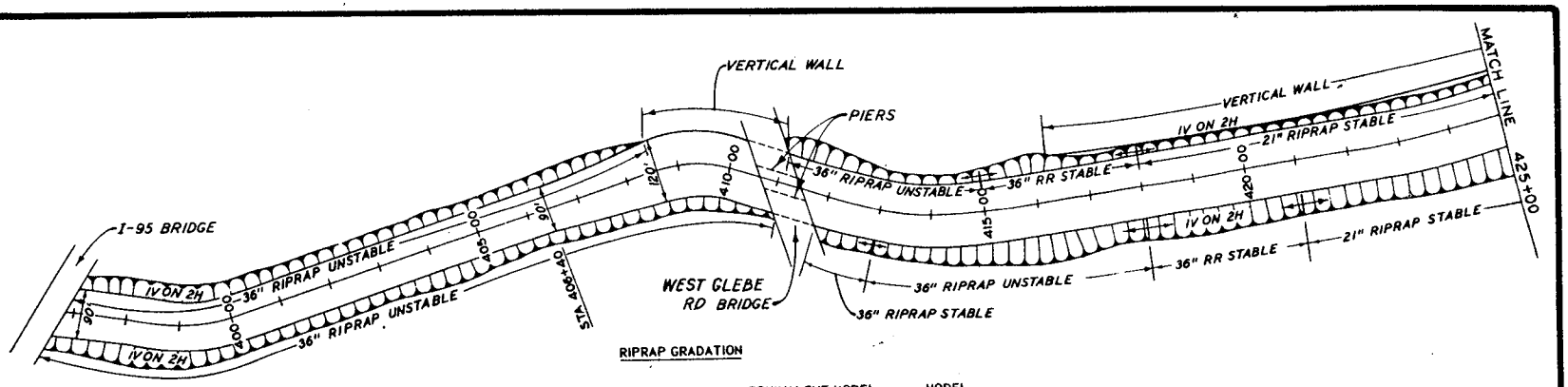
DROP STRUCTURE AND CHANNEL PROTECTION PROPOSED WITH IMPROVED NATURAL CHANNEL



PLAN

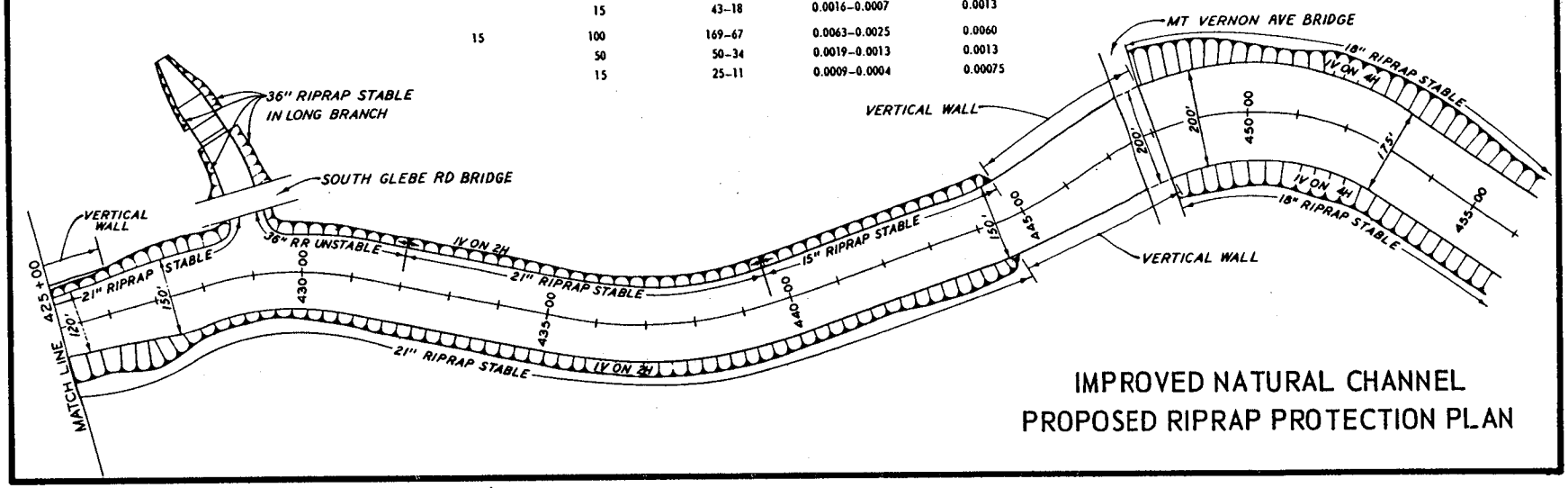
LONG BRANCH

PROPOSED GABION
PROTECTION PLAN

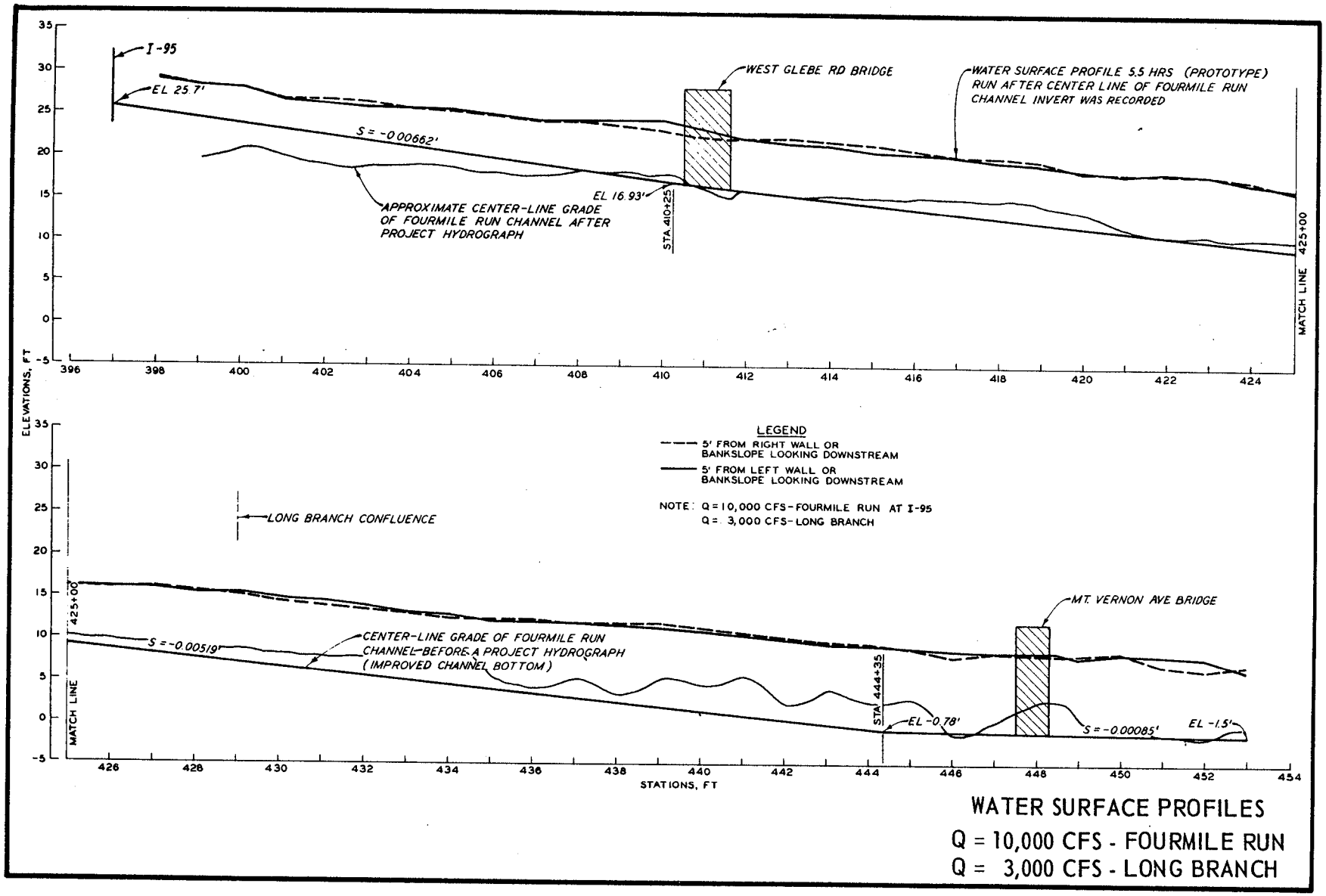


RIPRAP GRADATION

BLANKET THICKNESS, IN.	PERCENT LIGHTER BY WEIGHT	LIMITS OF STONE WEIGHT, LB	EQUIVALENT MODEL WEIGHT, LB	MODEL GRADATION, LB
36	100	2333–933	0.0864–0.0346	0.0500
	50	691–467	0.0256–0.0173	0.0220
	15	346–146	0.0128–0.0054	0.0060
21	100	463–185	0.0171–0.0069	0.0120
	50	137–93	0.0051–0.0034	0.0040
	15	69–29	0.0026–0.0011	0.0013
18	100	292–117	0.0108–0.0043	0.0060
	50	86–58	0.0032–0.0021	0.0025
	15	43–18	0.0016–0.0007	0.0013
15	100	169–67	0.0063–0.0025	0.0060
	50	50–34	0.0019–0.0013	0.0013
	15	25–11	0.0009–0.0004	0.00075



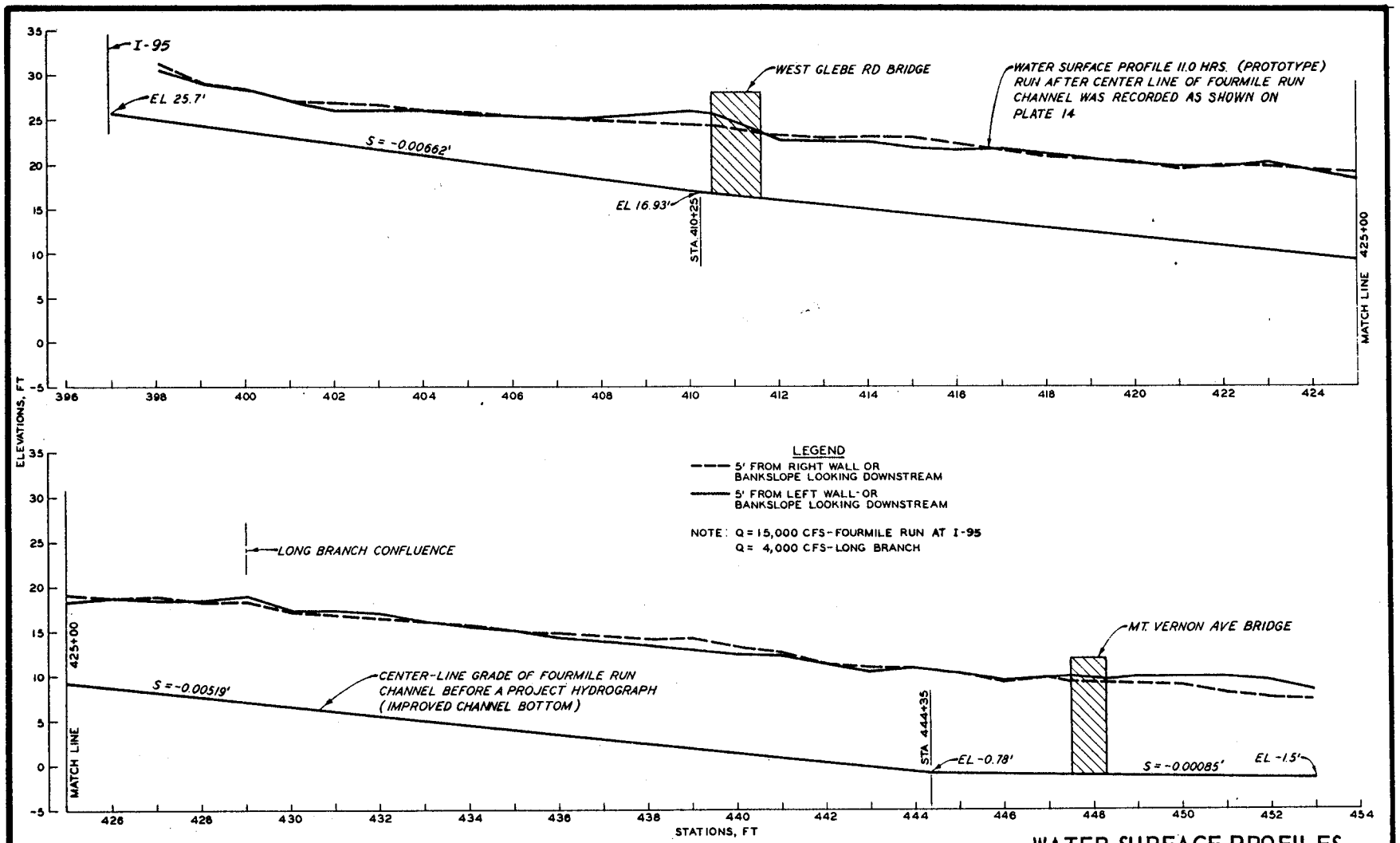
IMPROVED NATURAL CHANNEL
PROPOSED RIPRAP PROTECTION PLAN



WATER SURFACE PROFILES

Q = 10,000 CFS - FOURMILE RUN

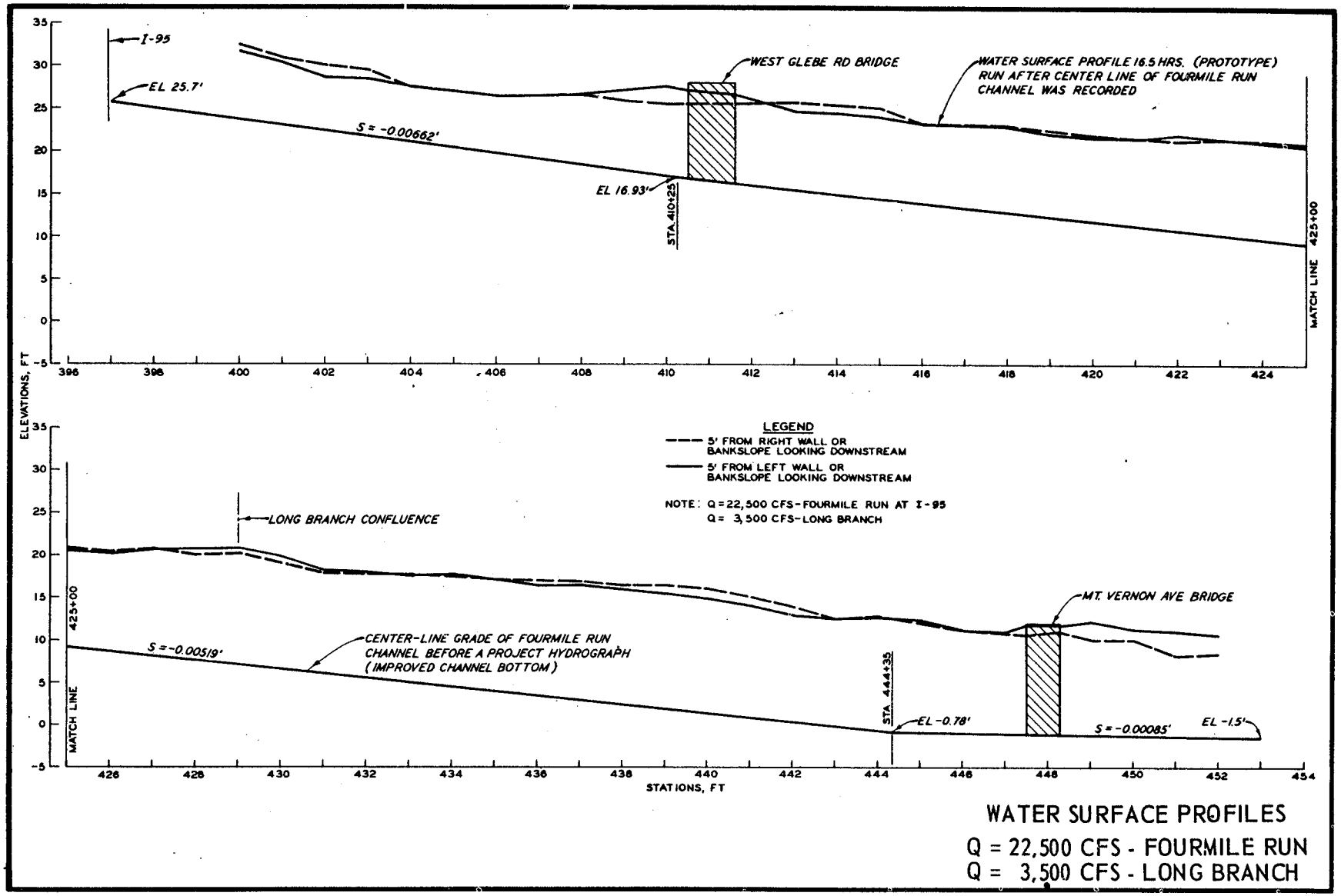
Q = 3,000 CFS - LONG BRANCH



WATER SURFACE PROFILES

Q = 15,000 CFS - FOURMILE RUN

Q = 4,000 CFS - LONG BRANCH

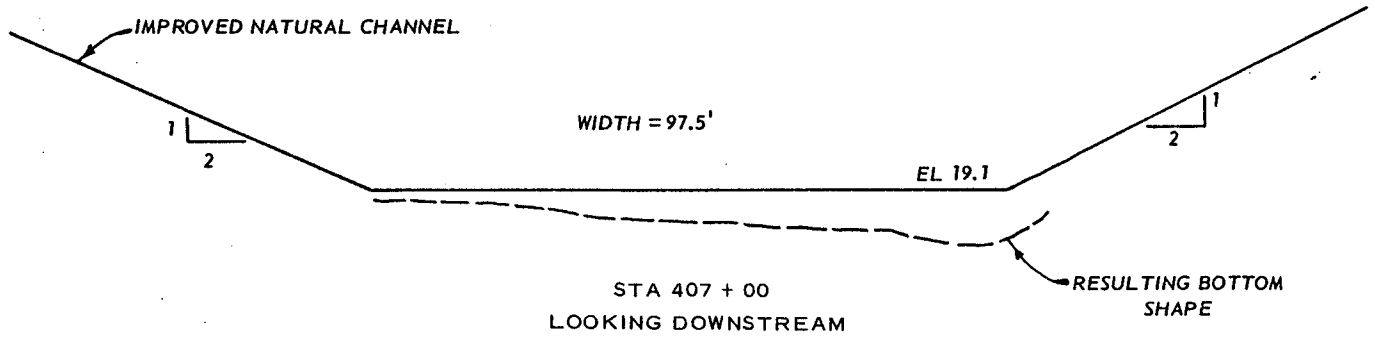


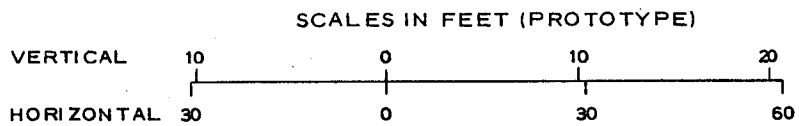
WATER SURFACE PROFILES

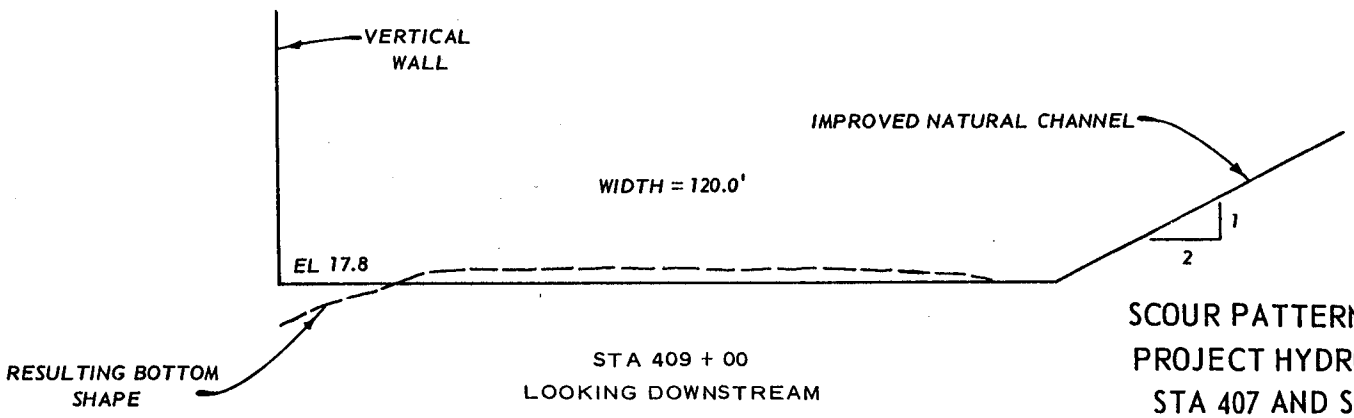
Q = 22,500 CFS - FOURMILE RUN

Q = 3,500 CFS - LONG BRANCH

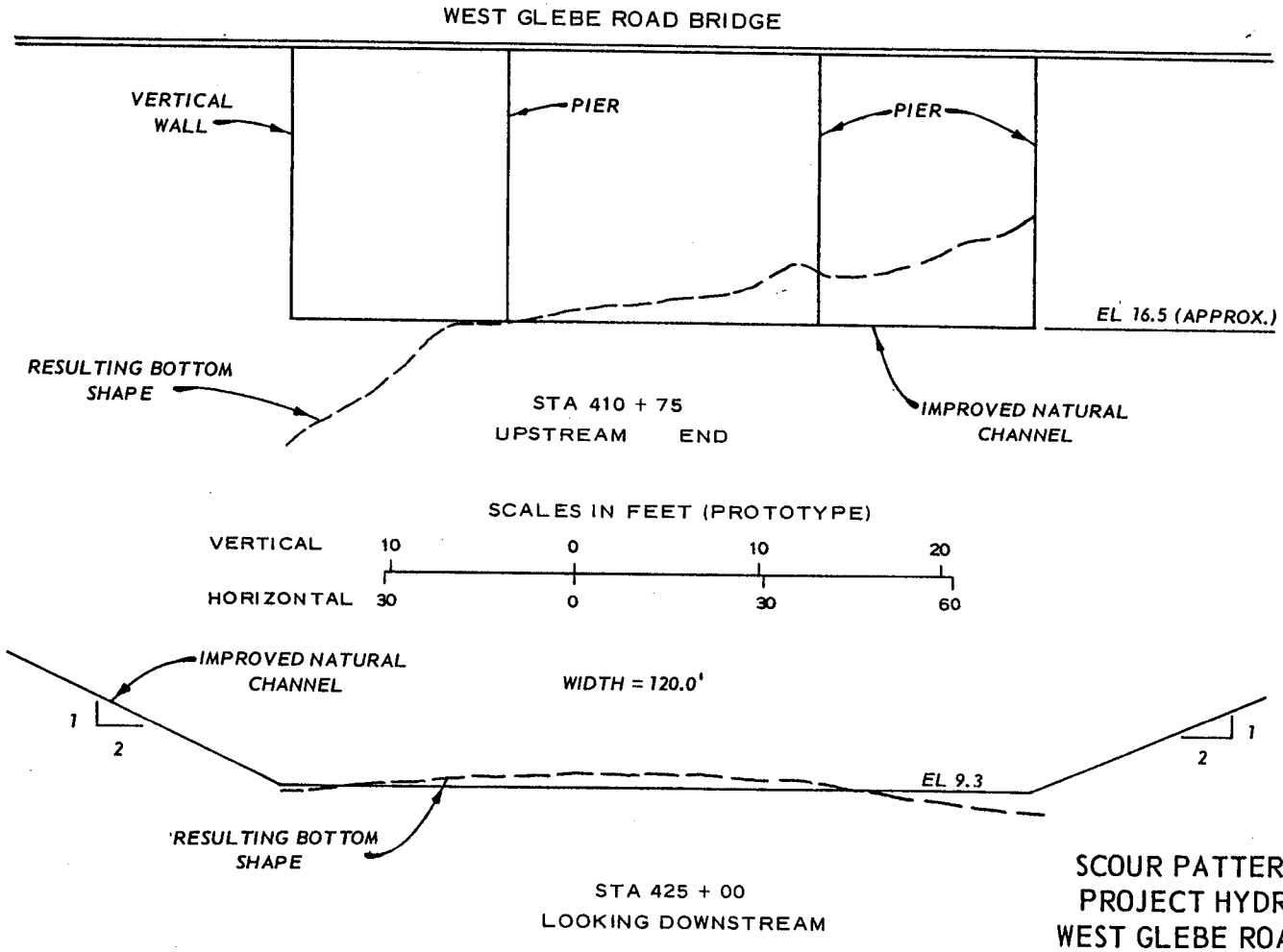
PLATE 16



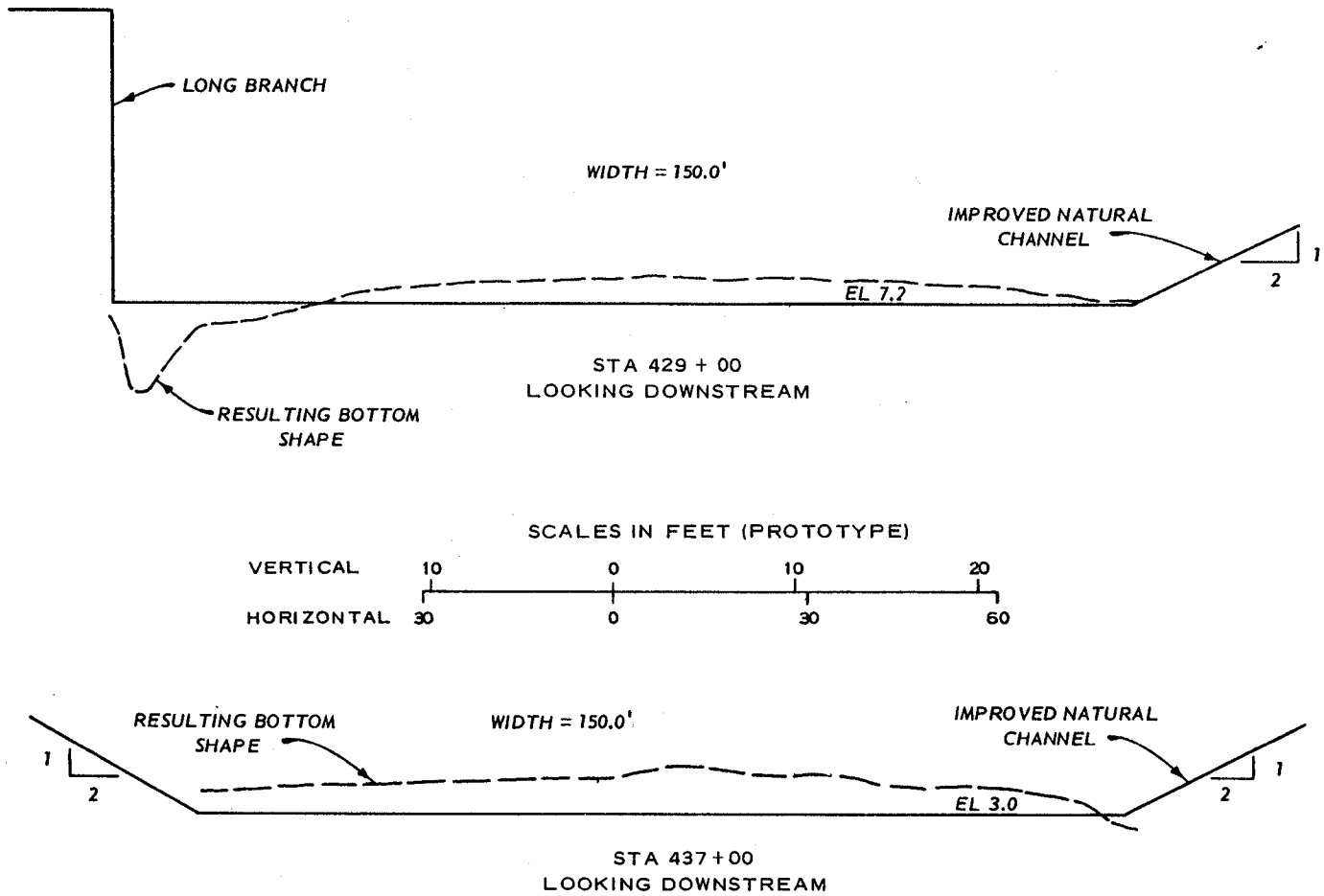




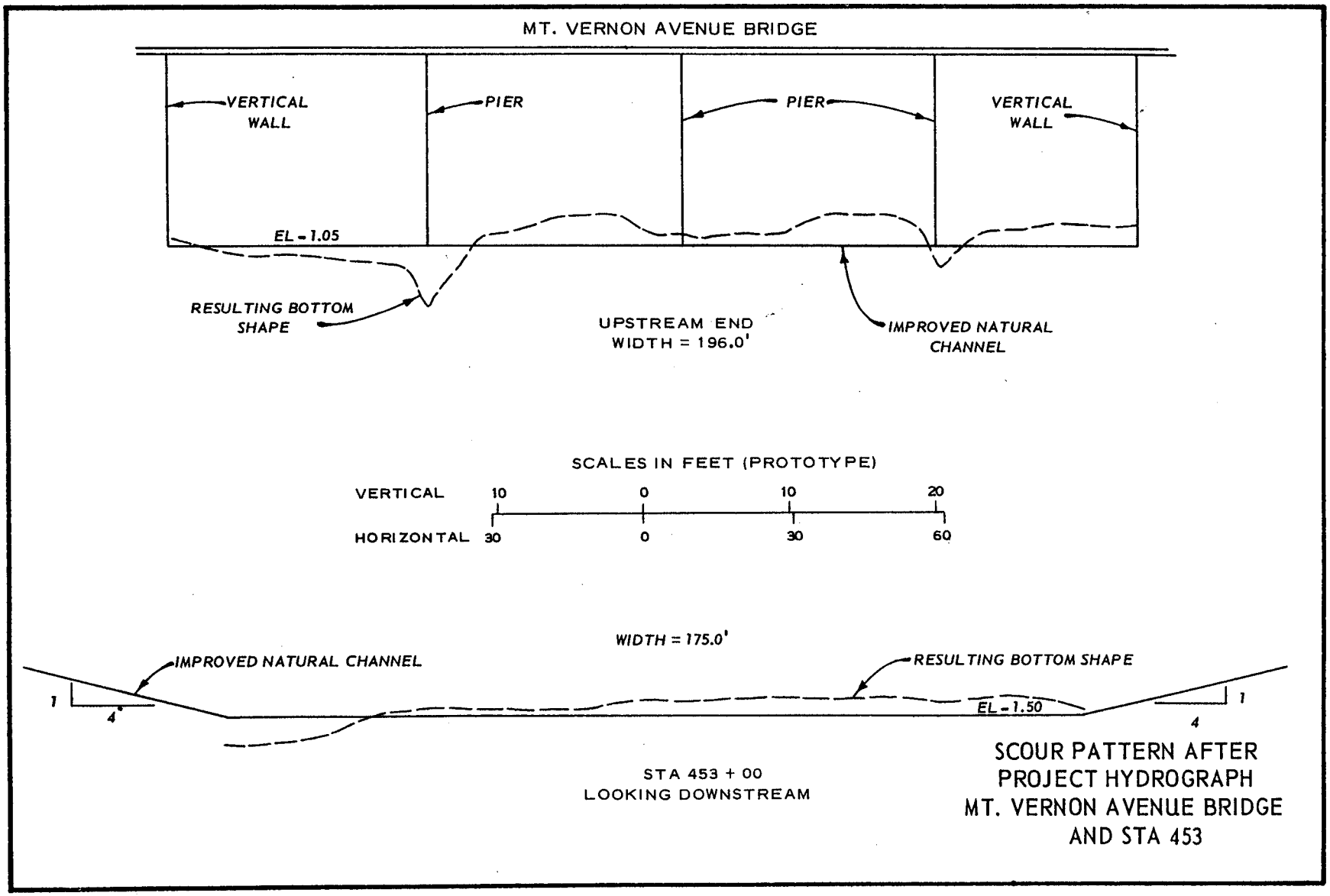
SCOUR PATTERN AFTER PROJECT HYDROGRAPH STA 407 AND STA 409



SCOUR PATTERN AFTER PROJECT HYDROGRAPH WEST GLEBE ROAD BRIDGE AND STA 425



SCOUR PATTERN AFTER PROJECT HYDROGRAPH STA 429 AND STA 437



SCOUR PATTERN AFTER PROJECT HYDROGRAPH MT. VERNON AVENUE BRIDGE AND STA 453

In accordance with ER 70-2-3, paragraph 6c(1)(b), dated 15 February 1973, a facsimile catalog card in Library of Congress format is reproduced below.

Oswalt, Noel R

Fourmile Run local flood-control project, Alexandria and Arlington County, Virginia; hydraulic model investigation, by Noel R. Oswalt, John F. George, and Glenn A. Pickering. Vicksburg, U. S. Army Engineer Waterways Experiment Station, 1975.

1 v. (various pagings) illus. 27 cm. (U. S. Waterways Experiment Station. Technical report H-75-19)

Prepared for U. S. Army Engineer District, Baltimore, Baltimore, Md.

1. Alexandria, Va. 2. Arlington County, Va. 3. Bank protection. 4. Channel improvement. 5. Flood control. 6. Fourmile Run. 7. Hydraulic models. 8. Open channel flow. I. George, John F., joint author. II. Pickering, Glenn A., joint author. III. U. S. Army Engineer District, Baltimore. (Series: U. S. Waterways Experiment Station, Vicksburg, Miss. Technical report H-75-19)

TA7.W34 no.H-75-19