TECHNICAL REPORT H-68-8

NAVIGATION CONDITIONS AT LOCK AND DAM NO. 3, ARKANSAS RIVER ARKANSAS AND OKLAHOMA

Hydraulic Model Investigation

by

J. J. Franco
C. D. McKellar

September 1968

Sponsored by

U. S. Army Engineer District
Little Rock

Conducted by

U. S. Army Engineer Waterways Experiment Station
CORPS OF ENGINEERS
Vicksburg, Mississippi

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ARMY-MRC VICKSBURG, MISS.

FOREWORD

The model investigation reported herein was authorized by the Office, Chief of Engineers, in a message dated 7 February 1961 to the Division Engineer, U. S. Army Engineer Division, Southwestern. The study was conducted for the U. S. Army Engineer District, Little Rock, by the U. S. Army Engineer Waterways Experiment Station during the period March 1961 to July 1965.

During the course of the model study, the Little Rock District was kept informed of the progress of the study through monthly reports and interim reports of the results of special tests. In addition, Messrs. J. P. Davis and R. P. Hobson of the Office, Chief of Engineers, GEN C. H. Dunn, Messrs. R. D. Field, E. B. Madden, C. A. Long, G. A. Makela, Leonard Hough, and A. J. Davis of the Southwestern Division, and Messrs. E. F. Rutt, J. C. Pyle, W. W. McMahon, W. A. Thomas, J. T. Clements, Jr., Tasso Schmidgall, D. R. Rippey, and C. W. Shelton, and Misses Irene Miller and Margaret Petersen of the Little Rock District visited the Waterways Experiment Station at intervals to observe model tests and discuss test results.

The investigation was conducted in the Hydraulics Division under the general supervision of Mr. E. P. Fortson, Jr., Division Chief, and Mr. G. B. Fenwick, Assistant Division Chief, and under the direct supervision of Mr. J. J. Franco, Chief of the Waterways Branch. The engineer in immediate charge of the model study was Mr. C. D. McKellar, Jr., assisted by Messrs. H. S. Austin, S. T. Mattingly, John A. Holliday, Allen E. Hullum, Lloyd Woods, E. E. Moorhead, R. T. Wooley, B. C. Rawls, and E. L. Coyner, Jr. This report was prepared by Messrs. Franco and McKellar.

Directors of the Waterways Experiment Station during the course of this investigation and preparation and publication of this report were COL Edmund H. Lang, CE, COL Alex G. Sutton, Jr., CE, COL John R. Oswalt, Jr., CE, and COL Levi A. Brown, CE. Technical Director was Mr. J. B. Tiffany.

CONTENTS

CONVERSION FACTORS, BRITISH TO METRIC UNITS OF MEASUREMENT

British units of measurement used in this report can be converted to metric units as follows:

Multiply	By	To Obtain
inches	2.54	centimeters
feet	0.3048	meters
miles	1.609344	kilometers
square miles	2.58999	square kilometers
cubic yards	0.764555	cubic meters
feet per second	0.3048	meters per second
cubic feet per second	0.0283168	cubic meters per second

SUMMARY

Lock and Dam No. 3 will consist of a 110- by 600-ft lock and a 1260-ft-long, gated, nonnavigable dam. A movable-bed model reproducing 12.8 miles of the Arkansas River, with a horizontal scale of 1:120 and a vertical scale of 1:80, was used to determine: (a) the suitability of the proposed site for the lock and dam structure; (b) adequacy of proposed regulating works in the reach upstream and downstream, including a proposed cutoff; (c) modifications required to provide adequate channel dimensions and safe navigation conditions with minimum maintenance; and (d) the effects of various cofferdam and diversion plans. Special studies were also conducted with the model converted to a 1:120 scale (undistorted). The results of the investigation indicated the following:

- a. Satisfactory navigation conditions can be developed with the lock and dam at the proposed site.
- b. The number of gate bays in the spillway can be reduced from 22 to 18 without any serious effect on water-surface elevations. Flow distribution would be generally better with a level gate sill than with the stepped sill included in the original design.
- c. Development of channel dimensions was affected to a considerable extent by the tendency for the channel to meander within the long, straight reach in the vicinity of the structures, and the effects of gate operation on the movement of sediment in the reach upstream of the dam.
- d. Regulating structures that provided adequate channel dimensions under typical flow conditions were developed, except in the lower lock approach channel.
- e. A satisfactory plan for the elimination of shoaling in the lower lock approach was not developed during this study. A wing dike at the end of the lower guard wall could be used to reduce the frequency of maintenance dredging with little effect on the amount of shoaling.
- f. The capacity of the ports in the upper guard wall would have to be increased to eliminate the hazardous crosscurrents in the upper lock approach and to reduce scour near the end of the upper guard wall.

g. A fill or dike would be required along the left overbank to prevent flow from the overbank moving toward the spillway from seriously affecting downbound tows approaching the lock during high flows.

h. The maximum scour with cofferdam during diversion will occur at the point of contraction (diversion end of cofferdam) with a relatively small amount of scour in the remainder of the diversion channel. Water-surface elevations along the upstream side, location of maximum scour, and scour pattern would be affected to some extent by the alignment of the upstream arm of the cofferdam. Material scoured from the cofferdam side of the diversion channel would be deposited downstream of the cofferdam. The alignment of the downstream arm of the cofferdam could have some effect on scour along the downstream face of portions of the cofferdam.

i. The point of maximum scour can be moved away from the cofferdam with a spur dike placed near the diversion end of the cofferdam and extending upstream of the upper arm of the cofferdam.

NAVIGATION CONDITIONS AT LOCK AND DAM NO. 3

ARKANSAS RIVER, ARKANSAS AND OKLAHOMA

Hydraulic Model Investigation

PART I: INTRODUCTION

Present Development Plan for the Arkansas River

1. The Arkansas River is considered a navigable stream from its mouth to the mouth of the Verdigris River (fig. 1). In this section, the slope of the stream averages 0.9 ft per mile* above Little Rock, and 0.7 ft per mile between Little Rock and the Mississippi River. Water-surface elevations and slopes in the lower river are affected by backwater from the Mississippi; these effects at times extend as far upstream as the vicinity of Pine Bluff, mile 111. During periods of low water, the controlling depth of the Arkansas River from its mouth to Little Rock is about 2 ft, and from Little Rock to the mouth of the Verdigris River, about 1 ft.

2. The Arkansas River multipurpose project as presently authorized provides for improvement of the Arkansas River and tributaries in Arkansas and Oklahoma by construction of coordinated developments to serve navigation, produce hydroelectric power, afford additional flood control, and provide related benefits such as public facilities for recreation and conservation of fish and wildlife.

3. The navigation feature of the project provides for a 9-ft-deep channel from Catoosa, Okla., on the Verdigris River, 52 miles downstream to the Arkansas River at mile 458, thence down the Arkansas River to Arkansas Post, about 46 miles above its mouth. From this point the Arkansas Post Canal will connect the Arkansas River with the White River; the navigation channel will then continue down the White River for about 10 miles to its junction with the Mississippi River. The 9-ft-deep

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* A table of factors for converting British units of measurement to metric units is presented on page vii.

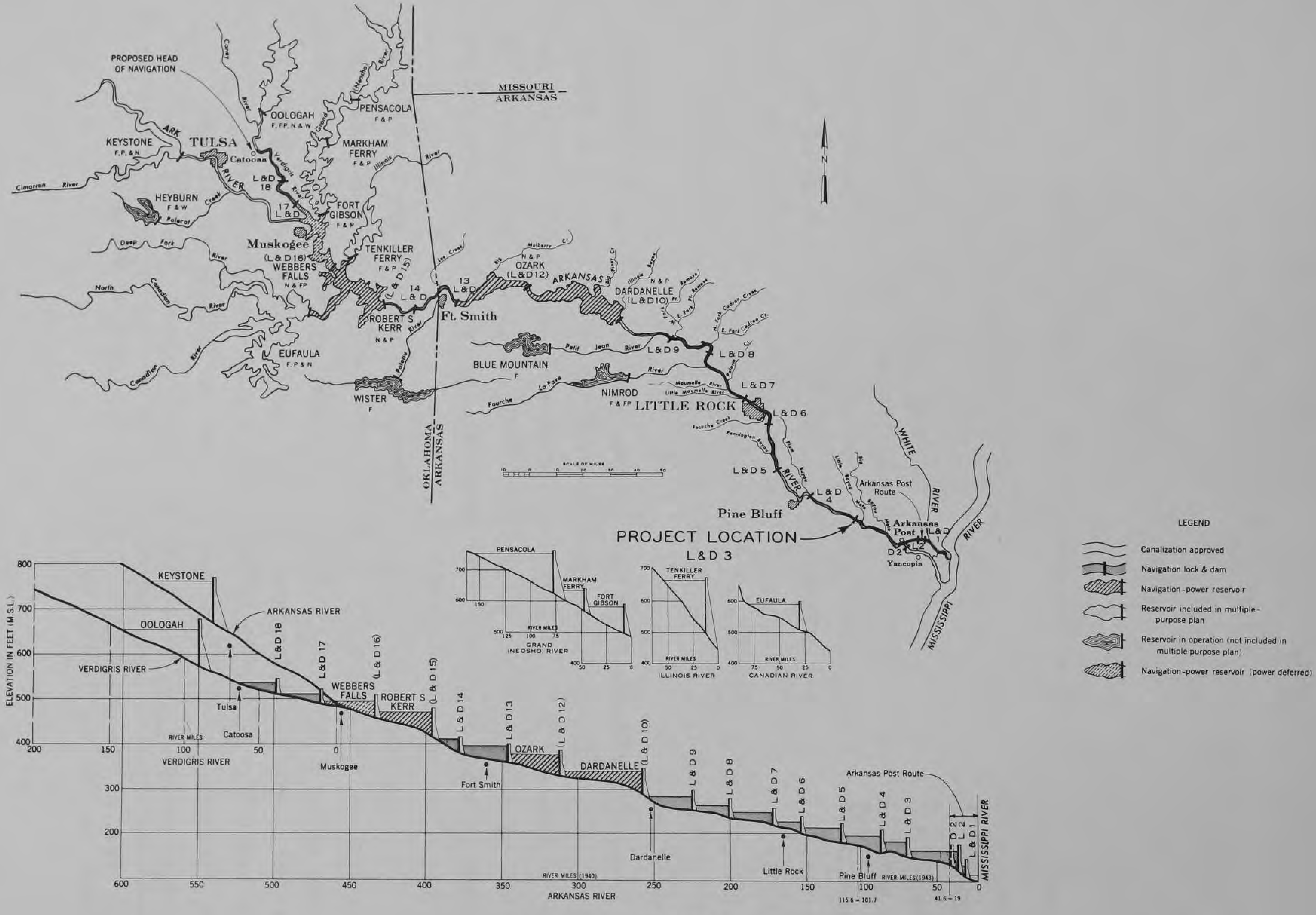

Fig. 1. Vicinity map

channel will be provided by a system of locks and dams, some of which will be used for both navigation and hydroelectric power production. Lock chambers will be 110 by 600 ft on the Arkansas River and in the canal connecting with the White River, and 83 by 600 ft on the Verdigris River. A minimum channel width of 150 ft is proposed for the Verdigris River section, 250 ft for the Arkansas and White River sections, and 300 ft in the Arkansas Post Canal. Bank stabilization and channel rectification works, such as training dikes, cutoffs, and revetments, are included in the multipurpose plan and are part of the proposed overall development of the Arkansas River.

Description of Lock and Dam No. 3 Structures and Channel Improvements

4. Lock and Dam No. 3 is one unit of the navigation portion of the multipurpose plan for the development of the Arkansas River. The structures will consist of a 110- by 600-ft lock and a nonnavigable dam 1260 ft long (fig. 2). The dam, as proposed, will consist of eighteen 60-ft-wide spillway bays and eighteen 10-ft-wide piers. The spillway will be controlled by eighteen 60- by 25-ft conventional tainter gates with the sills set at el 158.0.* The site of Lock and Dam No. 3 will be at river mile 72.3 (1943 survey) in Lincoln and Jefferson Counties about 20 miles southeast of Pine Bluff, Ark., and approximately 11 miles downstream from the Rob Roy Bridge. The proposed pool will extend approximately 16 miles upstream, and in general will be contained within the existing banks. The normal upper pool will be at el 182.0, and the normal lower pool, established by Lock and Dam No. 2, will be at el 162.0. The maximum lift of 20 ft will occur with both pools at normal elevation. The maximum navigation stage is based on a 10-year-frequency flood of 350,000 cfs and will result in upper and lower pool elevations of 191.6 and 190.5, respectively. Therefore, the Lock and Dam No. 3 lifts will vary between about 1 and 20 ft. The tops of the approach walls and lock walls will be set at el 194.0. This will provide for the higher of either a 10-ft freeboard

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

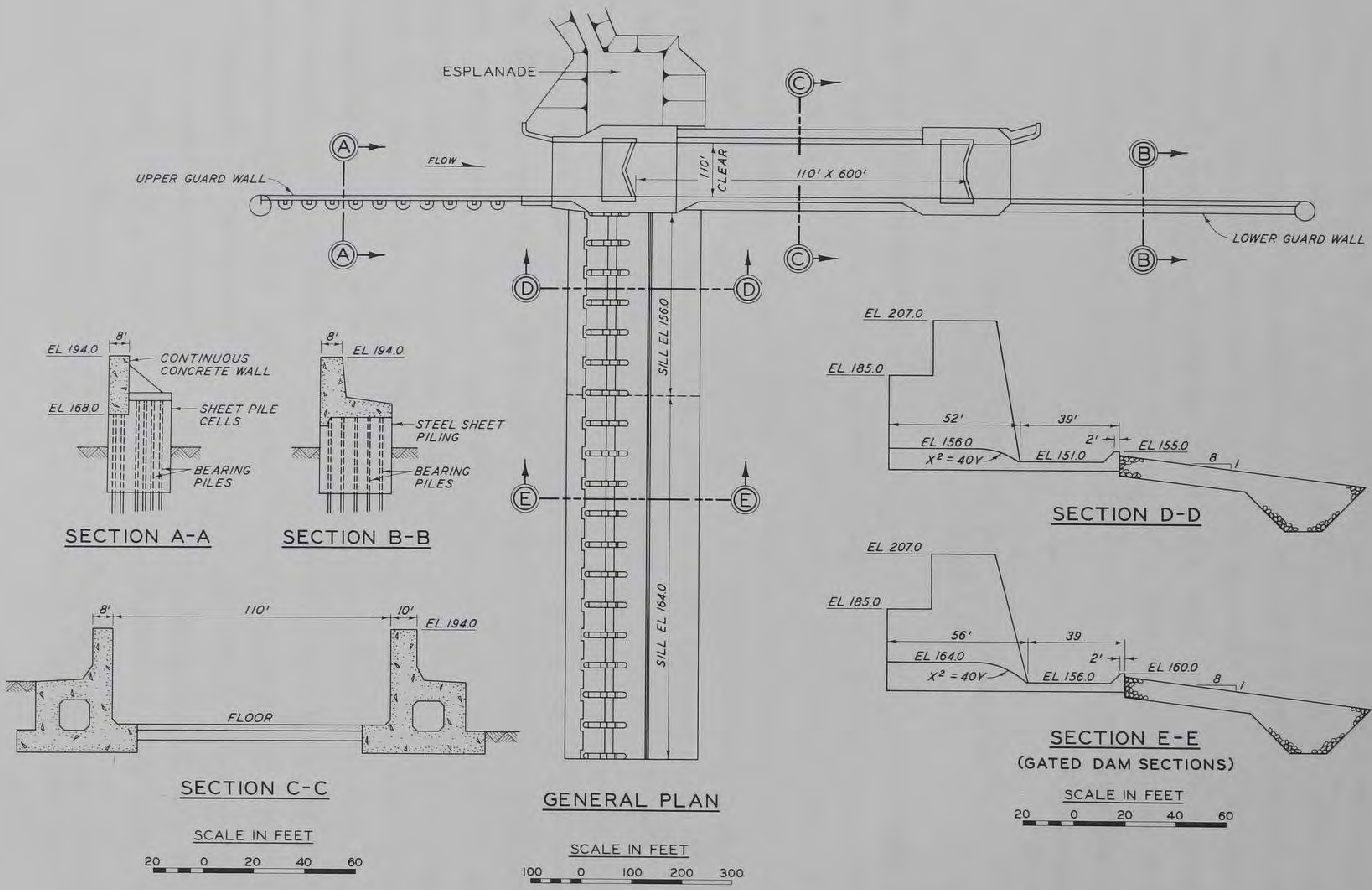


Fig. 2. General plan of lock and dam

above the normal upper pool elevation or a 2-ft freeboard above the 10-year-frequency flood elevation. Because the upper pool elevation for a 10-year-frequency flood is 191.6, the 2-ft freeboard above 10-year-frequency flood will control, and establish the top of walls at el 194.0. Both upstream and downstream guard walls will be 600 ft long.

5. Overflow embankments will be provided between the dam and the high ground or levees to maintain the normal pool and to provide a road to the lock. The 4300-ft-long left embankment will extend from the levee to the lock esplanade at el 192.0, conforming to the upstream level of the navigation or modified 10-year-frequency flood. It will have a crown width of 31 ft to provide for a concrete roadway 22 ft wide. The embankment on the right bank will be 960 ft long with a maximum height of about 25 ft and a crown elevation of 185.0, providing 3 ft of freeboard for wave action above normal upper pool and allowance for tailwater buildup before the embankment is overtopped. The crown width will be 20 ft, providing a broad-crested overflow weir. The upstream and downstream slopes will be protected with riprap designed to provide resistance to scour during overflow periods. A 150-ft-square esplanade with a top elevation of 194.0 will be constructed adjacent to the land side of the lock near the upper gate pintle.

6. In addition to the lock, dam, and pertinent structures, the improvement of the reach would include such regulating structures required to provide a channel of adequate depth and alignment and the development of the cutoff constructed about 4 miles downstream of the dam opposite the mouth of Little Bayou Meto.

7. The overall plan for canalization of the Arkansas River involves many channel development and maintenance problems. Analytical solutions to these problems, particularly those involving sediment, are complex and uncertain. Therefore, a hydraulic movable-bed model study was considered necessary to investigate the effects of various types of control works on channel stability below the lock and dam. The purposes of the model study were to determine the suitability of the selected site, the adequacy of the channel regulating works, effect of gate-sill elevation and gate operation on channel development, navigation conditions

in the lock approaches, backwater effect of the structures, and scour patterns which could be expected with various diversion plans (cofferdams), and to assist in the development of modifications that would tend to improve navigation conditions and minimize channel maintenance.

PART II: THE MODEL AND ITS VERIFICATION

Description

8. The model of Lock and Dam No. 3 (fig. 3) was a scale reproduction of a reach of the Arkansas River extending from mile 76.0 to mile 63.2, including sufficient overbank area to permit overbank flow across bends on both banks, but not to the limits of the floodway at all points. The model was built to linear scale ratios of 1:120 horizontally and 1:80 vertically, producing geometrically a slope scale ratio of 1.5 to 1, model to prototype. A small supplementary slope of 0.00007 (equivalent to about 0.37 ft per mile prototype), which was needed to provide satisfactory bed movement, was also incorporated in the model. The model was of the movable-bed type with fixed banks and overbank areas molded in sand-cement mortar. Folded strips of mesh wire were used to simulate the roughness effect of trees and underbrush on the overbank areas, as shown in fig. 4. The bed material was coal which had a median grain diameter of about 4 mm and a specific gravity of 1.30. The permeable pile dikes in the model were made of two rows of metal rods. The lock and dam were fabricated of sheet metal to prevent change in elevation which might have been caused by expansion or warping after the structures were set. The upstream and downstream dimensions of the dam were constructed to the vertical scale to produce the required jump within the basin. Each bay of the model dam was constructed to reproduce two prototype bays with the piers modified accordingly.

9. The fixed portions of the model were molded from the edge of the movable bed to the top bank in accordance with data shown in the Richland Bend to Round Lake survey dated 11-16 May 1960 and the Round Lake-Cummins Bend survey dated 17-18 February 1959. The remainder of the fixed bed was molded to 1940 data shown on chart 5 dated 1942. The initial contours of the movable bed of the model were in accordance with the configurations shown in plate 1. This configuration was based on the February 1959 and May 1960 surveys referred to above, except for a short reach of about 0.7 mile at the upstream end of the model which was based on the April 1961 survey.

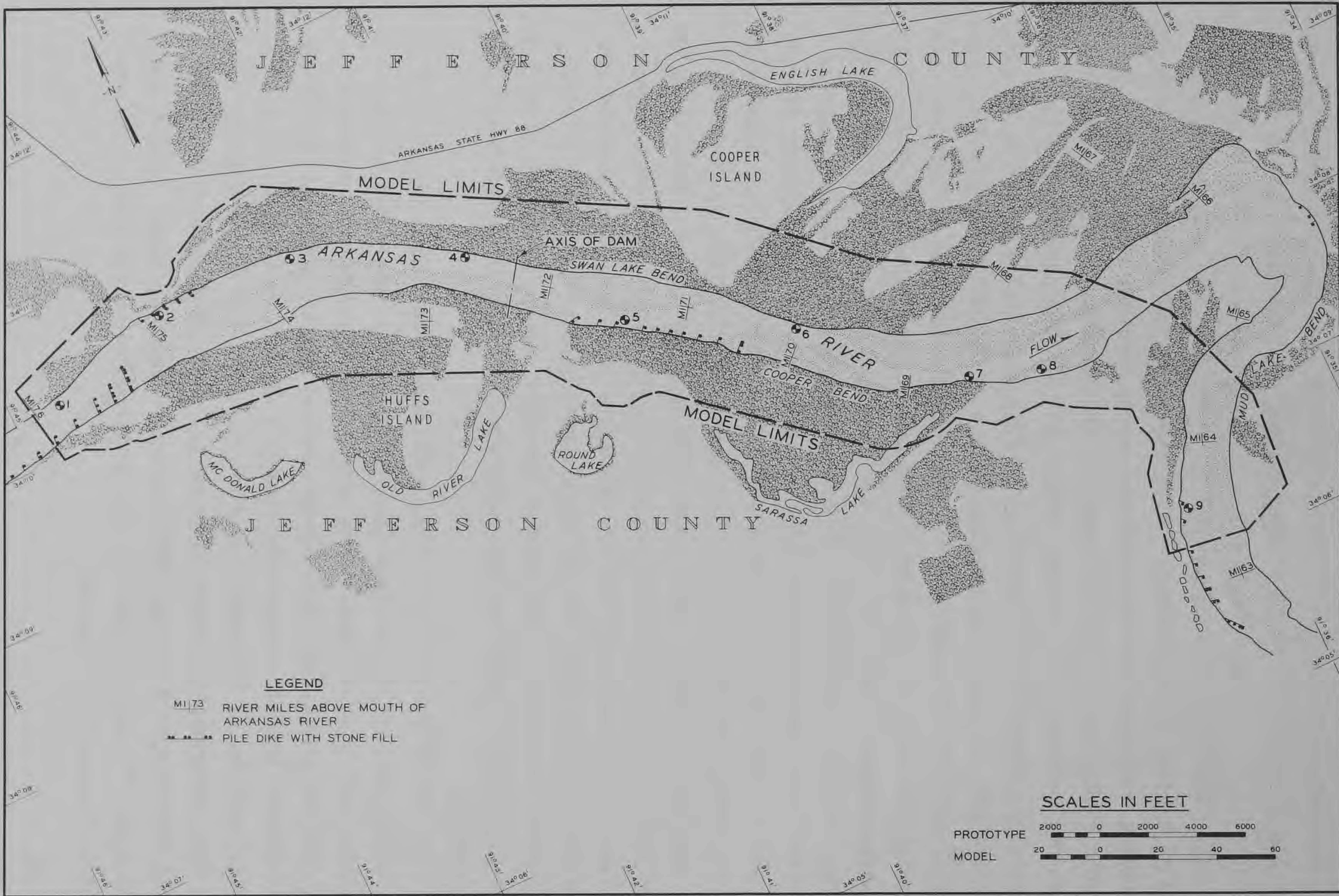


Fig. 3. Model layout and locations of gages

Appurtenances

10. Water was supplied to the model by a 10-cfs axial flow pump operating in a circulating system and was measured at the upper end of the model by means of two venturi meters of different sizes required to handle the range of discharge. Water-surface elevations throughout the model reach were measured with nine piezometer and two point gages. Tailwater elevations at the lower end of the model were controlled with an adjustable tailgate. Soundings in the model were obtained by use of a sounding rail and special sounding rod which permitted the reading of elevations in prototype feet.

Fig. 4. The model

Bed material to be introduced into the model was measured in a graduated container and introduced by hand at the upper end of the model. A sediment trap was provided at the lower end of the model where extruded material could accumulate and be measured to determine the amount discharged for any period. Sheet metal templates were used for molding the model bed prior to initiation of certain tests. A row of carefully graded, threaded iron rods was provided along each concrete bank of the channel to support templates at correct elevations and the rails from which the model bed was surveyed, and to control the grade of dredged cuts and structures during installation. Current directions were determined by plotting the paths of wooden floats with respect to ranges established for that purpose; floats were submerged to a depth of 8 ft, equivalent to the draft of a loaded barge. Velocities for current directions were measured by timing the travel of floats over known distances. A midget current meter was used to measure velocities at special locations.

11. A radio-controlled model tow and towboat, equipped with screw-type propellers powered by two small electric motors operating from batteries located in the tow, were used to study and demonstrate the effects of currents on navigation (fig. 5). The towboat could be made to run in forward or reverse, at various speeds comparable to that of towboats which will travel the Arkansas River, and with variable rudder settings.

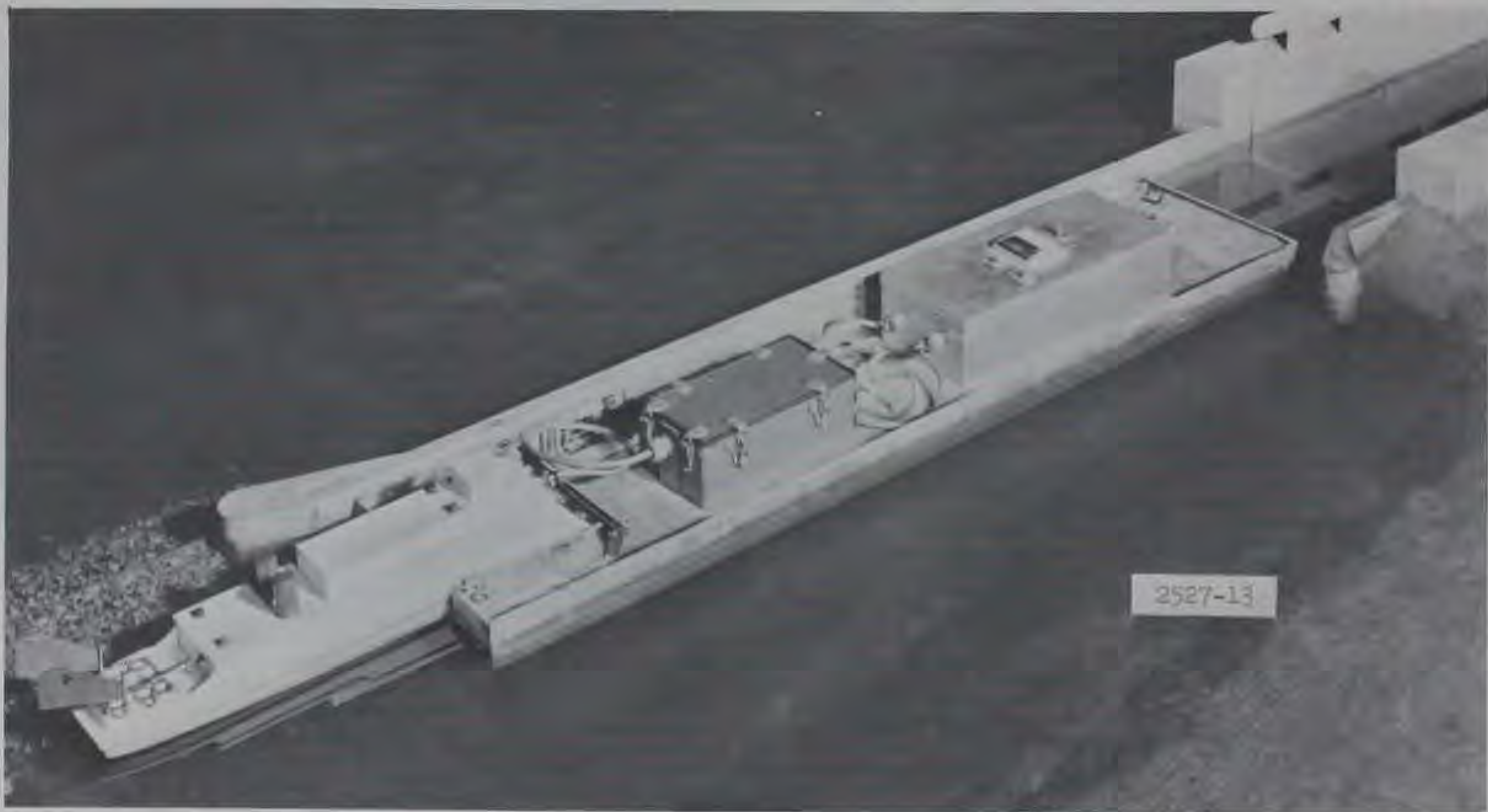


Fig. 5. Model towboat and tow

Model Adjustment

Conventional procedures

12. The reliability, or degree of similitude to the prototype, of the conventional movable-bed model used to develop plans for river regulative works for channel improvement and maintenance is established through the empirical process known as verification of the model. Verification is an intricate process of adjusting the various hydraulic forces, model-operating technique, and other factors until the model demonstrates the

ability to reproduce with acceptable accuracy the changes in bed configuration that are known to have occurred in the prototype between certain dates. In this manner, the future model-operating technique, the time, discharge, and bed-load movement model-to-prototype scale relations, and the degree of accuracy to be expected of the model are worked out.

13. A conventional movable-bed model that would develop results applicable to a specific reach of the river was not practicable for this study. The available documented knowledge of the Arkansas River was insufficient to support a movable-bed model verification of the conventional type, since no adequate hydrographic surveys timed to depict the history of bed configuration changes in various reaches of the river from stage to stage or from year to year were available.

Procedure used

14. Although a conventional verification of the model was infeasible, it was essential that the model be adjusted to reproduce channel configurations generally representative of those to be expected in the Arkansas River under similar conditions. Accordingly, a series of adjustment tests was conducted during which progressive changes were made in the discharge scale, time scale, and rate of introducing bed material until the movement of material during each stage and the configuration of the bed at the conclusions of the final tests were considered satisfactory.

15. The adjustment tests were started with the movable bed molded to the channel configurations shown in plate 1, obtained from the following surveys:

Date	Mile
April 1961	76.10 to 75.37
May 1960	75.37 to 71.75
April 1961	71.75 to 70.6
February 1959	70.6 to 66.8

The model was operated by reproducing the prototype stage hydrograph for the one-year period immediately preceding the April 1961 survey (plate 2). The water-surface elevations were based on the computed rating curve for mile 68.0 (plate 3) and stages were referred to the Pine Bluff gage. Except for the control of tailwater elevations, no other control was maintained over water-surface elevations during this test.

16. The adjustment test was repeated with the various factors (time scale, discharge scale, and rate of introducing bed material) modified until the model reproduced with a reasonable degree of accuracy the prototype configurations shown by the 1961 prototype surveys.

Results

17. Channel configurations. The results of the adjustment test are shown in plates 4 and 5. These results indicate that at ends of the first and second reproductions of the hydrograph, the configurations of the model bed were generally similar to that indicated by the prototype survey. The adjustment resulted in the development of operating techniques and scales sufficiently accurate to permit satisfactory reproduction of the general characteristics of the Arkansas River.

18. Scale relations. The horizontal, vertical, and slope scales of the model were selected on the basis of preliminary computations made from the known characteristics of the bed material selected. These scales were used in constructing the model as mentioned in paragraph 8. The other scale ratios were developed in the adjustment process and were as follows:

a. Time scale. The time scale was based on (1) the rate of development of changes in configuration during the various flows, and (2) the requirement that the total movement of bed material into and through the model be sufficient to ensure that the channel configuration at the end of any model test would be that produced by effects of the model forces and features being tested, rather than being unduly influenced by the configuration of the bed at the beginning of the test. The time-scale ratio was 1:240, or 6 min model time was equivalent to one prototype day.

b. Rate of bed movement. The initial rate of introducing bed material was based on prototype bed-load curves. This rate was progressively modified by general judgment based on (1) the rate of movement of material near the model entrance, (2) the amount of material extruded from the model, and (3) detailed observations of the movement of the material throughout the model channel. The rate at which bed material was introduced varied with total river discharge.

c. Discharge scale. The discharge scale used initially was based on the linear and slope scale ratios. Final adjustment of the model resulted in a variable discharge scale. The discharge ratios used were determined by general judgment based on (1) detailed observations of the relative movement of the model bed material with various flows reproduced, and (2) the requirement that movement occur in the same general areas as could be expected in the prototype, and that the rate of shoaling and scouring during the rising and falling stages of the hydrograph should reproduce the channel configuration shown by the prototype survey.

Test Procedure

19. Tests were concerned with determining the suitability of the selected site, adequacy of the regulating works, effect of gate-sill elevation and gate operation on channel development, navigation conditions in the lock approaches, and backwater effect of the structures, and assisting in the development of modifications that would tend to improve navigation conditions and minimize channel maintenance.

20. Testing of each plan was started with the bed as obtained in the preceding test except as modified by changes that could reasonably be expected with the installation of regulating structures, particularly where channel realignment was involved. Each test consisted of a series of "runs," with each run consisting of the reproduction of an annual hydrograph considered typical for the Arkansas River (plate 2) using the scale relations developed except as modified based on the effects of the lock and dam structures. Tests were conducted either to obtain an indication of the effectiveness of a proposed plan or to determine the ultimate conditions that could be expected from the plan. Uncontrolled river flows were reproduced by introducing the proper discharge without the dam or with the dam gates fully open and manipulating the tailgate to obtain the proper tailwater elevation below the dam. The controlled river flows were reproduced by introducing the proper discharges, setting the tailwater elevation for that discharge, and manipulating the gate openings until the required upper pool elevation was obtained. Gate openings, except where otherwise stated, were based on the requirement that the distribution of flow through the gates be proportional to five-thirds power of the actual depth or of an assumed depth of the tailwater over each gate sill. This condition was established by the Little Rock District, because of the difference in the elevation of the sill and stilling basin, to provide sufficient tailwater for the dissipation of energy and to provide adequate distribution of flow across the channel. Initially, the gate openings were computed based on a gate calibration

curve developed on a large-scale section model of a similar type structure. Special tests were conducted with the elevation of the channel bed changed to conform to the horizontal scale and with the discharge reproduced in accordance with the Froudian relation to permit accurate reproduction of currents affecting navigation.

Base Test

Description

21. A base test was conducted to establish the ultimate channel configuration that would develop with the channel rectified and stabilized according to the proposed bank stabilization plan for the reach of river modeled and the repeated reproduction of the typical hydrograph selected for model tests. The results of the base test were also used to determine the effects of subsequent plans proposed for improvement of the reach. The base test was conducted with the channel rectified and stabilized by dikes and revetments based upon plans prepared for the reach by the Little Rock District and shown in plate 6. After installation of the channel regulating structures and opening of the Little Bayou Meto Cutoff, the model channel was molded to typical cross sections from just above the proposed damsite to the lower end of the model. In the upstream reach the channel was the same as obtained at the end of the adjustment test, except for the addition of some bed material on the bar side of the channel based on the amount of material removed from the left bank. The areas behind the revetments and between dikes were filled, based on the assumption that the fills would occur by natural accretion or by the placement of dredged spoil.

22. The model was first operated for a period of time with representative stages reproduced to expedite the development of conditions with the modifications mentioned above. After this initial operation period, the model was operated by reproducing the prototype hydrograph for the period 10 February 1960 to 9 February 1961 for each of three runs.

Results

23. Results of the base tests, shown in photographs 1-3 and

plates 7-9, indicate that the alignment of the channel developed during the base test was generally good, except near the proposed damsite. Near the damsite, a bar developed along the left bank with the channel tending to cross toward the right bank (photograph 1). This tendency was indicated by the 1961 prototype survey and was not affected appreciably by the regulating works included at the start of the test (plate 6). Project depths were not obtained in the reach immediately below the proposed damsite and in the crossing toward the right bank at about mile 70 (photograph 2). The channel upstream of the cutoff did not form a short crossing from the right to the left bank, and the entrance into the cutoff channel was rather abrupt, producing a channel of poor alignment and a deep scour along the revetment through the cutoff (photograph 3). The model had not completely stabilized as indicated by developments at the ends of each of the three runs and by the amount of sediment extruded which was less than the amount introduced.

Tests for Development of Regulating Structures, Plans A through A-15

Plan A

24. Description. The results of the base test indicated that the channel within the long, straight reach between miles 71 and 73 would tend to meander and a satisfactory channel could not be developed without additional structures. Since construction of the lock and dam could have a significant effect on developments, plan A included the installation of the structures without other modifications. Except for the modifications required in connection with the lock and dam, the bed of the channel at the start of the test was the same as obtained at the end of the base test. The structures and modifications included in this plan were:

a. A 1508-ft-long dam with its axis at mile 72.4. The dam simulated in the model consisted of six 60-ft-wide by 17-ft-high gates with sill elevation of 156.0 and six 8-ft piers located nearest the lock, then a 12-ft pier, and sixteen 60-ft-wide gates with sill elevation of 164.0 and sixteen 8-ft piers. The stilling basins at the low- and high-sill gates were 5.0 and 8.0 ft below the elevation of the gate sill, respectively. The model dam was constructed so that each model bay simulated two prototype bays.

b. A lock with clear chamber dimensions of 110 by 600 ft, and 600-ft-long upper and lower guard walls. Top of lock walls was set at el 193.0. The upper guard wall contained eleven 9-ft-high by 23-ft-wide ports with the top set at el 169.0. The bottom of the channel bed between the upper guard wall and left bank was excavated and covered with a rock blanket, placing the bottom of the ports at el 160.0. At the end of run 1, the channel bed between the guard wall and left bank was lowered to el 149.0, increasing the height of ports to 20 ft.

c. A lower lock approach channel dredged to a bottom elevation of 150.0 and bottom width of 250 ft.

d. A 150-ft-square esplanade with crown elevation of 195.0, an access dike on the left overbank with a top elevation of 193.0 connecting to the levee, and a right bank dike with a crown elevation of 185.0 to retain the upper pool.

e. A riprap apron extending 35 ft upstream of the dam and 98 ft downstream of the stilling basin, except at the low-sill gates where it extended 195 ft downstream of the dam. Slope of the downstream apron was 1 on 8.

f. A 480-ft-long wing wall with top elevation of 164.0 installed at the downstream end of the lower guard wall (plate 11) prior to beginning of run 5. The wall was set at an angle of 11 degrees to the right of a line parallel to the land side of the river-side lock wall.

25. Results. Operation of the model at the start of the test indicated considerable scour below the riprap apron downstream of gates 4 to 8 and the riprap was extended downstream before the test was continued. It was also noted during run 1 that the capacity of the ports in the upper guard wall was inadequate and strong crosscurrents developed near the end of the guard wall. The crosscurrents produced considerable turbulence and scouring near the end and just downstream of the end of the wall. This condition was corrected by lowering the lock approach channel and the bottom of the ports to el 149.0 as mentioned in paragraph 24b.

26. The results of tests with the above modifications are shown in photograph 4 and plates 10 and 11. These results indicate that the channel above the dam would tend to aggrade, and that in the lower lock approach, shoaling of the approach channel would be heavy and continuous. The large

cell at the end of the lower guard wall appeared to increase the tendency for shoaling in the lock approaches. Installation of the wing dike at the end of the guard wall reduced shoaling in the approach channel and kept the deposition out of the left side of the channel. A narrow channel of project depth was maintained along the left bank during run 5 (plate 11). The configuration of the channel bed downstream of the dam was rather irregular (photograph 4).

27. A channel of project depth was maintained downstream of the lower lock approach except in the crossing at about mile 70. Depths over the crossing were slightly better than those obtained in the base test. The alignment of the channel approaching the cutoff channel was not as good as in the base test, and depths along the left bank of the cutoff channel were increased.

Plan A-1

28. Description. Plan A-1 was the same as plan A except for the following (plate 12):

a. Installation of two dikes downstream of the dam at about mile 72 and extension of the right bank dike system downstream of the dam between miles 72.0 and 70.6 to contract the channel at mile 72.0 and 71.0 to 1500 and 1200 ft, respectively.

b. Realignment of the wing dike at the end of the lower guard wall so as to place its upstream end on the river side of the cell at the end of the wall. Location of the downstream end was not changed.

c. Shifting of the right bank revetment riverward beginning at mile 69.4 and extending the dikes to contract the width of the channel to 1000 ft at mile 68.5, 1100 ft at mile 68.2, and 1200 ft at mile 68.0 (plate 12). These modifications were made to reduce shoaling in the lower lock approach and to improve the alignment of the channel over the crossing approaching the cutoff. Test of this plan was started with the channel bed as developed in the test of plan A except that a 250-ft-wide channel was dredged to el 150.0 in the lower lock approach.

29. Results. The results of tests of plan A-1 indicate that the tendency for the channel upstream of the dam to shoal would continue (plate 13). Shoal areas appeared in the approach channel with some tendency for the channel to meander. The amount of shoaling in the lower

approach was increased above that obtained with plan A. This could have been caused by the lowering of the channel bed downstream of the dam, which increased the amount of material moving into the approach channel. Some scouring occurred in the channel over the crossing at mile 70 and project depths were obtained. However, observations indicated that the channel over the crossing would be unstable because of the small concentration of low flows. The alignment of the channel over the crossing approaching the cutoff and depths in the channel over the crossing were improved considerably. With the improvement of the approach channel, depths along the left bank in the cutoff were reduced.

Plan A-2

30. Description. The features of plan A-2 were the same as plan A-1 except for the following (plate 14):

a. The left bank revetment downstream of the dam at about mile 70 was extended 1150 ft downstream, contracting the channel at the end of the revetment to 900 ft.

b. Two dikes were installed on the left side of the channel near the end of the revetment mentioned in a above: one 300 ft upstream from the end of the revetment and the other at mile 69.7, contracting the channel to 1000 ft. These revisions were designed to eliminate unstable conditions in the channel over the crossing at mile 70. This test was started with the channel bed as obtained at the end of the test of plan A-1 except for the installation of the structures and dredging in the lower lock approach.

31. Before the beginning of run 3 of this test, the following revisions were made in the right bank dike system upstream of the dam in an effort to improve channel depths in the upper approach to the lock:

a. The dike at mile 75.2 was extended to provide a 1000-ft opening between the end of the dike and the opposite bank. The dike was made impermeable and raised to el 182.0.

b. The dikes at miles 74.8 and 74.4 were raised to el 182.0 and made impermeable.

c. Two impermeable dikes with top elevation of 182.0 were installed at miles 73.7 and 73.4.

32. Results. The results of tests of plan A-2 are shown in plates 15 and 16. These results indicate that shoaling upstream of the dam had continued and by the end of run 2 the channel had shoaled to less than

project depth at miles 73.5 and 75.0 (plate 15). This development indicated the need for the additional structures which were installed before the start of run 3. With modification of the structures upstream of the dam, a channel of project dimensions or better developed in the reach upstream of the dam by the end of run 4 (plate 16). Shoaling in the lower lock approach continued to increase somewhat during testing of this plan. A part of this increase could have been caused by the deepening of the channel upstream which increased the flow of sediment through the dam.

33. The channel over the crossing downstream of the dam (mile 70) increased in depth and width, and satisfactory channel conditions were maintained downstream.

Plan A-3

34. Description. Plan A-3 was the same as plan A-2 except for the following modifications in the right bank dike systems immediately upstream and downstream of the dam (plate 17):

a. Lengths of the dikes just downstream of the dam were increased to reduce the channel width to 1150 downstream of the end of the lower guard wall and to 1000 ft at mile 71 in an effort to reduce shoaling in the lower lock approach (plate 17).

b. An impermeable dike was installed along the right bank upstream of the dam at mile 73.0 before the end of the high-water period of run 1.

c. A training or vane-type dike was placed downstream of the end of and parallel to the wing dike at the end of the lower guard wall before start of run 2 (plate 19). The opening between the lower end of the wing dike and upper end of the training dike was closed with riprap toward the end of run 4. Prior to the start of run 5, the training and closure dikes were removed. Tops of the dikes added in the upper pool were set at el 182.0 and those downstream of the dam were set 14 ft above the construction reference plane (about el 176.0).

d. Three additional impermeable dikes were installed along the right bank upstream of the dam at miles 73.1, 72.8, and 72.6 near the end of the high-water period of run 3.

35. Results. During run 1, shoaling in the upper approach continued, and controlling depths just upstream of the end of the upper guard wall

were only 10 ft below normal upper pool (plate 18). With the installation of the additional dike upstream of the dam near the end of run 1, depths in the upper lock approach channel increased slightly by the end of run 2 (plate 19). With the addition of the dikes mentioned in paragraph 34d, a controlling depth of 16 ft below normal upper pool was obtained by the end of run 4 (plate 20). There was little additional change in depths in the upper approach during runs 5 and 6 (plate 21). In the lower approach shoaling continued, although at the end of run 1 the amount of material dredged was considerably less than at the end of run 4 of plan A-2 and a narrow channel of project depth was maintained along the revetment (photograph 5). The amount of shoaling had increased by the end of run 2, indicating that the second wing dike was not effective in improving channel dimensions (photograph 6). Because of the changes made during the test, the amounts of material dredged in the lower approach channel were erratic, varying from 64,000 to 188,000 cu yd during runs 1 to 5 and increased to 320,000 cu yd during run 6 (table 1). The shoaling in the lower lock approach was probably affected to some extent by developments produced by dikes upstream of the dam. Good channel alignment and depths were maintained downstream of the lower lock approach. Depths in the crossing at mile 70 were limited to 10 ft below normal lower pool.

Plan A-4

36. Description. Plan A-4 was the same as plan A-3 except that the length of dam and number of dam gates were reduced and the first three dikes downstream of the dam were modified. The number of gates in the dam were reduced from 22 to 18 by eliminating the four gates along the right bank and extending the right overflow embankment to the right abutment. Top of the embankment was at el 183.5. The first dike downstream of the dam was moved about 425 ft upstream and shortened so that its riverward end was in line with the end of the dam, and the second and third dikes downstream of the dam were shortened 200 and 210 ft, respectively (plate 22).

37. Results. The effects of the change in the length of the dam and modification of the dike system along the right bank downstream are indicated by the results shown in plate 23 and table 1. The bed of the

channel along the right side of the dam was lowered and a deeper channel developed along the ends of the right bank dikes. Because of the increase in depths along the right side, the shoal area downstream of the end of the lower guard wall was increased compared with that obtained at the end of plan A-3. The amount of maintenance dredging, however, was considerably less. Depths in the channel along the lower reach of the left bank revetment downstream of the dam were increased. The increase in depth extended over the crossing at mile 70 where controlling depth was about 15 ft below normal pool. The reduction in the number of gate bays in the dam increased the swellhead at the dam about 0.3 ft with the 255,000-cfs flow with little change in water-surface elevation upstream of the dam.

Plan A-5

38. Description. Plan A-5 involved modification of the dikes along the right bank upstream of the dam and the realignment of the channel downstream of the dam as follows (plate 24):

a. The first dike along the right bank downstream of the dam was realigned by reducing the angle toward downstream.

b. Dikes between miles 71.0 and 70.6 were modified to provide a 1000-ft channel width between the ends of dikes and the left bank revetment.

c. The left bank revetment was shortened approximately 2500 ft.

d. The alignment of the right bank revetment below the crossing at mile 70.5 and of the left bank revetment upstream and through the cutoff was modified as shown in plate 24.

e. The dikes along the right bank upstream of the dam to mile 74.0 were shortened to provide a 1200-ft width from the riverward end of the dikes to the construction reference plane along the left bank.

39. Before the start of run 3, the left bank revetment in the vicinity of mile 70.6 was extended 200 ft downstream and the alignment was modified by continuing the 10,000-ft radius to the end of the revetment. Shortly after the start of run 3, the right bank revetment in the vicinity of mile 68.9 was extended on a tangent 300 ft downstream; however, about midway of this run, the alignment of the extension was modified by continuing the 10,000-ft-radius curve so that the lower end of the revetment was in line with the riverward end of the dike downstream. Before the start of

run 4, the left bank revetment in the vicinity of mile 70.5 was extended 700 ft downstream, on a 10,000-ft-radius curve, to about mile 70.3 to provide a 900-ft-wide channel at the end of the revetment as shown in plate 26.

40. Results. The results of tests of plan A-5 are shown in plates 25 and 26 and photographs 7 and 8. These results indicate that shoaling along the ends of the dikes just upstream of the dam continued to increase with a corresponding increase in the depth of the channel approaching the lock (photograph 7). Downstream of the dam the shoal area had extended upstream toward the center of the channel and caused divided flow (photograph 8). Although depths in the lower lock approach channel were greater, the shoal area extended farther downstream than with plan A-4, and there was little difference in the amount of maintenance dredging required. By the end of run 2, controlling depths in the crossing at mile 70.5 were only 8 ft below normal lower pool. Extending the left bank revetment increased depths over the crossing about 3 ft. Shoaling also occurred in the crossing approaching the cutoff channel (mile 68.4). Although project depths were obtained over the crossing, the alignment of the channel was not entirely satisfactory for navigation.

Plan A-6

41. Description. Plan A-6 was the same as plan A-5 except for modifications near the end of the lower guard wall designed to reduce shoaling in the lower lock approach. This plan involved the removal of the 480-ft wing dike and the addition of a 3000-ft extension to the guard wall as shown in plate 27. The alignment of the extension was such as to provide a channel with a bottom width of 200 ft and a depth of 12 ft below normal pool elevation.

42. Near the beginning of run 2, the dike at mile 71.9 was lengthened 30 ft, and the dike at mile 71.7 was shortened 40 ft to provide an opening between the guard wall extension and the ends of these dikes of 950 ft.

43. Results. Results obtained at the end of run 1 indicated that the long guard wall would tend to move the location of the shoal area downstream without any material effect on the amount of maintenance dredging required (photograph 9). Testing of this plan was discontinued when modifications of the lengths of the dikes mentioned above did not produce any

significant change in the material moved into the approach channel.

Plan A-7

44. Description. Plan A-7 was the same as plan A-6 except that the length of the lower guard wall extension was reduced to 1000 ft and a splitter wall was installed downstream of the dam starting at the end of the pier separating the sills of different elevations. The splitter wall extended 600 ft downstream of the end of the pier and had a top elevation of 172.0. However, before the beginning of the high-water period of the first run, the splitter wall was removed and a 1200-ft-long training or vane-type dike with a top elevation of 165.5 was installed along the right side of the channel in the crossing at about mile 70.4. About midway of the first half of the run, shortly after the start of the high-water period, additional training dikes of 700- and 650-ft lengths with top elevation of 165.0 were placed just downstream of the 1200-ft dike (plate 28). Near the middle of the run, the extension to the lower guard wall was extended 1000 ft downstream, making a total length of 2000 ft.

45. Results. Testing of plan A-7 was exploratory in nature to determine modifications that might be effective in reducing or eliminating shoaling in the lower lock approach and in the crossing at mile 70. Results of the tests of the modifications indicated the following (plate 29):

a. The length of the guard wall extension would affect the location of the shoal but have little effect on the amount of shoaling.

b. The splitter wall downstream of the dam appeared to improve action at the stilling basin and reduced scouring downstream of the end sill. However, the wall was removed early in the run after it became apparent that the deeper channel would form along the splitter wall and that the wall would increase the flow to the right side of the channel downstream of the wall, thus increasing shoaling in the lock approach.

c. The training dikes or crossing groins (mile 70.4) produced some increase in channel width opposite the dikes, but had little effect on depths in the channel over the crossing just downstream.

Plan A-8

46. Description. Plan A-8 was the same as plan A-7 except that the

extension to the lower guard wall of plan A-7 was replaced with a 480-ft-long wing dike having crest elevation of 164.0 (same as in plan A-5) and the training dikes in the crossing at mile 70 were removed. In addition, the procedure for operating the dam gates during controlled river flows was modified to determine its effect on shoaling in the lower lock approach channel. During this test, all gates were opened the same regardless of the depth of tailwater over the gate sill.

47. Results. The results of the test of plan A-8, shown in plate 30, indicate that the change in the gate operating procedure increased the amount of shoaling along the lower guard wall side of the channel although the amount of material in the approach channel was about the same as in the previous test. The deposition in the lock approach channel occurred landward of the wing dike and close along the revetment. There was some reduction in depths upstream of the dam on the lock side which could have been caused by the reduction in flow on that side during controlled river stages. Shoaling in the crossings near miles 70 and 68.2 had increased, reducing controlling depths by about 2 ft.

Plan A-9

48. Description. Plan A-9 was the same as plan A-8 except for the modification of the dike system upstream of the dam and realignment of the channel and regulating structures downstream as follows (plate 31):

a. Dikes upstream of the dam (between mile 73.4 and the dam) were shortened so that their ends would form a straight line extending from the right abutment of the spillway perpendicular to the axis of the dam. The controlling width of the channel at mile 73.4 provided by this alignment was approximately 1300 ft.

b. The left bank revetment downstream of the lock was lengthened to about mile 70.4 and realigned on a radius of 11,240 ft, providing a controlled channel width of 840 ft at its lower end.

c. The dike system along the right bank downstream of the dam (between the dam and mile 71.0) was modified to increase the contraction near the end of the lower guard wall to 800 ft. A stone-fill revetment extending from the right abutment of the spillway along the ends of the first two dikes was included as part of the system. When flow over the revetment scoured the fill behind the revetment during high flows, an additional spur dike was added downstream of the end of the first dike opposite the end of the wing dike.

d. The right bank revetment between miles 70.8 and 69.5 was realigned on a radius of 10,800 ft and extended 1700 ft downstream, contracting the channel to 900 ft at its lower end (mile 68.4).

e. The wing dike at the end of the lower guard wall was removed, but was restored at the beginning of run 2.

49. Results. The results shown in plate 32 indicate that shortening of the right bank dikes upstream of the dam would increase flow through that side of the dam during uncontrolled flows. Scouring increased downstream of the stilling basin on the right side of the dam and a relatively deep channel developed along most of the rock-fill revetment. A channel of sufficient depth but limited width was maintained in the lower lock approach, and the amount of maintenance dredging was reduced appreciably.

50. A channel of generally good alignment and adequate depth developed in the reach downstream of the lower lock approach and in the crossings of miles 70 and 68.2. Water-surface elevations at the dam were higher (as much as 1.5 ft) with this plan than those with plan A-8 as a result of the added contraction of the channel downstream.

Plan A-10

51. Description. Plan A-10 was the same as plan A-9 except for the following (plate 33):

a. Alignment of the revetment along the right bank immediately downstream of the dam was modified.

b. Revetment was placed from the first dike upstream of the dam to the right abutment pier of the dam.

c. Three wing dikes were placed downstream of the end of the lower guard wall. Tops of the dikes sloped from el 166.0 at the guard wall to 164.0 at the downstream end of the dikes.

52. Results. Results of tests of this plan at the end of one run are shown in plate 34. Material from the left side of the channel moved over the lower two wing dikes and into the downstream approach channel during high flows. By the end of the high stages, the lower two dikes were practically covered with sediment, and some material moved into the approach through the opening between the first and second wing dike (photograph 10).

The amount of maintenance dredging required was higher than with plan A-9. Water-surface elevations were about 1.0 ft lower than those with plan A-9 with uncontrolled river flows. There was some reduction in the width of the channel over the crossing at mile 70, but depths and channel width increased over the next crossing downstream.

Plan A-11

53. Plan A-11 was the same as plan A-10 except for the installation of a 1240-ft-long training dike extending downstream from the pier between spillway gate bays 8 and 9. The dike curved toward the lock side of the river with the end 400 ft riverward of the land side of the lower guard wall (plate 33). At the start of the test, the top of the dike at the upper and lower ends was at el 167.0 and 166.0, respectively. Modifications were made during the run which included the moving of the downstream end 60 ft toward the left bank and raising the end near the dam to el 173.0. Near the end of the run, the two wing dikes farthest away from the guard wall were removed. Since observations indicated that these modifications would not be effective in reducing shoaling in the lower approach, the test was discontinued before it was completed.

Plan A-12

54. Plan A-12 was the same as plan A-10 except for the following (plate 35):

a. The right bank revetment from the dam downstream to about mile 71.8 was modified to provide a channel width of 800 ft.

b. The revetment along the right bank upstream of the dam was extended to the right bank to reduce disturbance near the end of the dike.

c. Two training dikes 1000 and 1300 ft long extending from the dam piers on the right side of gates 6 and 12 were placed downstream; their crest elevations varied from 173.0 at the dam to 166.0 at the downstream end. The alignment and lengths of these dikes were developed during a series of preliminary tests. The training dikes were simulated with pieces of sheet metal.

d. The cell at the end of the lower guard wall was streamlined with a piece of sheet metal extending from the river side of the guard wall to the river side of the cell.

e. The three wing dikes at the end of the lower guard wall were removed and a 480-ft-long wing dike was installed with crest elevation varying from 169.0 to 165.0, upstream to downstream.

f. Gate operation was modified as follows: gates 1-12 only were operated to control flows up to 60,000 cfs; with flows of 35,000 cfs or less, the openings for gates 1-6 were twice as much as for gates 7-12; with flows above 35,000 and including 60,000 cfs, gates 1-12 were opened equally; with flows above 60,000 cfs, all gates were opened equally.

55. Observations indicated that shoaling in the lower lock approach would continue with the structures and gate operation procedures of plan A-12 and the test was discontinued before the completion of run 1.

Plan A-13

56. Description. Plan A-13 was the same as plan A-12 except for modification of the left and right bank revetments between miles 72.0 and 68.5 as follows (plate 36):

a. The radius of curvature of the left bank revetment (between the lock and mile 70.3) was increased from 11,240 to 15,200 ft and extended downstream 400 ft, providing a controlled channel width of 850 ft at the end of the revetment.

b. The radius of the right bank revetment between miles 71 and 68.5 was reduced from 10,800 to 9800 ft between the PCC and PT, and the upper 1560 ft of the revetment was changed from a curve to a tangent. The above changes, together with the gate operating procedure mentioned for plan A-12, were designed to improve depths over the crossing at mile 70.5 and to reduce shoaling in the lower lock approach channel.

c. The two training dikes extending downstream of the dam were left in place and lengthened 200 ft before the start of run 2 to determine their effectiveness in reducing shoaling in the lower lock approach channel. The length of the left dike (extending from the pier between gates 6 and 7) was reduced 100 ft shortly after the start of the run.

57. Results. Results of tests of plan A-13, shown in plates 37 and 38, indicate that modification of the revetments along the left and right banks downstream of the dam produced a satisfactory channel through the reach from just below the lower lock approach channel to the lower end of the model reach; depths and widths of the channel over the crossings at miles 70.5 and 68.4 increased during run 2, with good channel alignment.

58. Shoaling in the lower lock approach channel continued during the

run although a channel of limited width was maintained along the revetment. When the two training dikes were extended downstream, a deep scour hole developed at the end of the dike on the lock side of the channel soon after the start of run 2. The length of the lock-side dike was shortened 100 ft and the depth of the scour was reduced appreciably. The longer training dikes produced some reduction in the amount of shoaling in the lower approach, but the difference was not appreciable.

59. The distribution of flow through the dam gates during uncontrolled river flows and the height of deposition over the gate sill are shown in table 2. These results indicate that flow through the dam was fairly uniform except for gates 1-4 and 17 and 18 with the 160,000-cfs flow. The distribution of flow was better with the 255,000-cfs flow; however, in each case flow through gates 1 and 2 was less than half of the flow through most of the other gates. This is attributed to the direction of currents from the approach channel moving through the ports in the upper guard wall toward the spillway. It also appeared that with higher (uncontrolled) flows, most of the sediment passed through the gates near the lock.

Plan A-14

60. Description. Plan A-14 was the same as plan A-13 except that all gate sills were set at el 158.0 and the stilling basin was set at el 145.0. After installation of the modified dam, the model bed was molded to the conditions obtained at the end of the test of plan A-13. Controlled river stages were maintained by opening each gate the same amount.

61. Results. The distribution of flow through the revised dam was generally better (table 2). The amount of material deposited over the gate sills was considerably less and more evenly distributed. The amount of material deposited in the lower approach channel was about the same as with plan A-13 (plate 39). Scouring along the right bank revetment extending from the right abutment of the dam was less than with plan A-13. Good channel depths and alignment were maintained downstream of the dam.

Plan A-15

62. Description. The results of previous tests indicated that most of the sediment passed through the dam along the left side where it could

be moved toward and into the lock approach channel. The training dikes downstream of the dam were designed in an effort to force the movement of sediment toward the right side and away from the approach channel after passing through the dam. Since the structures placed downstream of the dam had only a limited effect on shoaling in the lower approach, the next series of tests was designed to reduce the movement of sediment through the left side of the dam. Accordingly, plan A-15 was developed during preliminary tests with structures upstream of the dam. The structures included in plan A-15 were the same as for plan A-14 except for the following (plate 40):

- a. Three training dikes (A, B, and C) were installed upstream of the dam. The dikes, with a crest elevation of 182.0 and lengths of 840, 1050, and 510 ft from upstream to downstream, were placed at a slight angle to the direction of flow in an effort to divert bottom currents and sediment.
- b. Alignment of the right bank revetment extending from the dam downstream was revised without affecting the controlled channel width near the downstream end.
- c. The wing dike at the end of the lower guard wall was sloped by raising the upstream end from el 164.0 to 167.0 before the start of run 4.

63. In addition to the above, the bed of the model was molded to reproduce the channel configurations obtained at the end of test of plan A-9 (plate 32), and the method of gate operation was modified as follows:

- a. Gates 14-18 were closed with flows of less than 40,000 cfs and gates 1-4 were opened equally to compensate for the closure.
- b. Gates 15-18 were closed with flows of 40,000 to 75,000 cfs and gates 1-4 were opened equally to compensate for the closure.
- c. Openings of other gates with flows of 75,000 cfs and lower, and for all gates with flows greater than 75,000 cfs were the same.

64. After completion of run 2, the following gate operating schedule was used:

- a. Gates 13-18 were closed with flows of less than 40,000 cfs and gates 1-12 were opened the same amount.
- b. Gates 15-18 were closed with flows from 40,000 to 75,000 cfs and gates 1-14 were opened the same amount.

c. With flows above 75,000 cfs all gates were opened the same amount.

65. Results. The results of tests of plan A-15, shown in plates 41 and 42, indicate that the training dikes and gate operating schedule produced a considerable change in the configuration of the bed upstream of the dam and in the distribution of sediment through the dam. A considerable amount of bed material moving along the channel side of the training dikes was diverted toward the right, forming a shoal area to the right and downstream of dike A (farthest upstream) and a shoal area extending from dike B to dike C. Scouring occurred at the upper end of dike B and a channel formed between the shoal areas of dikes A and B. The channel along the ends of the right bank dikes upstream of the dam was deepened. The channel along the left sides of the training dikes was deepened, and some shoaling occurred in the approach channel upstream of the end of the upper guard wall.

66. Material moving through the dam was generally small with flows of less than 70,000 cfs; with flows of about 70,000 to 160,000 cfs, good movement of bed material was observed to the right of the line of training dikes and most of the material moved through gates 6-10 and 13-18. The amount of material moving through the gates on the left side of the dam (gates 6-10) appeared to be greater than that moving through the other gates. With the highest flow reproduced (255,000 cfs) most of the material moved from the shoal area downstream of dike C through dam gates 10-14.

67. A considerable amount of the material moving through the dam continued toward the right side of the channel, reducing the depth of the channel along the revetment extending from and downstream of the dam; and the amount of material deposited in the channel along the lock side downstream of the dam was less than that observed in other tests. The amount of material deposited in the lower lock approach was reduced progressively during each run. Channel depths of 12 ft below normal lower pool were maintained in the lower lock approach over a width of at least 100 ft during runs 3 and 4 (plate 42). The amount of material deposited during runs 3 and 4 was about 25 percent of that measured during tests of plans A-12

and A-14. Satisfactory channel depths and alignment were maintained in the reach downstream of the lock approach.

Navigation Study

68. Before continuing the investigation for the development of regulating structures required to produce satisfactory channel dimensions, special tests were conducted to determine navigation conditions in the lock approaches. For these tests, the discharge scale was modified to conform with the Froudian scale. Operation of the model during these tests and observation of the model towboat indicated that downbound tows would experience considerable difficulty in approaching the upper guard wall because of the effects of currents from the left overbank moving riverward. The effects of these currents could be minimized by the construction of a fill along the left top bank. It was determined during these tests that a fill extending from the esplanade to a point at least 1800 ft upstream of the end of the upper guard wall would be required to eliminate crosscurrents in the reach where tows would have to reduce speed and lose steerageway. The elevation of the fill would have to be sufficient to eliminate flow from the overbank up to the maximum navigable discharge.

Tests for Development of Regulating Structures, Plans A-16 Through A-19

Plan A-16

69. Description. Plan A-16 was the same as plan A-15 except that the downstream dike (dike C) in the upper pool was moved 200 ft toward the left bank and maintained parallel to its alignment for plan A-15 and fill along the left overbank, as developed in the navigation study mentioned above, was placed at el 194.0 and extended 1800 ft upstream of the end of the upper guard wall. The gate operating schedule was the same as used in tests of plan A-15, runs 3 and 4.

70. Results. The results of tests of this plan, shown in plate 43, indicate that moving dike C toward the left bank produced some backwater effect. Scouring occurred upstream of the end of the dike and shoaling

occurred in the upper lock approach channel. Scouring at the upstream end of the next dike upstream (dike B) was also increased with some increase in the depth of the channel along the right bank. There was little change in the conditions downstream of the lock. Shoaling in the lower lock approach had increased. This change could be attributed to developments upstream caused by the relocation of the dike upstream of the dam which increased flow toward the right side of the dam.

Plan A-17

71. Description. Plan A-17 was the same as plan A-16 except that dike C in the upper pool was removed and the upper lock approach channel was dredged to el 165.0 to remove the material deposited in the approach channel during tests of plans A-15 and A-16. Other conditions and the gate operating schedule were the same as for plan A-16.

72. Results. The results of tests of this plan, shown in plate 43, indicate that removal of dike C would have little effect on shoaling in the upper lock approach channel. The depth of scour at the upper end of dike B was reduced, and a shoal area appeared along the river side of the upper guard wall. Shoaling in the lower lock approach was considerably less than with the previous test (table 1), and a channel of limited width was maintained in the lower lock approach.

Plan A-18

73. Description. Plan A-18 was the same as plan A-17 except that dike B was removed, and the material deposited in the upper lock approach channel during the test of plan A-17 was removed to el 165.0.

74. Results. The results of tests of plan A-18, shown in plate 44, indicate that shoaling in the upper lock approach channel would be increased with the removal of dike B. The center bar formed with the three dikes upstream of the dam (plans A-15 to A-17, plates 42 and 43) was reduced in elevation and length. There was no significant change in developments downstream of the dam.

Plan A-19

75. Description. Plan A-19 was the same as plan A-18 except that training dike A along the center of the channel upstream of the dam was removed, and the bed of the model from miles 74 to 71 was molded to

reproduce conditions obtained at the end of the plan A-15, run 1 (plate 41). Operating procedures for tests of this plan were the same as for plans A-15 to A-18.

76. Results. Results of tests of plan A-19 are shown in plates 44 and 45. These results indicate that the rate of shoaling in the upper lock approach was reduced from that obtained in tests of plans A-16 to A-18. Depths in the lock approach at the end of run 3 were greater near the end of the upper guard wall than at the end of run 2, indicating the probable effects of developments with the center dikes removed. Developments downstream of the dam were not affected appreciably by the modification of this plan. The amount of material deposited in the lower lock approach channel averaged about 80,000 cu yd per run (table 1).

Swellhead Tests

77. Tests were conducted to obtain swellhead information for use by the Little Rock District in design of the structures and to check results obtained by analytical methods. Tests were designed to obtain the following:

- a. Swellhead at the dam prior to overtopping of the embankments with steady flow and with rapid-rise hydrograph.
- b. Swellhead at the spillway (main channel) and the overbank after the embankments were overtopped with steady flow and rapidly rising stages.
- c. Data for use in checking analytical methods used by the District.
- d. Conditions with the tailwater higher in the overbank than in the main channel since this would affect the proportion of flow through the spillway with respect to overbank flow and the protection required for the overflow embankments.

Swellhead tests 1 and 2

78. Swellhead tests 1 and 2 were conducted with the model reproducing the channel configuration and regulating structures obtained at the end of plan A-14, run 2 (plate 39) except that the training dikes or vanes immediately downstream of the spillway were removed, and the revetment extending from the right dam abutment downstream was modified to

conform with the plan A-10 condition (plate 33).

79. Flows were controlled in accordance with rating curves (plates 46 and 47) furnished by the Little Rock District for steady flows and rapid-rise hydrograph based on the May 1943 flood. The differences in the model and prototype discharges were estimated flows over the overbank area not reproduced in the model. The flow passing through the old channel at Bayou Meto Cutoff was withdrawn through the old channel in accordance with the schedule shown in plate 46. Tests were conducted initially with selected steady flows and then with the rapid-rise hydrograph. Gage readings were obtained at special intervals during the tests, and discharge measurements were obtained with the 500,000-cfs flow to determine the distribution of flow over the left overflow embankment, through the spillway, and along the extension of the axis of the dam over the right overbank. Discharges were reproduced using the theoretical discharge scale based on the linear scales of 1:80 vertically and 1:120 horizontally with model roughness assumed to be correct. The tests were conducted with various conditions as outlined below.

	Test 1	Test 2
Crown of left overflow embankment, ft msl	193.0	192.0
Top of esplanade, ft msl	195.0	194.0
Top of lock walls, ft msl	193.0	194.0
Crown of right overflow embankment, ft msl	185.0	186.0

80. Water-surface elevations and overbank velocities measuring during these tests are shown in tables 3-7. Discharge measurements in the model indicated a flow distribution of 10, 70, and 20 percent for the left overflow embankment, spillway, and right overbank, respectively, with the 500,000-cfs flow as it was reproduced in the model with the test 1 condition. Considering the estimated flow over the overbank not reproduced in the model, the distribution of flow would be about 18, 64, and 18 percent for the left overbank, spillway, and right overbank, respectively. With the test 2 condition, the flow along the left overbank was increased 2 percent with lowering of 1 percent through the spillway and over the right overbank.

81. With the conditions of test 1, the swellhead across the left overbank with steady flow varied from 1.1 to 1.3 ft with the 500,000-cfs and 1.8 to 2.0 ft with the 350,000-cfs flows (table 3). The drop through the spillway and along the right overbank was comparatively small for these flows. The swellhead with varying hydrograph was increased about 0.2 to 0.3 ft (table 4).

82. Lowering of the crest of the left overflow embankment (test 2) decreased the swellhead over the embankment to a maximum of 1.7 ft which was obtained with the 350,000-cfs steady flow just at the time the embankment began to be overtopped. There was only a slight increase with varying hydrograph run.

83. Velocity measurements obtained along the right overbank indicated little difference with the 500,000-cfs flow for the two conditions tested (table 5). Observation of currents along the overbank indicated that a navigable pass should be located near the right abutment or near the levee because of the alignment of currents and the high ground between the two locations mentioned.

Swellhead test 3

84. Tests 1 and 2 were conducted with the model reproducing the conditions obtained at the end of run 2 of plan A-14 (plate 39) except for modification of the regulating structures as mentioned in paragraph 78. Conditions for swellhead test 3 were the same as for test 2 except that the vertical scale was made the same as the horizontal scale to eliminate distortion of the linear scales. In addition to the change in the vertical scale, a navigable pass was placed in the overbank section near the right dam abutment as shown in plate 48 and the fill developed during the navigation study was placed along the left bank with crest at el 194.0 from the esplanade to a point 1800 ft upstream of the end of the upper guard wall.

85. Tests with steady flow were conducted with and without the fill along the left bank. Tests with the rapid-rise hydrograph were conducted without the fill.

86. Results of this test, shown in tables 8-11, indicate that the water-surface elevations upstream of the dam and the swellhead at the left overflow embankment were higher than in test 2. This is attributed

partly to the effect of surface tension which is greater with the small vertical scale, and partly to the roughness which should have been less with the new linear scale but was not changed from that of test 2. The fill along the left bank upstream of the lock had little effect on stages along the left overbank upstream of the overflow structures except with the 350,000-cfs flow when the structure was beginning to be overtopped (tables 8 and 11). The distribution of flow in percent obtained at three measuring ranges with flows of 400,000 and 500,000 cfs is shown in table 9. The distribution of flow shown indicates an increase in flow along the left overbank for the 500,000-cfs flow over that obtained in test 2, with some decrease in discharge through the spillway. This increase can be attributed to the higher stage and greater drop across the overflow structure. Currents moving through the navigation pass were generally straight with a slight angle to the left, particularly with the lower flows, and velocities were less than 5 fps (table 10). Observations indicate no serious difficulties for tows navigating the pass.

Cofferdam Tests

87. These tests were conducted to obtain indications of scour and flow patterns that could be expected with various cofferdam plans and the extent of protection required, and to develop methods that could be used to reduce swellhead through the contracted reach and to move the point of maximum scour away from the base of the cofferdam. Both two- and three-stage construction was considered for the project.

88. During the first series of tests (tests A-J) model scales were the same as used during the tests of improvement plans (1:80 and 1:120); for the second series of tests (tests 1-14), the model was converted to eliminate the distortion of the linear scales. In the latter series of tests, the vertical scale was reduced to 1:120, the same as the horizontal scale. All tests, except tests 11-14 which were designed to simulate conditions in Lock and Dam No. 6, were conducted with conditions as obtained at the end of the base test (plate 8). All proposed structures included in the model since that test were removed from the model. Before the

cofferdam tests were undertaken, the model was operated with a 300,000-cfs constant flow to develop the configuration of the bed that could be expected with the flow to be used in testing the cofferdam. The results obtained during this test, designated test A, were used to determine the effects of the cofferdam on channel developments (plate 49).

89. The cofferdam consisted of cells 63.66 ft in diameter and spaced 67.62 ft center to center with the layout as shown in plate 50. The elevation designated for the top of the cofferdam varied from 190.0 at the downstream arm to 193.0 at the upstream arm. In the model, the cofferdam was set at an elevation of 197.0 to prevent it from being overtopped during the tests. The elevation required for the prototype cofferdam would then be based on water-surface elevations measured in the model with the 300,000-cfs flow.

90. All cofferdam tests including the base test were conducted with a steady-flow discharge of 300,000 cfs only. This flow was reproduced and bed material introduced continuously until the bed of the model and depth of scour became stable except as noted in description of test. Except for test A, tests were started with the cofferdam in place and the bed of the model molded to conform to the configuration obtained at the end of test A (plate 49). During tests with the second-stage cofferdam, the lock gates were opened, allowing flow through the lock. Upstream and downstream miter gate sills were set at el 164.0 and 148.0, respectively.

Test B

91. Description. Test B was a test of stage I of a two-stage cofferdam located as shown in plate 50. The lock and spillway bays 1-11 would be enclosed in the cofferdam. Since the upper guard wall extending outside of the cofferdam would be constructed sometime during stage I, it was included in the model during the test.

92. Results. Results of test B indicate that the maximum scour would occur against the cofferdam at its upstream corner, reaching a depth of a little more than 100 ft below the original bed (plate 50). The depth of scour was limited to some extent by the base of the model which was at el 58.0. Scour extended along the upstream arm about 500 ft from the upstream corner to about 300 ft from the downstream corner along the

downstream arm. Scouring in the channel along the right bank was slight. Material scoured from along the face of the cofferdam was deposited along and downstream of the lower arm of the cofferdam. The maximum water-surface elevation along the upstream face of the cofferdam was about 190.6 (table 12). Water-surface elevations indicated a drop of 0.6 ft in water surface at the upstream corner of the cofferdam (between gages E and F, plate 50). Total drop through the reach (gages A to J) was about 0.9 ft.

Test C

93. Description. Conditions for this test were the same as those for test B except that a spur dike was installed at the upstream corner of the cofferdam (plate 51) designed to move the point of maximum scour away from the cofferdam. The dike simulated a two-row, three-pile clump structure with stone fill. Top elevation of the piling was 192.0; top of the stone was at el 173.0 at the cofferdam, and sloped to el 165.0 at the end. The dike was 120 ft long set at an angle of 20 degrees riverward of a line extending along the river arm of the cofferdam.

94. Results. Results of this test indicated that the installation of the spur dike would reduce scour along the upstream face of the cofferdam. The area of maximum scour was developed just downstream of the end of the spur dike although the depth of scour was limited by the base of the model; the area of the scour hole appeared to be larger than in test B. The maximum depth of scour was moved at least 100 ft from the nearest face of the cofferdam. With the spur dike, depth of scour along the right side of the channel was increased.

95. Maximum water-surface elevation along the upstream face of the cofferdam was increased about 0.4 ft to el 191.0 (table 12). The drop in water surface across the spur dike was about 1.2 ft between gages E and F. There was little difference in the water-surface elevation farther upstream from the cofferdam.

Test D

96. Description. Conditions for this test were the same as those for test C except that the spur dike at the upstream corner of the cofferdam was angled 30 degrees to the left of the line forming an extension of the center of the cells forming the river-side face of the cofferdam

(plate 52). The spur dike was 120 ft long and similar in construction and elevation to the spur dike used in test C.

97. Results. Results of test D indicate that the spur dike angled away from the diversion channel would reduce the maximum depth of scour by at least 12 ft from that obtained in tests B and C (plate 52). The point of maximum scour was about the same distance away from the face of the cofferdam as with the test C dike. Scour along the land side and around the end of the spur dike and near the downstream corner of the cofferdam was somewhat less than in test B. Also, the depth of scour along the right side of the diversion channel and downstream was considerably less than with the test C dike. The dike angled away from the diversion channel appeared to be more beneficial than the dike angled toward the channel since it tended to streamline flow through the opening between the cofferdam and right bank resulting in a reduction in the maximum scour and scour along the right bank.

98. Maximum water-surface elevations (table 12) along the upstream face of the cofferdam were about 0.4 ft higher (el 191.4) than with the test C dike, due mostly to the change in velocity head caused by the angle of the dike with respect to the direction of flow along the upper face of the cofferdam. Total drop in water-surface elevations through the diversion channel (gages A to J) was about 1.2 ft.

Test E

99. Description. Test E conditions were the same as test C conditions except that the 120-ft-long spur dike was set at an angle of 10 degrees instead of 20 degrees riverward of an extension of the center line of the group of cells forming the diversion channel arm of the cofferdam (plate 53).

100. Results. Results of this test indicate that scour along the face of the upstream arm of the cofferdam and around the end of the spur dike would be less than in test C. The maximum depth of scour and the general scour pattern along the left side of the diversion channel were about the same as in test C. Scour along the right bank of the diversion channel was less, reducing the depth about 10 ft.

101. Water-surface elevations (table 12) were about the same as

obtained in test C except along the center of the diversion channel (gages G and H) where they were about 0.4 ft lower.

Test F

102. Description. Test F involved the study of conditions with stage II of the two-stage cofferdam. The layout of the cofferdam and the completed lock and portion of the dam are shown in plate 54. Elevation of the top of the cofferdam was set higher than specified to ensure that it would not be overtopped during the tests. A stone blanket extending 325 ft downstream of the stilling basin was placed on a 1-on-8 slope to the fixed base of the model and an 80-ft rock apron was placed level at el 155.0 upstream of the dam. Operating procedures for the test were the same as those for the stage I cofferdam. The lock gates and gates in the completed bays were open.

103. Results. In this test excessive scour occurred near the end of the upper guard wall, at the upstream corner of the cofferdam, and immediately downstream of the stone blanket. At these points, the depth of scour was limited by the fixed base of the model.

104. Maximum water-surface elevation along the upstream face of the cofferdam was 191.4 and the drop in water-surface elevation through the dam (gages L and M) was about 1.3 ft (table 13). Velocities through the spillway gates varied from 13.6 to 16.2 fps and velocities through the lock and over the right overbank were about 17.8 and 6.0 fps, respectively (table 14). Discharge measurements indicated a flow distribution of 79.7, 14.4, and 5.9 percent for the spillway, lock, and right overbank, respectively.

Test G

105. Description. Conditions for test G were the same as those for test F except for the addition of a spur dike at the end of the upstream guard wall and at the upstream corner of the cofferdam as shown in plate 55. The dikes were designed to reduce the amount of scour obtained near the structures in test F. The dike at the end of the upper guard wall was 100 ft long, with a top elevation of 173.0, set at an angle of 15 degrees riverward of the center line of the guard wall. The dike at the upstream corner of the cofferdam was 185 ft long, with a top elevation of 173.0, set at an angle of $26^{\circ}35'$ landward of a line extending upstream from

the face of the cofferdam arm normal to the axis of the dam.

106. Results. Installation of the dike at the end of the upper guard wall had little effect on the maximum depth of scour. The scour area was smaller and away from the structure (plate 55). There was only a slight reduction in the maximum depth at the upstream corner of the cofferdam and little change in the scour downstream of the stilling basin except that a scour hole developed along the side of the lower guard wall.

107. Water-surface elevations during this test were about the same as those obtained in test F. Velocities and flow distribution were not affected appreciably by the installation of the two spur dikes (table 14).

Test H

108. Description. Test H was a test of stage II of the three-stage cofferdam. This stage II cofferdam would be constructed after completion of the lock and appurtenant walls. The layout of the stage II cofferdam included in this test is shown in plate 56. As in other tests, the cofferdam was constructed to a confining grade. The lock gates with upper and lower miter gate sills at el 164.0 and 148.0, respectively, were kept open during the test.

109. Results. Results of this test indicate that considerable scour would occur along the upstream side and corner of the cofferdam (plate 56). Maximum scour along the upstream face of the cofferdam and at the upstream corner was limited by the fixed base of the model.

110. Maximum water-surface elevation along the upstream face of the cofferdam was 190.8 (table 15). Total drop in water-surface elevations through the diversion channel was 0.7 ft between gages G and H and 1.1 ft between gages A and J. Discharge measurements (table 15) indicated a flow distribution of 10.2, 83.9, and 5.9 percent for the lock, channel, and right overbank, respectively.

Test I

111. Description. Conditions for test I were the same as those for test H except for the addition of a 120-ft-long spur dike set at angle of 30 degrees to the left of a line perpendicular to the axis of the dam (plate 57). The dike simulated a two-row, three-pile clump structure with stone fill. The top elevation of the piling was 192.0; the crest elevation

of the stone was 173.0 at the cofferdam, and sloped down in an upstream direction to el 165.0 at the end of the structure.

112. Results. Results of this test indicate that the dike was effective in eliminating scour along the upstream face of the cofferdam (plate 57). Scouring occurred along the upstream side of the dike and extended around the end and channel side of the dike and part of the cofferdam. The maximum depth of scour occurred channelward of the upstream corner of the cofferdam. A relatively deep channel developed, starting about 300 ft channelward of the upstream corner of the cofferdam and extending diagonally toward and along the right bank downstream of the cofferdam.

113. Maximum velocities of 9.4 fps obtained along the cofferdam at gage E and 8.9 fps 40 ft to the left of the dike. Maximum water-surface elevation along the upstream face of the cofferdam was 191.4 (gage C, table 15). The total drop through the diversion channel was about 0.3 ft between gages G and H, and about 1.0 ft between gages A and J. Discharge measurements (table 15) indicated a flow distribution of 9.5, 84.3, and 6.2 percent for the lock, channel, and right overbank, respectively.

Test J

114. This test was the same as test I except that the spur dike was lengthened to 240 ft (plate 58). Results indicated that scour around the end and along the channel side of the dike was reduced about 10 ft with the longer dike. There was some increase in the scour area along the right bank downstream of the cofferdam.

115. Velocity measurements indicated a maximum velocity of 14.3 fps, 11.9 fps along the cofferdam at gage E, and 7.7 fps 40 ft to the left of the dike. Maximum water-surface elevation along the upstream side of the cofferdam was 191.5, slightly higher than with the shorter dike (table 15). Total drop through the diversion channel was about 0.9 ft (between gages A and J). Discharge measurements (table 15) indicated a flow distribution of 10.9, 82.7, and 6.4 percent for the lock, channel, and right overbank, respectively.

Test 1

116. Description. Test 1 was a test of stage II of the two-stage

cofferdam with test conditions the same as those for test G except that the model was converted to an undistorted scale of 1:120 and the length of stone blanket downstream of the stilling basin was reduced 50 ft. Conditions for this test were as follows (plate 59):

a. A rock dike 100 ft long with a top elevation of 173.0 was installed at the end of the upper guard wall and set at an angle of 15 degrees riverward of the center line of the guard wall.

b. A rock dike 180 ft long with a top elevation of 173.0 was installed at the left upstream corner of the cofferdam and set at an angle of 26°35' landward of a line perpendicular to the axis of the dam, forming an extension of the angled corner of the cofferdam.

c. A stone blanket extended 275 ft downstream of the stilling basin on a 1-on-8 slope and 30 ft upstream of the dam at el 155.0.

117. Results. Results of this test indicated that the scour pattern with the undistorted model was generally similar to that obtained with test G. Depths of scour were greater, due mostly to the change in vertical scale and reduced velocities that were not sufficient to move bed material into the scoured area from upstream.

118. Water-surface measurements indicated a maximum water-surface elevation of 191.5 along the upstream side of the cofferdam and a drop of 1.1 ft through the spillway (gages L to M, table 16). The maximum water-surface elevation above the cofferdam was 0.1 ft higher than with the distorted model, and the drop in water-surface elevation through the spillway was 0.2 ft lower. Discharge measurements (table 17) indicated a flow distribution of 79.6, 13.3, and 7.1 percent for the spillway, lock, and right overbank, respectively, compared with a distribution of 80.7, 13.9, and 5.4 with the distorted model. Velocities in the spillway varied from about 10.1 fps, gates 1 and 2, to about 14.8 fps, gates 7 and 8; velocity through the lock was about 14.8 fps.

Test 2

119. Description. Conditions for this test were the same as those for test 1 except for modification of the stone protection in the vicinity of the structures (plate 60). These modifications included extending the stone protection upstream of the dam along the line of the upstream face

of the cofferdam, adding stone protection along the upper guard wall and along the river side of the lock wall downstream of the dam, and reducing the length of the rock apron downstream of the stilling basin to line up with the lower face of the cofferdam. The size of the rock protection placed in the model was approximately to scale.

120. Results. Results of this test indicated a considerable reduction in the depth and amount of scour near the upper guard wall and near the upper corner of the cofferdam (plate 61). Scour along the lock wall and in the lock approach was eliminated. Modification of the stone apron downstream of the stilling basin had little effect on scouring downstream of the stilling basin.

121. The results of velocity measurements over the stone protection and in the lock approaches are shown in plate 62. Average velocity through the lock was 14.7 fps and velocities in the lock approaches varied from about 4.6 to 8.7 fps. The modification of the stone protection had little effect on water-surface elevations and distribution of flow (tables 16 and 17).

Test 3

122. Description. Conditions for this test were the same as those for test 2 except that all ports in the upper guard wall were closed to determine their effect on scour along the wall.

123. Results. Results of this test indicated that closure of the ports would cause an increase in scour along the upper guard wall and move the area of maximum scour downstream of the stilling basin below the stone blanket closer to the lock wall than with test 2 (plate 63). The depth of maximum scour at the upstream corner of the cofferdam was about the same as that obtained in test 2, but the scour area was smaller.

124. Closing of the ports in the upper guard wall had little effect on water-surface elevations (table 16). The maximum velocity along the cofferdam was about 15.3 fps at the upstream corner, which was slightly higher than that obtained in test 2.

Test 4

125. Description. Test 4 was the same as test 2 except that the stone apron upstream and the stone blanket downstream of the spillway were

shortened 95 and 145 ft, respectively, and a stone blanket was added to protect the area where maximum scour occurred along the channel side of the dike at the upstream corner of the cofferdam (plate 64).

126. Results. Results of this test (plate 64) indicate that installation of the stone along the dike at the upstream corner of the cofferdam would cause the point of maximum scour to be moved away from the dike and cofferdam. The depth of scour at the new location was not as great as with test 2. The reduction in the amount of stone protection above and below the dam had no appreciable effect on the scour above and below the spillway. Some increase in scour occurred along and just downstream of the end of the upper guard wall. The modification in the stone protection and change in scour had no significant effect on water-surface elevations and distribution of flow (tables 16 and 17). The maximum velocity obtained along the cofferdam was 13.4 fps at the upstream corner of the cofferdam.

Test 5

127. Description. Test 5 was a test of stage I of the two-stage cofferdam, and test conditions were the same as those for test B (paragraph 91). This test was conducted with the undistorted model.

128. Results. Scour patterns and channel developments (plate 65) were generally similar to those obtained in test B (plate 50). The depth of scour was somewhat greater in this test; as in the case of test 1, this is attributed to the reduction in bed material moving from upstream because of the change in scale and reduction in velocities.

129. The maximum water-surface elevation along the upstream face of the cofferdam was about 190.8 (gages B and C, table 18), which was about 0.2 ft higher than in test B. The drop in water surface at the upstream corner of the cofferdam between gages E and F was 0.2 ft, and the total drop through the diversion channel (gages A and J) was about 1.2 ft, about 0.3 ft greater than observed in test B. The maximum velocity near the cofferdam obtained near its upstream corner and was 11.3 fps.

Test 6

130. Description. Conditions for this test were the same as those for test 5 except for the installation of the spur dike at the upstream

corner of the cofferdam (plate 66) as in test D. The dike was 120 ft long set at an angle of 30 degrees landward of the center line of the row of cells parallel to the channel. The dike simulated a two-row, three-pile clump structure with stone fill. The top elevation of the piling was 192.0; top elevation of the stone was 173.0 at the cofferdam, and sloped down in an upstream direction to el 165.0 at the end of the structure.

131. Results. The effect of the spur dike on scouring was similar to that noted in test D (plate 52). The point of maximum scour was moved channelward 150 ft from the toe of the dike (plate 66). The depth of maximum scour and of all scour areas was generally greater than in test D except along the upstream face of the cofferdam and upstream of the end of the dike.

132. Maximum water-surface elevation along the upstream face of the guard wall was about 191.3 (gages B and C), about 0.1 to 0.2 ft lower than in test D (table 18). The total drop in water-surface elevation through the diversion channel (gages A to J) was about 1.5 ft, about 0.3 ft more than obtained in test D.

133. Velocities of 1.3 to 8.7 fps were measured along the upstream face of the cofferdam with the maximum obtained at the upstream corner of the cofferdam; velocities along the channel side of the cofferdam were low and only about 5.0 to 6.1 fps along the downstream arm of the cofferdam. Maximum velocity observed in the vicinity of the dike was 12.3 fps.

Test 7

134. Description. Test 7 was a test of stage I of the two-stage cofferdam with the downstream arm of the cofferdam straightened from the downstream corner to the end of the lower guard wall (plate 67). A stone blanket 10 ft high and 15 ft wide at the top was placed along the cofferdam on the bed of the model before the start of the test; the side slope of the stone along the edge was 1 on 1.5. A stone spur dike 150 ft long was placed at the upper corner at an angle of 30 degrees to the left of a line perpendicular to the axis of the dam. Elevation of the top of the dike was 173.0 at the cofferdam and sloped down in an upstream direction to el 165.0 at the end of the structure.

135. Results. Results of this test indicated a considerable

reduction in scour along the upper face of the cofferdam and around the upper end of the dike compared with results obtained in test 6 (compare plates 66 and 67). Scour along the right bank was about the same as obtained in test 6. The deep scour hole opposite the upper corner of the cofferdam was moved farther away from the cofferdam and the depth of scour was about 10 ft less than that obtained in test 6. The outer portion of the stone blanket between the cofferdam and the scour hole was unraveled, and the loose stones formed a blanket along the cofferdam side of the scour hole. Just downstream of the scour hole, the stone blanket remained intact and bed material was deposited over the blanket along the lower arm of the cofferdam. The scour pattern developed in test 7 did not differ appreciably from those obtained from other similar tests. At the start of the test an eddy formed along the lower arm of the cofferdam, causing material scoured from the diversion channel below the spur dike to be deposited downstream. After the development of the scour hole, the eddy downstream of the cofferdam disappeared and currents were generally as shown in plate 68. Velocities along the face of the cofferdam were generally less than 6 fps, except along the spur dike where velocities as high as 11.3 fps were measured.

136. The maximum water-surface elevation along the upstream side of the cofferdam was about 191.3 (gages B and C) and the drop in water-surface elevation through the diversion channel was about 1.3 ft (gages A and J).

Test 8

137. Description. Conditions for this test were the same as those for test 7 except that the dike at the upstream corner of the cofferdam and the stone blanket along the cofferdam were of rock simulating A-1 riprap having a maximum diameter of 24 in., d_{35} of 12 in., and minimum diameter of 4 in. According to the linear scale the stone used in the model was almost twice the required size. The spur dike was 150 ft long with a top width of 40 ft at el 175.0 and had 1-on-1.5 side slopes (plate 69). The base of the dike and stone blanket were set to provide a minimum of 15 ft of stone below el 173.0.

138. Results. Results obtained during this test (plate 69) were generally similar to those obtained in test 7 (plate 67). The depth of

scour along the upstream face of the cofferdam was less, with little difference in the scour hole opposite the upstream corner of the cofferdam. The area of maximum scour was about 350 ft from the cofferdam, farther from the cofferdam than in tests 6 and 7. Only the slope of the dike and stone protection were undermined along the edge of the scour hole, and sufficient stone was available to form a blanket over the slope of the scour hole as indicated by the dashed line in plate 69. Stone on the crest of the dike and on the blanket around the cofferdam was not disturbed.

139. Maximum water-surface elevation at the upper arm of the cofferdam was about 0.7 ft less than in tests 6 and 7. The drop through the diversion channel (gages A and J) was about the same as in tests 6 and 7. The maximum velocity measured near the dike was about 10.6 fps.

Test 9

140. Description. Tests 1 to 8 were conducted with undistorted model scales which resulted in a reduction in velocities to the point that no material moved in the reach upstream and downstream of the structure. Test 9 was the same as test 8 except that bed material was introduced during the run where sediment movement was observed upstream of the contracted section to determine its effect on developments.

141. Results. The results of this test indicated that developments would be seriously affected by deposition downstream since model velocities outside the restricted reach were not sufficient to move bed material downstream. There was considerable deposition along the downstream arm of the cofferdam and a deep channel was scoured diagonally across the diversion channel and along the right bank (plate 70). Depths in the deep scour hole opposite the upstream corner of the cofferdam were about 50 ft less than in test 8, but scouring along the cofferdam near the lock side of the dike was increased.

142. Because of the deposition downstream, water-surface elevations upstream of the cofferdam and the drop through the diversion channel were considerably higher than in the other tests. The results of this test were seriously affected by the lack of sufficient velocities in the model to continue the movement of sediment downstream of the cofferdam.

Test 10

143. Description. Conditions for this test were the same as those for test 4 except that the spur dikes (at the upper end of the upper guard wall and the upstream corner of the cofferdam) were constructed of stone, and bed material was introduced upstream of the dam. The length of the dike at the end of the upper guard wall was 100 ft and the length of the dike at the cofferdam was 180 ft (plate 71). Both dikes had a crown width of 30 ft with top elevation of 173.0. The stone was graded approximately to scale and placed to provide no less than 15 ft of stone below el 173.0.

144. Results. Results of this test indicated that conditions upstream of the spillway were about the same as for test 4 except that the scour along the upper guard wall was eliminated. Scouring below the stone blanket downstream of the stilling basin was reduced appreciably. More material was deposited along the right side of the diversion channel downstream of the cofferdam. Water-surface elevations obtained in this test were about the same as for test 4 (table 16).

Test 14

145. Description. Test 14 was a test of stage I of a two-stage cofferdam plan proposed by the contractor for the construction of Lock and Dam No. 3. Features of the plan were as follows:

a. Cofferdam, with top elevation of 191.0 (constructed to confining grade in model), with alignment as shown in plate 79.

b. Stone protection constructed as shown by sections A-A and B-B in plate 79. Dimensions for depth of stone (A) and top width of blanket (B) shown in section B-B were as follows:

Cell No.	A Depth of Stone, ft	B Top Width of Blanket, ft
1-10	8	5
17-24	10	15
24-26	8	10
26-34	8	5

Stone protection for cells 12-14 was as shown in section A-A.

c. A spur dike oriented perpendicular to the axis of the dam. The dike was 270 ft long, 15 ft wide at the top, with

1.5-on-1 side slopes, and top elevation of 191.0.

d. The bottom of the dike and stone rested on the riverbed.

e. The test was started with channel bed as developed at the end of test A (plate 52).

146. Results. The results of this test, shown in plates 79 and 80 and table 18, indicated that the maximum depth of scour would be greater than with any of the other plans in which the spur dike was used in connection with the cofferdam. However, the maximum scour developed farther upstream than with the other plans and was about 300 ft from the cofferdam. Scouring extended upstream to just above the end of the dike with little scour along the upstream face of the cofferdam, and extended downstream along the cofferdam through the constricted reach.

147. Maximum water-surface elevation along the upstream side of the cofferdam was about 191.4, indicating that the cofferdam would be overtopped slightly with a 300,000-cfs flow along the side upstream of the dike (table 18); downstream of the dike, water-surface elevations were 190.2 at gage E and 190.1 at gage F. Current directions indicate an eddy forming along the left side of the upstream arm of the cofferdam and just downstream of the spur dike (plate 80). Velocities along the lower corner of the cofferdam were in the order of 8 to 9 fps. Except near the upper end, considerable deposition occurred along and downstream of the lower arm of the cofferdam.

Tests of Lock and Dam No. 6 Cofferdam

Test 11

148. Description. The purpose of this test was to determine conditions that could be expected with stage I of the two-stage cofferdam planned for Lock and Dam No. 6 on the Arkansas River. Conditions for this test were simulated with the stage I of the Lock and Dam No. 3 cofferdam modified in accordance with the alignment shown in plate 72. The protective stone, spur dike, and channel bed were the same as those used in test 8 for the Lock and Dam No. 3 cofferdam tests.

149. Results. The results of this test indicated that the area of

maximum scour would be farther downstream than with any other similar plans tested and would extend a considerable distance from the face of the cofferdam (about 450 ft). Considerable shoaling occurred downstream of the cofferdam, and the stone protection along the toe of the cofferdam from about the axis of the dam downstream was covered with sediment. A channel developed starting along the right side of the scour hole opposite the cofferdam and extending downstream generally parallel to the right bank. The scour along the right bank occurred opposite the cofferdam about 600 ft upstream of the location observed in previous tests.

150. Water-surface elevations along the upstream arm of the cofferdam varied from 191.1 at gage A to 190.2 at gage D compared with 190.6 at these gages in test 8. The total drop in water-surface elevation through the diversion channel was about 1.2 ft (gages G and J). Flow along the cofferdam and through the diversion channel was generally smooth with velocities 50 ft from and along the cofferdam generally less than 7 fps except near the spur dike (plate 73). Velocities varied from about 8 fps just upstream of the end of the dike to 12 fps near the cofferdam end of the dike.

Test 12

151. Description. Conditions for this test were the same as for test 11 except for the channel bed at the start of the test and the revetment along the left bank. The channel bed of the model was molded to reproduce conditions at Lock and Dam No. 6, based on data furnished by the Little Rock District, as shown in plate 74.

152. Results. The maximum depth of scour developed during this test was about the same as obtained in test 11. A channel of considerable depth (el 99.0) was formed along and generally parallel to the revetment line along the right side of the diversion channel. No failures in the stone protection were observed along the cofferdam except for some sloughing of the stone along the diversion side near the scour hole. The amount of stone available was more than sufficient to form a protective blanket along the cofferdam side of the scour hole. Deposition over the protective stone along the downstream arm of the cofferdam was noted at the start of test and most of the stone was covered with sediment by the end of the test.

153. Water-surface elevations along the upstream side of the cofferdam were about the same as in test 11 with a maximum of 191.0. The total drop in water-surface elevation through the diversion channel was about 2.0 ft (gages G and J). Water-surface elevation was 0.3 ft higher at gage G and 0.5 ft lower at gage J than in test 11. The water-surface elevation at gage J was affected to some extent by local conditions along the edge of the revetment where flow from the right overbank moved into the diversion channel (plate 75). Current directions and velocities shown in plate 76 indicate that flows along the cofferdam were generally similar to those obtained in test 11 (plate 73). Currents along the right side of the diversion channel and over the area landward of the revetment moved toward the left after passing the axis of the dam and were concentrated generally along the revetment line.

Test 13

154. Description. Conditions for this test were the same as for test 12 except that the size of the cofferdam was reduced to increase the distance between the channel side of the cofferdam and the right bank revetment (plate 77). This was accomplished by moving the diversion channel arm of the cofferdam 155 ft toward the left. Reducing the width of the cofferdam along the axis of the dam decreased the angle of the upstream and downstream arms of the cofferdam with respect to the normal direction of flow.

155. Results. Results of this test indicated that the scour pattern and the location of maximum scour with respect to the cofferdam would be generally similar to that obtained in test 12. The scour area along the upstream face of the cofferdam increased and the depth of maximum scour was greater by about 14 ft. The amount of stone unraveled along the channel face of the cofferdam was greater and deposition along the downstream arm of the cofferdam was less than in test 12. There was no indication that the stone protection along the channel face of the cofferdam was inadequate. Scour along the right bank revetment was considerably less than in test 12, except immediately opposite the cofferdam where depths had increased because of the concentration of overbank flow farther upstream as shown in plate 78.

156. Water-surface elevations along the upstream face of the cofferdam varied from about 191.4 to about 190.3 from gage A to gage D. The drop in water-surface elevation through the diversion channel (gages G and J) was about 1.4 ft, which was about 0.6 ft less than in test 12. Flow conditions and velocities were generally similar to those of test 12 except for the following (plate 78):

a. Flow from the overbank landward of the revetment moved into the channel farther upstream as mentioned above.

b. There was less tendency for currents to bend toward the left bank after passing the restricted reach.

c. Velocities were slightly higher near the spur dike and generally higher along the downstream arm of the cofferdam.

The results of this test indicate that the alignment of the spur dike was too close to the alignment of the upper arm of the cofferdam for the dike to have any appreciable effect on developments.

Navigation Tests of Plans A-9 and A-14

Description

157. Before tests on the model were concluded, a study was made to determine currents and velocities that would affect navigation, particularly in the lock approaches, with the plans developed. For these tests the model was remolded to undistorted scales to reproduce conditions developed during tests of plans A-9 and A-14. The regulating structures for these tests were the same as shown in plates 32 and 39, respectively, except for the addition of a fill along the left bank extending 2500 ft upstream of the esplanade. Preliminary tests indicated that with a flow moving from the overbank during high stages toward the spillway, strong currents would develop across the lock approach. Because of these cross-currents, conditions for downbound tows attempting to approach the guard wall would be difficult and hazardous during flows extending over the left overbank. It was determined during these tests that a fill or other structure would be required along the left overbank to cause flow from the left overbank to enter the channel a sufficient distance upstream to permit tows

to move through these currents before having to reduce speed and begin to lose steerageway.

Results

158. Current directions and velocities obtained during these tests are shown in plates 81-86. These results indicated that velocities would tend to be high in the upper approach (about 10 or 11 fps) with the 150,000-cfs flow and decrease with increase in discharge. The alignment of currents was generally straight and no serious difficulties were indicated. The differences in conditions between plans A-9 and A-14 upstream of the lock were not sufficient to have any significant effect on navigation conditions. Current directions and velocities in the lower lock approach and observations with the model towboat indicated that conditions for navigation would be better with the 480-ft wing dike of these plans during the 150,000- and 230,000-cfs flows. With the 350,000-cfs flow, conditions would be somewhat better without the dike because of the directions of the currents over the dike that would tend to move upbound tows landward (plate 86). Although this tendency increased with an increase in flow, tows should not encounter any serious difficulty in approaching the guard wall even with the maximum navigable flow. Flow from the spillway toward the left bank tended to resist the movement of the head of a downbound tow away from the left bank and some difficulty was experienced in making the turn downstream of the lock. To overcome this difficulty, it was necessary to position the towboat stern as far to the left as possible so that the entire tow was angled to the right with respect to the left bank. This condition could be improved considerably by excavation of the left bank downstream of the lock at least 50 ft landward to permit the stern of downbound tows to be moved landward before the head passes the end of the guard wall extension (wing dike).

PART IV: EVALUATION OF RESULTS AND CONCLUSIONS

Limitations of Model Results

159. In evaluating the results of tests of channel regulating works, it should be considered that the conventional process of verifying a movable-bed model was precluded in this case by the lack of necessary recorded data for the reach of the Arkansas River reproduced. The scale ratios established, therefore, can be considered to approximate only in a general way the relations between the model and various reaches of the prototype. The time scale in particular was by necessity established somewhat arbitrarily; and although it was used as a basis for reproducing the stage hydrograph (plate 2), it should not be considered an accurate indication of the time required in the prototype for model-predicted developments to occur under similar conditions.

160. Developments in the model occurred mostly as a result of bed movement. Although some of the coal forming the bed of the model channel was thrown into suspension during the higher stages, the model did not even roughly simulate the effects of suspended sediment in a river of this type.

161. In evaluating the results of tests of various plans, consideration must also be given to the fact that some tests were continued until the model reached stability while others were conducted only until the general effects of the plan being tested were indicated. All tests were conducted with the hydrograph shown in plate 2 and with full sediment load. The effect of a reduction in sediment load was not explored. Other factors that should be considered in the evaluation of results are as follows:

- a. Development of Bayou Meto Cutoff was accomplished with an assumed deterioration of Mud Lake Bend (old bendway).
- b. Portions of the overbank areas subject to overbank flow were not reproduced in the model, and model discharges during tests with considerable overbank flow had to be adjusted based on estimates furnished.
- c. Tests of plans for channel development were based on the hydrograph shown in plate 2 which was considered as typical of flow conditions in the reach of the river under study. Prolonged or unusual low- or high-water periods could

produce results somewhat different from those expected with a normal-flow hydrograph.

d. Cofferdam tests were conducted only with a flow of 300,000 cfs, which was continued until conditions became reasonably stable, a condition that can be considered as extreme. These tests were also affected to some extent by the various procedures and scale relations used. The depths of scour in the prototype could be affected by the movement of sediment into the scour area, magnitude and duration of flow, and the change in gradation and characteristics of material found at various depths. Developments in the model during some of these tests were affected by the fixed base of the model which restricted to some extent the depth of scour, by the change in scales which did not result in sufficient velocities to produce movement of the bed material except in the restricted reach, and possibly by the uniformity of the bed material from top of initial bed to the bottom of the scour hole.

162. It is believed that, in spite of the above limitations, the model adjustment resulted in the development of operating techniques and scale ratios sufficiently accurate to permit satisfactory reproduction of the characteristics of the Arkansas River. The reliability and usefulness of the results of tests of improvement plans are further supported by utilization of the base test. This test was conducted with the channel rectified and stabilized according to the normal bank stabilization plan for the river, that is, with the concave banks revetted or stabilized by dikes and crossings contracted to widths of 1000 to 1200 ft. By using the base test results as a basis for evaluating the results of the tests of other plans, it is possible to obtain an indication of the relative effectiveness of the various plans as well as general indications of the type of effects that they will produce.

163. The cofferdam tests were considered adequate in providing indications as to the scour pattern that could be expected with the various plans, relative effectiveness of various supporting structures, and the area that should be protected and degree of protection required.

164. The analysis of the results of navigation tests is based principally upon a study of current directions and velocities and the effects of these currents on the behavior of the model tow. The current directions and velocities were indicated by wooden floats submerged to a

depth of 8 ft (prototype). In evaluating test results, it should be borne in mind that small changes in the direction of flow or in velocities are not necessarily changes produced by a change in plan, since several floats introduced at the same point under the same flow conditions may follow different paths or move at different velocities, or both, because of pulsating currents and eddies.

Conclusions

165. The results, conclusions, and general indications developed from the model investigation are summarized as follows:

a. Results of the base test indicate that because of the long, straight reach in which the lock and dam will be located, the tendency for the channel to meander and make a crossing toward the right bank near the proposed site for the dam, noted in some of the prototype surveys and in the adjustment test, would continue after the installation of proposed regulating structures and the Bayou Meto Cutoff. Installation of the lock and dam structures would not eliminate this tendency.

b. Operation of the dam gates to maintain upper pool level will tend to induce shoaling in the reach upstream of the dam during controlled river flows. Location of deposition will vary with river discharge and amount of gate opening. The tendency for a channel to be maintained along the concave side and particularly in a flat bend upstream of the lock and dam would be reduced because of the increase in the water-surface elevation and decrease in velocities during low flows caused by the control of pool elevation.

c. Raising of the Lock and Dam No. 3 pool will not be sufficient to maintain navigable depths in the channel immediately upstream of the structures and in the upper lock approach without additional regulating structures. Spur dikes along the right bank to reduce flow along that side and force the extension downstream of the sandbar in the bend were effective in developing satisfactory depths in the approach channel. Without the movement of the sandbar, scour would normally occur along the ends of the spur dikes along the right bank just upstream of the dam, and their effectiveness in producing a channel along the opposite bank in the lock approach would be reduced. Vane-type dikes could be used to force the movement of sediment toward the right side and increase depths in the approach channel. However, if the dikes are not properly designed, scour would develop along the upstream ends of the dikes which would increase flow and depths along the right side of the channel. To be effective, these dikes would have to have a crest elevation at least up to the normal upper pool level.

d. Satisfactory channel depths were developed in the reach upstream of the lock and dam and in the upper lock approach with the regulating structures of plans A-9 and A-14. Conditions in the crossing at mile 75 appeared to be somewhat unstable and could be troublesome under certain flow conditions.

e. Navigation conditions in the upper approach to the lock would be seriously affected by flow from the left overbank moving across the approach channel toward the spillway. The hazardous effects of the crosscurrents could be eliminated by constructing a fill or dike along the left top bank, extending about 2500 ft upstream of the esplanade. Because of the need to maintain adequate channel depths, velocities in the upper approach channel will tend to be high during some flows, but should not seriously affect navigation since with adequate ports in the upper guard wall the direction of currents would tend to be straight.

f. Capacity of the ports in the upper guard wall should be sufficient to pass all of the flow the wall tends to intercept to eliminate crosscurrents near the end of the wall and the deep scour near the end and just downstream of the end of the wall. Increasing the height of the ports to 20 ft by lowering the bed at the bottom of the ports would be sufficient to eliminate the difficulties mentioned. To maintain a uniform size of ports from the upper to the lower end of the wall, the bottom of the ports should be fixed at the selected elevation.

g. The number of gate bays in the dam could be reduced from 22 to 18 without any appreciable effect on swellhead during uncontrolled river flows. There was little difference in water-surface elevation between the section of dam with stepped sill and that with the level sill. Distribution of flow and movement of sediment through the dam appeared to be better with the level sill.

h. Shoaling in the lower lock approach channel can be reduced but not eliminated as long as there is movement of sediment through the dam. Sediment deposited in the area below the dam during high flows will continue to be moved downstream and into the approach channel during most controlled river flows. A large-diameter cell at the end of the lower guard wall will increase the tendency for sediment to be moved into and deposited in the lock approach channel. The need to contract the channel downstream of the dam would tend to raise the tailwater elevation at the dam, to reduce the swellhead at the dam and scour downstream of the stilling basin, and to increase deposition of sediment below the dam in a location from which it can be moved into the lower lock approach. Gate operation schedules can be established to force some of the sediment deposited downstream of the dam to move to the right side of the channel away from the lower lock approach channel.

i. A wing dike at the end of the lower guard wall can be used to reduce the frequency of dredging, but would have little effect on the amount of dredging. The performance of the wing dike at the end of the lower guard wall and the factors affecting its performance could not be fully evaluated in these tests since other modifications were included in the plans tested that could have affected developments in the vicinity. It appears that the performance of wing dikes of this type would be affected by their crest elevation, angle with respect to the alignment of the lock wall, and length. Indications are that the elevation of the dike should be such as to permit surface flows to move over the top of the dike into the approach channel during most of the critical flows and which would tend to prevent bottom sediment-carrying currents from moving into the approach channel.

j. Since the wing dike would normally perform as an angled spur dike, it should tend to increase rather than decrease shoaling on the approach channel side. The performance of the wing dike therefore depends to a considerable extent on flow over the top of the dike which would tend to prevent bottom currents from moving toward the left side and into the approach channel. Increasing the length of the dike would tend to increase shoaling along the dike upstream to such an extent that sediment would move over the top of the dike into the approach channel. Placing a second dike or second and third dikes downstream and parallel to the original dike would tend to have the same effect as a longer dike with the same total deflection, and to cause sediment to move into the approach channel through the opening between dikes, if any. Extending the lower guard wall downstream would only move the shoaling area downstream.

k. The wing dike at the end of the lower guard wall (included in plan A-14) would improve navigation conditions in the lower approach except with flows above 250,000 cfs. With the maximum navigable flow the tendency for upbound tows to be moved landward would be greater with the wing dike. This effect did not appear to be sufficient to produce any serious difficulties.

l. Downbound tows in the lower lock approach would experience difficulty in moving riverward away from the left bank revetment after leaving the lock because of the direction of flow from the spillway toward the left bank. Conditions in the lower approach could be improved by providing additional area along the left bank to permit the stern of the tow to be moved landward before the head of the tow passes the end of the wing dike.

m. Plans were developed that produced satisfactory channel conditions with the hydrograph used for the investigation, except in the lower lock approach. Because of the relatively flat bends in the reach between the structures and the cutoff, the regulating structures would have to be extended into the crossing to provide the concentration of flow necessary to produce and maintain adequate channel dimensions and alignment over the two crossings. The regulating structures included in plan A-14 should produce a satisfactory channel downstream of the lower lock approach.

n. Cofferdam tests indicated that the point of maximum scour will normally occur near the upstream end of the cofferdam with little scouring of the diversion channel away from the cofferdam. The depth, location, and area of maximum scour can be affected by the alignment of the upper arm of the cofferdam. The extent of scour along the cofferdam is increased as the angle of the upstream arm with respect to the direction of flow is increased. Material removed from the scoured area would be deposited downstream of the cofferdam. Reducing the angle of the lower arm of the cofferdam and eliminating bends would reduce scour along that side of the cofferdam.

o. The area of maximum scour can be moved away from the cofferdam with a spur dike placed near the upstream corner. The location of maximum scour would depend on the length, elevation, and angle of the dike with respect to the diversion channel side of the cofferdam. A spur dike extending upstream from the upstream corner of the cofferdam would tend to increase the water-surface elevation along the upper arm of the cofferdam and reduce scour along that side. Scour along the opposite bank would depend on overbank flow entering the diversion channel downstream of the axis of the cofferdam.

p. Satisfactory navigation conditions insofar as currents are concerned would require the construction of a fill along the right bank along the upper approach channel extending about 2500 ft upstream of the esplanade and construction of the wing dike at the end of the lower guard wall. Conditions in the lower approach could be improved considerably by excavation of the left bank at least 50 ft landward to provide additional maneuver area for downbound tows leaving the lock.

Table 1

Material Dredged from Lower Approach Channel

Plans A to A-19

Plan	Run	Material Dredged cu yd	Plan	Run	Material Dredged cu yd
A	1	27,800	A-6	1	243,400
A	2	--	A-7	1	303,200
A	3	57,600	A-8	1	128,100
A	4	17,100	A-8	2	213,500
A	5	19,200	A-9	1	119,600
A-1	1	40,600	A-9	2	72,600
A-1	2	51,200	A-9	3	59,800
A-1	3	59,800	A-9	4	106,800
A-1	4	55,500	A-10	1	115,300
A-1	5	106,700	A-11	1	149,500
A-1	6	106,700	A-12	1	183,600
A-2	1	145,200	A-13	1	192,200
A-2	2	158,000	A-13	2	170,800
A-2	3	151,240	A-14	1	192,150
A-2	4	192,150	A-14	2	175,100
A-3	1	85,400	A-15	1	117,400
A-3	2	115,300	A-15	2	81,100
A-3	3	64,000	A-15	3	59,800
A-3	4	187,900	A-15	4	44,800
A-3	5	175,100	A-16	1	128,100
A-3	6	320,300	A-17	1	72,600
A-4	1	234,900	A-18	1	59,800
A-4	2	213,500	A-19	1	64,100
A-4	3	264,700	A-19	2	98,200
A-5	1	192,200	A-19	3	76,860
A-5	2	119,600			
A-5	3	243,400			
A-5	4	230,600			

Table 2

Distribution of Flow and Deposition of Bed Material at Dam

Run 2 of Plans A-13 and A-14

Gate No. Left to Right	Percent of Total Discharge		Height of Deposition over Gate Sill, ft	
	Plan A-13	Plan A-14	Plan A-13	Plan A-14
		18-ft Stage, 160,000 cfs		
1-2	5.7	5.0	8.0	8.0
3-4	9.1	11.0	8.0	2.4
5-6	11.3	12.1	8.0	1.6
7-8	12.7	10.6	0.0	3.2
9-10	13.5	12.3	0.0	1.6
11-12	10.1	11.0	0.0	1.6
13-14	15.2	12.6	0.0	0.0
15-16	13.6	14.5	0.0	0.0
17-18	8.8	10.9	0.0	0.0
		22-ft Stage, 255,000 cfs		
1-2	5.4	4.2	8.0	3.2
3-4	10.6	9.5	8.0	3.2
5-6	11.0	13.5	8.0	0.8
7-8	12.0	13.2	0.0	0.0
9-10	12.5	12.1	0.0	0.0
11-12	12.5	12.9	0.0	0.0
13-14	15.0	14.3	0.0	0.0
15-16	12.0	11.6	0.0	0.0
17-18	9.0	8.7	0.0	0.0

Table 3

Water-Surface Elevations

Swellhead Test 1, Constant Stage

Gage No.	Discharge, cfs 200,000	230,000	300,000	350,000	400,000	450,000	500,000
2	189.4	190.7	193.6	195.1	196.3	197.4	198.4
3	187.0	188.2	191.4	193.1	194.5	195.7	196.7
4	184.8	186.6	190.3	192.1	193.6	195.0	195.8
Upper pool	184.8	186.2	189.8	191.8	193.3	194.6	195.7
Lower pool	184.7	186.2	189.5	191.7	193.0	194.5	195.5
5	183.2	184.6	188.2	190.0	191.6	193.1	194.2
6	181.0	182.4	186.0	188.2	189.8	191.4	192.6
8	179.5	180.6	183.8	185.9	187.8	189.4	190.9
9	177.7	179.1	182.0	183.9	185.6	187.2	188.6
A	Dry	Dry	190.6	192.3	193.8	195.4	196.0
B	Dry	Dry	Dry	190.5	192.2	193.6	194.9
C	186.7	187.4	190.6	192.5	193.9	195.2	196.2
D	Dry	Dry	188.6	190.7	192.2	193.5	195.1
E	186.2	187.4	190.6	192.4	193.8	195.3	196.0
F	Dry	Dry	188.3	190.4	192.2	193.4	194.7
G	184.9	186.5	190.0	191.8	193.3	194.6	195.5
H	184.3	186.2	189.7	191.7	193.2	194.5	195.5
I	Dry	Dry	187.0	189.1	191.6	193.8	195.0
J	Dry	Dry	186.9	189.1	191.6	193.8	195.0

Note: All elevations are in feet referred to mean sea level.
Location of regular and special gages is shown in plate 49.

Table 4

Water-Surface Elevations

Swellhead Test 1, Varying Stage

	Discharge, cfs													
Gage No.	200,000	222,500*	230,000	252,000	275,000	300,000**	322,000	350,000	369,000†	400,000	416,000	450,000	480,000	500,000
2	189.0	††	190.5	††	††	194.2	††	195.0	††	196.2	††	197.4	††	198.4
3	186.6	††	188.2	††	††	191.4	††	193.4	††	194.6	††	195.5	††	196.7
4	184.2	††	186.1	††	††	190.1	††	192.1	††	193.8	††	194.7	††	195.8
Upper pool	184.2	185.2	185.7	187.1	188.0	189.4	190.4	191.6	192.2	193.3	193.7	194.3	195.2	195.5
Lower pool	184.2	185.1	185.6	186.8	188.0	189.3	190.2	191.4	192.1	193.1	193.5	194.2	195.0	195.4
5	182.5	††	184.2	††	††	188.2	††	190.0	††	191.8	††	193.1	††	194.4
6	179.6	††	181.8	††	††	185.8	††	188.3	††	190.0	††	191.4	††	192.9
8	177.0	††	179.2	††	††	183.2	††	185.8	††	187.8	††	189.3	††	190.9
9	174.1	††	177.0	††	††	181.2	††	183.8	††	185.5	††	187.0	††	188.5
A	Dry	††	Dry	††	††	Dry	191.1	192.6	193.1	194.0	194.3	195.1	195.6	196.2
B	Dry	††	Dry	††	††	Dry	189.4	190.5	191.1	192.0	192.4	193.3	194.3	194.9
C	184.5	187.0	187.1	188.2	189.2	190.3	191.3	192.6	193.1	194.1	194.4	195.1	195.5	196.2
D	Dry	Dry	Dry	Dry	Dry	185.8	189.3	190.4	191.1	192.0	192.5	193.4	194.4	195.0
E	184.4	187.0	187.1	188.0	189.0	190.2	191.2	192.4	193.1	193.9	194.2	194.9	195.5	196.0
F	Dry	Dry	Dry	Dry	Dry	185.8	189.1	190.4	191.0	191.9	192.3	193.3	194.1	194.9
G	184.2	185.4	186.2	187.3	188.6	189.8	190.7	192.0	192.7	193.4	193.7	194.6	195.4	195.8
H	183.7	184.7	185.8	186.8	188.0	189.5	190.5	191.5	192.2	193.3	193.5	194.3	195.1	195.4
I	Dry	Dry	Dry	Dry	Dry	185.8	187.9	189.7	190.6	192.2	193.0	193.8	194.4	195.4
J	Dry	Dry	Dry	Dry	Dry	181.5	187.4	189.6	190.5	192.1	192.9	193.8	194.2	195.2

Note: All elevations are in feet referred to mean sea level.
Gage locations shown in plate 49.

* Flow at which right embankment near dam was overtopped.

** Flow at which right embankment near levee was overtopped.

† Flow at which left embankment near levee was overtopped.

†† No reading.

Table 5

Right Overbank Velocities, Swellhead Tests 1 and 2

Discharge cfs	Test 1 Distance, ft, from Right End of Dam 200	400	600	Test 2 Distance, ft, from Right End of Dam 200	400	600
			Constant Stage			
350,000	4.4	4.9	4.3			
400,000	4.8	5.3	4.5	4.1	4.5	4.5
450,000	5.0	5.0	4.8	4.8	5.0	4.8
500,000	5.0	4.8	4.5	4.8	4.8	4.8
			Varying Stage			
350,000	4.3	5.0	4.5			
400,000	4.5	4.7	5.3	4.1	4.3	4.5
450,000	4.7	5.3	4.7	4.3	4.5	4.5
500,000	5.0	5.6	5.6	4.5	4.5	4.5

Note: Velocities are in feet per second.

Table 6

Water-Surface Elevations

Swellhead Test 2, Constant Stage

Gage No.	Discharge, cfs 350,000	400,000	450,000	500,000
2	195.0	196.2	197.4	198.4
3	193.0	194.4	195.7	196.7
4	192.1	193.5	194.7	195.8
Upper pool	191.6	193.1	194.4	195.4
Lower pool	191.6	193.0	194.3	195.4
5	190.0	191.6	193.0	194.2
6	188.3	190.0	191.4	192.7
8	185.9	187.8	189.3	190.7
9	183.9	185.6	187.2	188.6
A	192.1	193.5	194.8	195.6
B	190.4	192.2	193.7	195.1
C	192.2	193.5	194.6	195.6
D	190.5	192.2	193.6	195.0
E	192.2	193.5	194.8	195.6
F	190.5	192.2	193.6	194.9
G	191.7	193.1	194.3	195.5
H	191.5	193.0	194.2	195.4
I	189.0	191.7	193.8	194.8
J	188.9	191.6	193.4	194.8
K		194.1	196	196.0
L		193.9		196.3
M		193.9		196.2
N		193.8		195.9
O		193.4		195.6
P		193.1		195.5
Q		191.9		194.5

Note: All elevations are in feet referred to mean sea level.
Gage locations shown in plate 49.

Table 7

Water-Surface Elevations

Swellhead Test 2, Varying Stage

Gage No.	Discharge, cfs 322,000	350,000*	369,000	400,000	416,000	450,000	480,000	500,000
2	**	195.1	**	196.2	**	197.4	**	198.4
3	**	193.3	**	194.5	**	195.6	**	196.8
4	**	192.2	**	193.6	**	194.6	**	195.9
Upper pool	190.5	191.6	192.2	193.0	193.4	194.3	195.0	195.5
Lower pool	190.3	191.5	192.1	192.9	193.1	194.2	194.8	195.5
5	**	190.0	**	191.8	**	193.0	**	194.3
6	**	188.2	**	190.2	**	191.4	**	192.9
8	**	186.0	**	187.8	**	189.2	**	190.6
9	**	183.8	**	185.4	**	187.0	**	188.4
A	191.2	192.2	192.7	193.4	193.8	194.5	195.1	195.7
B	189.4	190.5	191.2	192.2	192.4	193.4	194.3	195.0
C	191.3	192.1	192.6	193.4	193.7	194.5	195.1	195.8
D	189.3	190.4	191.0	191.9	192.4	193.5	194.2	195.0
E	191.3	192.1	192.7	193.4	193.8	194.5	195.1	195.7
F	189.3	190.3	191.1	192.0	192.3	193.4	194.2	194.8
G	190.7	191.9	192.3	193.2	193.4	194.4	195.0	195.7
H	190.6	191.8	192.1	193.0	193.2	194.0	194.9	195.6
I	188.0	189.9	190.7	192.0	192.6	193.6	194.6	195.3
J	187.5	189.7	190.6	191.9	192.6	193.5	194.3	195.1

Note: All elevations are in feet referred to mean sea level.
Gage locations shown in plate 49.

* Flow at which left embankment was overtopped.

** No reading.

Table 8

Water-Surface Elevations

Swellhead Test 3, Constant Stage

Gage No.	200,000 Fill Out	200,000 Fill In	230,000 Fill Out	230,000 Fill In	300,000 Fill Out	300,000 Fill In	350,000 Fill Out	350,000 Fill In	400,000 Fill Out	400,000 Fill In	450,000 Fill Out	450,000 Fill In	500,000 Fill Out	500,000 Fill In
	Discharge, cfs													
2	189.7	189.7	191.4	191.3	193.9	194.0	195.7	195.7	196.8	196.8	197.6	197.6	198.7	198.8
3	186.7	188.6	188.6	188.6	191.6	191.7	193.4	193.5	194.8	194.9	195.8	195.8	197.1	197.1
4	184.7	184.7	186.8	186.7	190.3	190.2	192.1	192.0	193.5	193.4	194.6	194.5	196.1	196.1
Upper pool	184.2	184.2	186.2	186.1	189.8	189.7	191.6	191.6	193.3	193.2	194.4	194.4	195.7	195.7
Lower pool	183.9	183.9	186.0	186.0	189.3	189.4	191.4	191.4	193.1	193.1	194.3	194.3	195.6	195.5
5	182.7	182.7	184.7	184.7	188.0	188.1	189.9	189.9	191.6	191.6	192.8	192.8	194.0	194.1
6	181.1	181.1	182.4	182.4	186.0	186.0	188.3	188.3	190.1	190.1	191.4	191.4	192.8	192.7
A	Dry	Dry	Dry	Dry	Dry	Dry	192.5	192.9	193.9	194.0	195.0	195.1	195.8	195.9
B	Dry	Dry	Dry	Dry	Dry	Dry	190.4	190.5	192.5	192.5	193.5	193.6	195.3	195.4
C	186.1	186.1	187.9	187.8	190.6	191.0	192.5	192.8	193.9	194.1	195.0	195.1	195.9	195.8
D	Dry	Dry	Dry	Dry	187.2	187.2	190.4	190.5	192.4	192.5	193.4	193.7	195.2	195.3
E	186.2	186.1	187.9	187.9	190.7	191.0	192.6	192.9	194.0	194.1	195.0	195.2	195.8	195.9
F	Dry	Dry	Dry	Dry	187.2	187.3	190.4	190.4	192.5	192.5	193.4	193.5	195.2	195.5
G	184.5	184.4	186.2	186.1	189.7	189.7	191.5	191.5	193.3	193.3	194.4	194.3	195.7	195.6
H	184.2	184.2	186.1	186.1	189.7	189.7	191.5	191.5	193.3	193.3	194.3	194.3	195.6	195.6
I	Dry	Dry	Dry	Dry	Dry	Dry	188.7	188.7	190.9	190.9	192.3	192.4	194.3	194.3
J	Dry	Dry	Dry	Dry	Dry	Dry	188.7	188.7	190.9	190.8	192.1	192.2	194.0	194.1
K									194.7	194.9			196.7	196.8
L									194.4	194.4			196.6	196.7
M									194.4	194.5			196.7	196.7
N									193.4	193.5			195.7	195.7
O									193.4	193.3			195.7	195.7
P									193.2	193.3			195.5	195.5
Q									191.6	191.6			194.5	194.5
R									192.3	192.2			194.7	194.7
S									190.5	190.5			193.8	193.8
T									190.8	190.9			193.5	193.4
U									190.9	191.0			193.5	193.6
V									190.5	190.5			193.4	193.3

Note: All elevations are in feet referred to mean sea level.
Gage location shown in plate 49.

Table 9

Distribution of Flow

Swellhead Test 3, Constant Stage

Discharge, cfs	Location	Distribution of Flow, % Left Overbank	Channel	Right Overbank
400,000*	Range 22.5	14.4	71.4	14.2
	Axis of dam	6.0	79.7	14.3
	Range 26.5	3.9	78.6	17.5
400,000**	Range 22.5	17.4	68.9	13.7
	Axis of dam	9.2	77.0	13.8
	Range 26.5	7.2	75.9	16.9
500,000*	Range 22.5	18.4	63.0	18.6
	Axis of dam	10.0	71.0	18.4
	Range 26.5	6.7	69.8	23.5
500,000**	Range 22.5	26.0	57.1	16.9
	Axis of dam	18.3	65.0	16.7
	Range 26.5	15.3	63.4	21.3

* Flow adjusted to compensate for left overbank area not reproduced in model.

** Adjusted discharge added to the left overbank flow.

Table 10

Navigation Pass Velocities and Flow Directions

Swellhead Test 3, Varying Stage

Discharge cfs	Direction of Flow*	Velocity, fps, at Stations**			
		71+40	72+73	72+67	33+60
350,000	15° left	3.5	4.6	4.7	4.6
400,000	10° left	3.7	4.6	4.2	4.2
450,000	5° left	4.4	4.4	4.6	4.4
500,000	5° left	4.4	4.4	4.4	3.7

* Direction of flow is referred to a line through center of pass normal to the axis of dam.

** Stations are shown in plate 49.

Table 11

Water-Surface Elevations

Swellhead Test 3, Varying Stage

Gage No.	Discharge, cfs												
	200,000	230,000*	252,000	275,000	300,000	322,000	350,000**	369,000	400,000†	416,000	450,000	480,000	500,000
2	189.6	191.1	††	††	193.9	††	195.9	††	197.0	††	198.1	††	199.1
3	186.5	188.3	††	††	191.4	††	193.8	††	195.1	††	196.1	††	197.3
4	184.2	186.2	††	††	190.1	††	192.5	††	194.0	††	195.0	††	196.2
Upper pool	183.6	185.5	186.6	187.9	189.5	190.4	191.7	192.5	193.4	193.8	194.5	195.2	195.7
Lower pool	183.5	185.3	186.3	187.7	189.2	190.2	191.5	192.2	193.2	193.5	194.3	195.1	195.6
5	181.9	183.7	††	††	189.0	††	190.2	††	191.7	††	192.8	††	194.3
6	179.6	181.7	††	††	184.9	††	188.4	††	190.2	††	191.4	††	192.8
A	Dry	Dry	Dry	Dry	Dry	Dry	192.3	193.2	193.9	194.3	194.8	195.7	196.2
B	Dry	Dry	Dry	Dry	Dry	Dry	190.2	191.3	192.7	193.1	194.1	194.8	195.7
C	185.7	187.1	187.7	188.6	189.8	190.8	192.3	193.1	193.9	194.2	194.7	195.6	196.1
D	Dry	Dry	Dry	Dry	Dry	186.8	190.2	191.3	192.7	193.0	193.8	194.6	195.5
E	185.7	187.1	187.7	188.6	189.9	191.0	192.3	193.2	193.9	194.3	194.7	195.6	196.1
F	Dry	Dry	Dry	Dry	Dry	187.0	190.3	191.4	192.6	193.1	193.8	194.7	195.5
G	183.8	185.9	187.1	187.4	189.6	190.5	191.7	192.6	193.5	193.4	194.5	195.1	195.1
H	183.0	184.9	187.1	187.4	189.4	190.1	191.4	192.2	193.4	193.3	194.1	194.6	195.1
I	Dry	Dry	Dry	Dry	Dry	187.8	189.1	190.1	191.4	191.9	192.8	193.6	194.5
J	Dry	Dry	Dry	Dry	Dry	187.5	188.9	189.9	191.2	191.8	192.7	193.5	194.4

Note: All elevations are in feet referred to mean sea level.
Gage locations shown in plate 49.
* Flow at which right embankment near dam was overtopped.
** Flow at which left embankment was overtopped.
† Flow at which upper lock gate was overtopped. Water-surface elevation 194.1 at upper lock gate.
†† No reading.

Table 12

Water-Surface Elevations, Tests B-E

Two-Stage Cofferdam, Stage I

Discharge 300,000 cfs

Gage No.	Water-Surface Elevation			
	Test B	Test C	Test D	Test E
2	192.8	193.1	193.1	193.1
3	191.5	191.6	191.5	191.4
4	190.6	191.0	190.7	190.7
5	188.6	188.6	188.8	188.6
6	186.7	186.6	187.0	186.9
8	184.5	184.4	184.7	184.5
9	182.0	182.0	182.0	182.0
A	190.4	190.6	190.8	190.5
B	190.6	190.8	191.4	190.7
C	190.6	190.8	191.3	190.7
D	190.6	191.0	190.9	190.7
E	190.4	191.0	191.0	191.0
F	189.8	189.8	190.3	189.8
G	190.3	190.6	190.4	190.2
H	190.2	190.5	190.3	190.0
I	189.9	189.9	190.2	189.8
J	189.5	189.4	189.6	189.5

Note: All elevations are in feet referred to mean sea level.
Lettered gage location shown in plate 51; numbered gage location in fig. 2.

Table 13

Water-Surface Elevations, Test F-G

Two-Stage Cofferdam, Stage II with Dikes

Discharge 300,000 cfs

Gage No.	Water-Surface Elevation Test F	Test G	Gage No.	Water-Surface Elevation Test F	Test G
2	194.0	193.9	K	191.1	191.0
3	192.0	191.9	180*	191.0	190.8
4	191.4	191.4	60*	191.0	190.8
5	188.6	188.5	L	190.9	190.7
6	187.0	187.0	60**	190.8	190.6
8	184.6	184.7	120**	190.7	190.6
9	182.0	182.0	180**	190.0	189.8
K	191.1	191.0	240**	189.2	189.0
L	190.8	190.7	325**	187.9	187.8
M	189.5	189.6	360**	188.7	188.6
N	189.6	189.8	420**	189.0	189.0
O	191.2	191.0	480**	189.1	189.0
P	189.3	189.4	540**	189.3	189.4
Q	191.4	191.4	600**	189.4	189.5
R	189.2	189.5	M	189.5	189.6
			120†	189.5	189.4

Note: Lettered gage location shown in plate 55; numbered gage location in fig. 2.
All elevations are in feet referred to mean sea level.
* Distance in feet from L on line from K to L.
** Distance in feet from L on line from L to M.

Table 14

Velocities and Distribution of Flow, Tests F and G

Two-Stage Cofferdam Tests, Stage II

Position	Velocity, fps	
	Test F	Test G
Gates 1-2	13.6	14.9
Gates 3-4	14.8	15.3
Gates 5-6	13.7	13.6
Gates 7-8	16.2	14.6
Gate 9	13.6	13.6
Lock	17.8	18.3
Right overbank	6.0	6.0

Position	Distribution of Flow, Percent of Total	
	Test F	Test G
Spillway	79.7	80.7
Lock	14.4	13.9
Right overbank	5.9	5.4

Table 15

Water-Surface Elevations and Flow Distribution, Tests H-J

Three-Stage Cofferdam, Stage II

Discharge 300,000 cfs

Gage Number	Water-Surface Elevation Test H	Test I	Test J
2	193.6	194.1	194.1
3	191.3	191.3	191.4
4	190.4	190.7	191.0
5	188.5	188.8	189.0
6	187.0	186.8	186.8
8	184.5	184.6	184.6
9	182.0	182.0	182.0
A	190.2	190.4	190.5
B	190.6	191.3	191.4
C	190.8	191.4	191.4
D	190.8	191.2	191.5
E	190.4	191.0	191.2
F	189.8	190.2	190.2
G	190.2	190.4	190.5
H	189.5	190.1	190.4
I	189.2	189.6	189.7
J	189.1	189.4	189.6
K	189.0	189.0	189.0

Position	Distribution of Flow Percent of Total Plan H	Plan I	Plan J
Lock	10.2	9.5	10.9
Channel	83.9	84.3	82.7
Right overbank	5.9	6.2	6.4

Note: Lettered gage location shown in plate 57; numbered gage location in fig. 2.
All elevations are in feet referred to mean sea level.

Table 16

Water-Surface Elevations, Tests 1-4 and 10

Two-Stage Cofferdam, Stage II with Dikes

Undistorted Scale

Gage No.	Water-Surface Elevation				
	Test 1	Test 2	Test 3	Test 4	Test 10
2	194.0	194.0	194.1	194.0	194.1
3	192.7	192.6	192.7	192.6	192.6
4	191.7	191.5	191.6	191.6	191.7
5	187.8	187.8	187.8	187.8	187.8
K	191.3	191.1	191.3	191.1	191.3
L	190.6	190.5	190.5	190.4	190.6
M	189.5	189.7	189.7	189.6	189.6
N	188.9	189.1	188.9	188.9	189.0
O	190.6	190.5	190.6	190.6	190.7
P	188.7	188.6	188.9	188.9	189.0
Q	191.5	191.7	192.0	192.0	191.7
R	189.0	189.0	189.1	189.0	189.0
K	191.3	191.1	191.3	191.1	191.3
180*	190.7	190.5	190.6	190.6	190.7
60*	190.7	190.5	190.6	190.5	190.7
L	190.7	190.5	190.5	190.4	190.6
60**	190.6	190.2	190.3	190.3	190.6
120**	190.6	189.7	189.7	189.8	189.8
180**	190.4	189.6	189.7	189.7	189.7
240**	190.5	190.4	190.4	190.3	190.6
300**	188.0	187.8	188.0	187.9	188.2
360**	188.7	188.5	188.9	188.9	189.1
420**	189.0	189.0	189.2	189.2	189.3
480**	189.1	188.9	189.2	189.3	189.4
540**	189.1	188.9	189.2	189.3	189.4
600**	189.2	188.7	189.2	189.4	189.4
M	189.5	188.7	189.7	189.6	189.6
120†	189.1	188.6	189.6	189.6	190.3

Note: Lettered gage location shown in plate 60; numbered gage location in fig. 2.
All elevations are in feet referred to mean sea level.
* Distance in feet from L on line from K to L.
** Distance in feet from L on line from L to M.
† Distance in feet from M on line from M to N.

Table 17

Velocities and Distribution of Flow, Tests 1-4

Two-Stage Cofferdam Tests, Stage II

Undistorted Scale

Gate No.	Velocity, fps Test 1	Test 2
1-2	10.1	10.1
3-4	14.3	14.7
5-6	14.5	14.5
7-8	14.8	14.7
9	14.3	13.9
Lock	14.8	14.7
Right overbank Near dam	7.6	7.6
Right overbank Near levee	0.6	0.6

Position	Distribution of Flow, Percent of Total Test 1	Tests 2 and 3	Test 4
Spillway	79.6	79.3	79.5
Lock	13.3	13.3	13.2
Right overbank	7.1	7.4	7.3

Table 18

Water-Surface Elevations, Tests 5-9 and 11-14

Two-Stage Cofferdam, Stage I

Discharge 300,000 cfs

Gage No.	Water-Surface Elevation								
	Test 5*	Test 6*	Test 7*	Test 8*	Test 9*	Test 11**	Test 12**	Test 13**	Test 14†
2	193.7	193.8	193.6	193.8	194.5	194.9	195.1	195.5	195.3
3	192.1	192.1	192.0	192.0	193.1	192.7	192.5	192.6	192.7
4	190.9	191.1	191.1	191.0	192.3	191.4	191.3	191.1	191.5
5	187.8	187.8	187.8	187.8	187.8	187.8	187.8	187.8	187.8
A	190.4	190.5	190.3	190.6	191.7	191.1	191.2	191.4	191.4
B	190.8	191.3	191.3	190.6	192.5	190.9	191.0	190.8	191.4
C	190.8	191.1	191.3	190.6	192.2	190.7	190.6	190.4	191.4
D	190.2	190.9	190.8	190.6	192.5	190.2	190.3	190.3	191.1
E	190.3	190.8	190.6	190.3	192.2	190.2	189.6	189.7	190.2
F	190.1	189.8	190.4	189.7	191.0	189.7	189.2	189.7	190.1
G	190.2	190.4	190.3	190.4	190.9	190.4	190.7	190.4	190.7
H	189.9	189.8	189.6	189.7	189.8	190.3	190.2	190.2	190.6
I	189.5	189.5	189.1	189.3	189.7	189.9	189.8	189.7	189.8
J	189.2	189.0	189.0	189.2	188.7	189.2	188.7	189.0	189.2

Note: Numbered gage location shown in fig. 2.
All elevations are in feet referred to mean sea level.
* Lettered gage location shown in plate 66.
** Lettered gage location shown in plate 73.
† Lettered gage location shown in plate 80.

Photograph 1. Base test, run 1, 0-ft stage. View from about mile 73.0, showing bar formation along left bank near proposed damsite, indicating tendency for channel to meander

Photograph 2. Base test, run 1, 0-ft stage. View from about mile 72.2 showing channel along pile revetment downstream of proposed damsite

Photograph 3. Base test, run 1, 0-ft stage. View from about mile 69.4 showing crossing at mile 68.5 and channel into and through cutoff

Photograph 4. Plan A, run 2, 2.0-ft stage, pool el 167.1. Bar has formed near end of lower guard wall and along right bank dikes

Photograph 5. Plan A-3, run 1, -1.0-ft stage. Bed configuration downstream of dam. Note bar in front of right bank dikes

Photograph 6. Plan A-3, run 4, -1.0-ft stage, pool el 165.2. Bar formation in lower lock approach; lower end of guard wall in foreground

Photograph 7. Plan A-5, run 4, water surface lowered to el 168.0. Note bar formation opposite ends of right bank dikes

Photograph 8. Plan A-5, run 4, water-surface el 164.0. Note bar formation and division of channels below the dam

Photograph 9. Plan A-6, run 1, pool el 166.1. Note shoal area in the approach channel downstream of the end of the guard wall extension

Photograph 10. Plan A-10, run 1, -1.0-ft stage. End of lower guard wall and wing dikes downstream. Note deposition over and downstream of lower two wing dikes and in lock approach channel

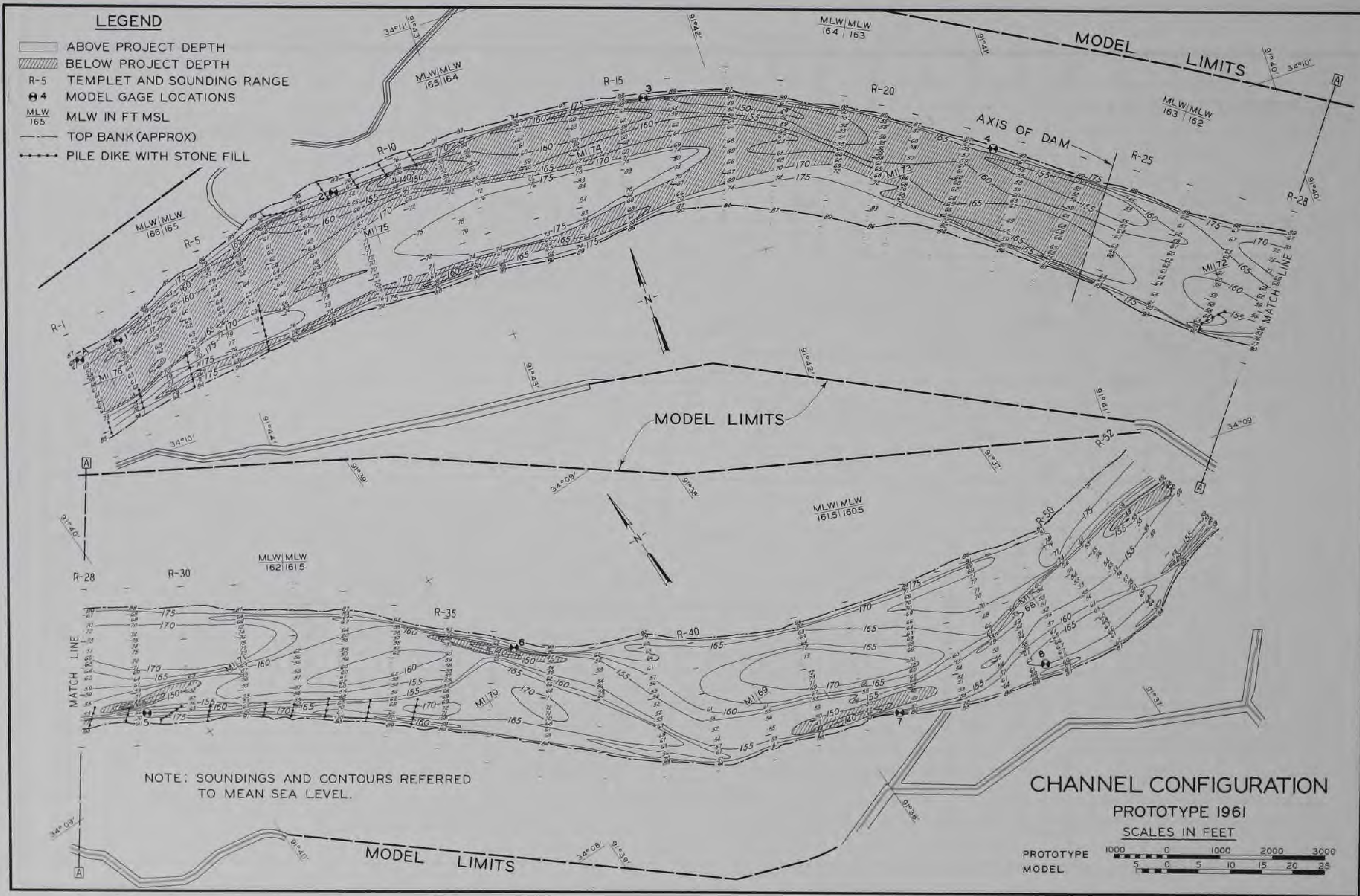
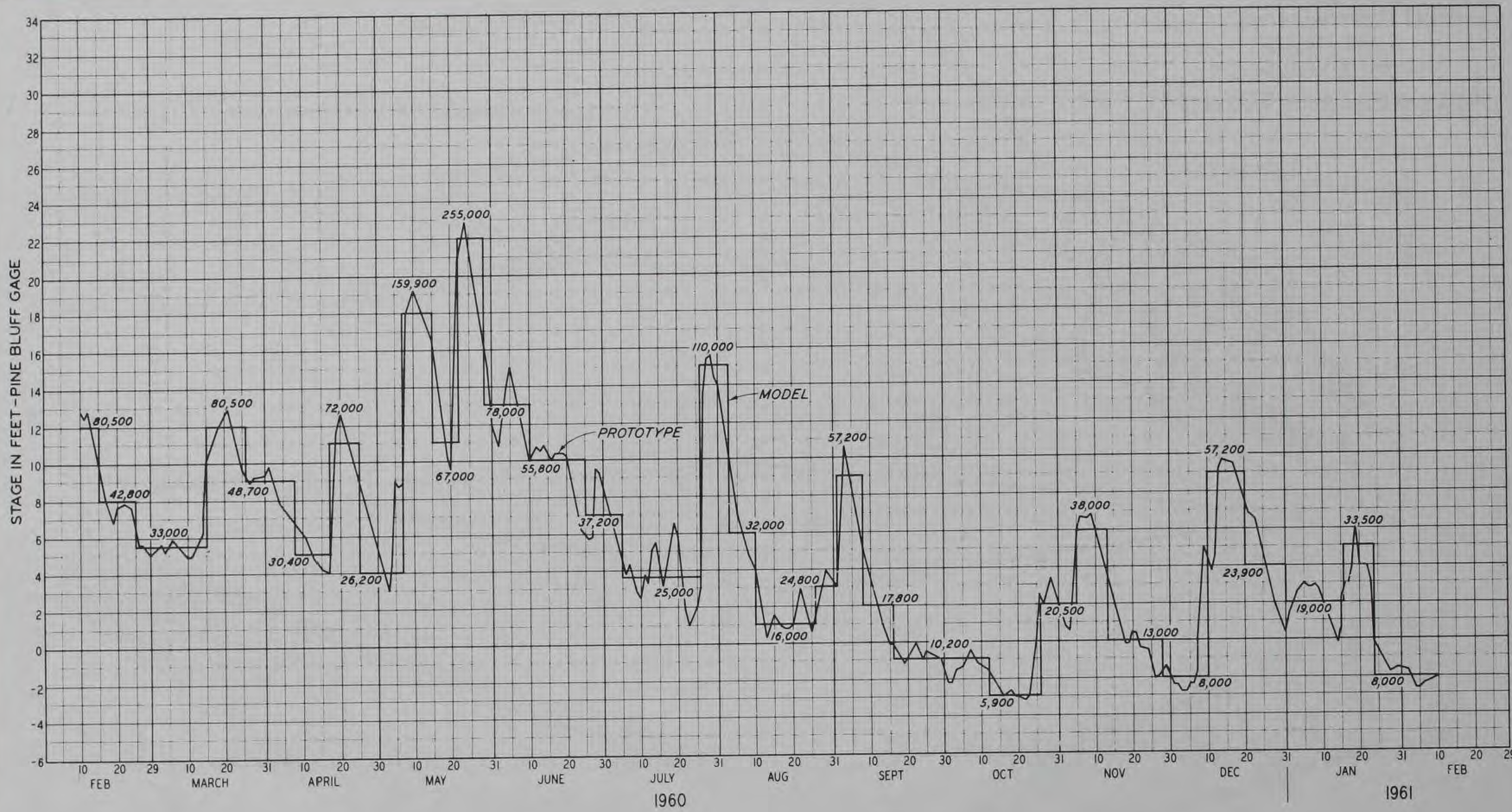

LEGEND
ABOVE PROJECT DEPTH
BELOW PROJECT DEPTH
R-5 TEMPLET AND SOUNDING RANGE
4 MODEL GAGE LOCATIONS
MLW / 165 MLW IN FT MSL
TOP BANK (APPROX)
PILE DIKE WITH STONE FILL
MODEL LIMITS
AXIS OF DAM
MODEL LIMITS
MATCH LINE
MATCH LINE
MODEL LIMITS
NOTE: SOUNDINGS AND CONTOURS REFERRED TO MEAN SEA LEVEL.
CHANNEL CONFIGURATION
PROTOTYPE 1961
SCALES IN FEET
PROTOTYPE 1000 0 1000 2000 3000
MODEL 5 0 5 10 15 20 25

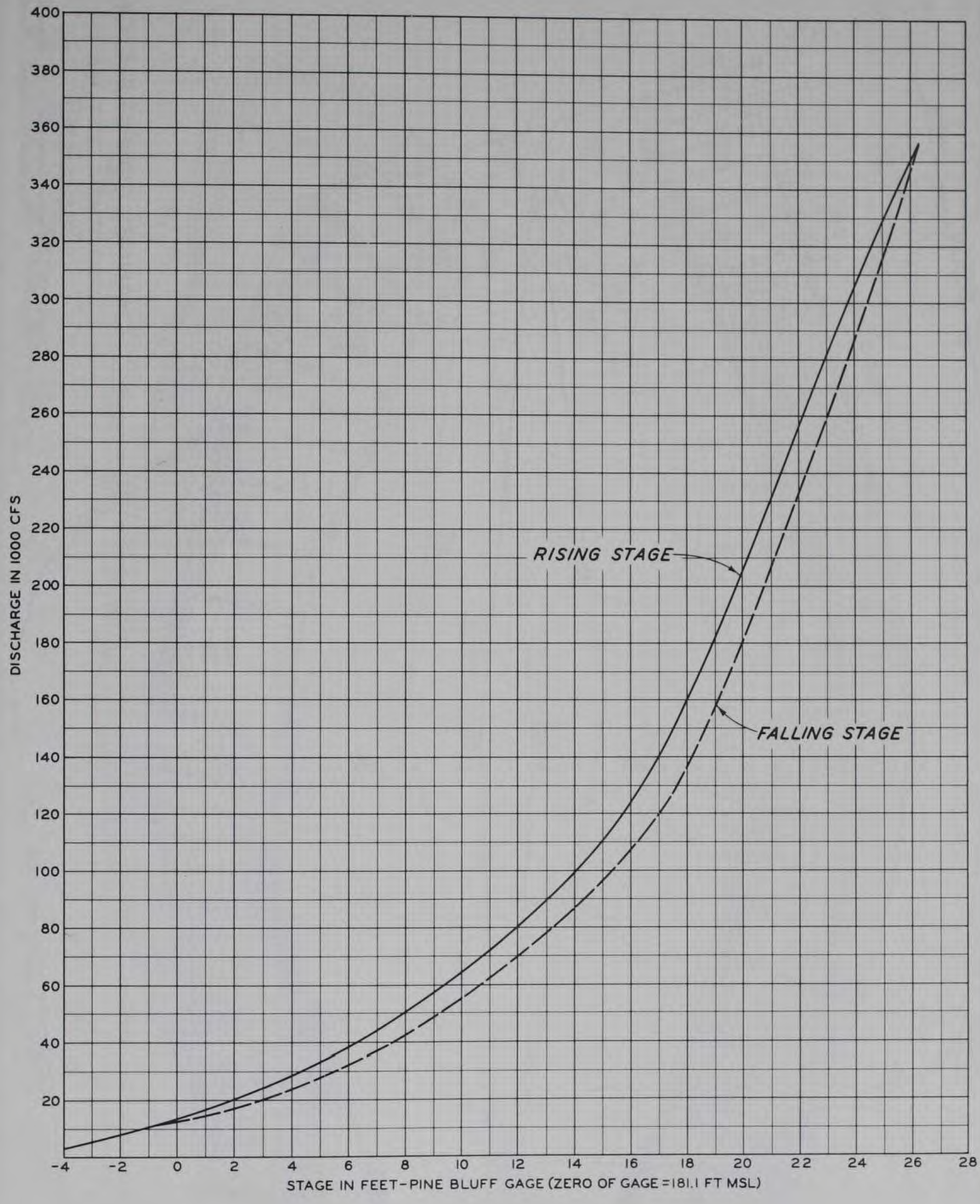
PLATE 1

PLATE 2



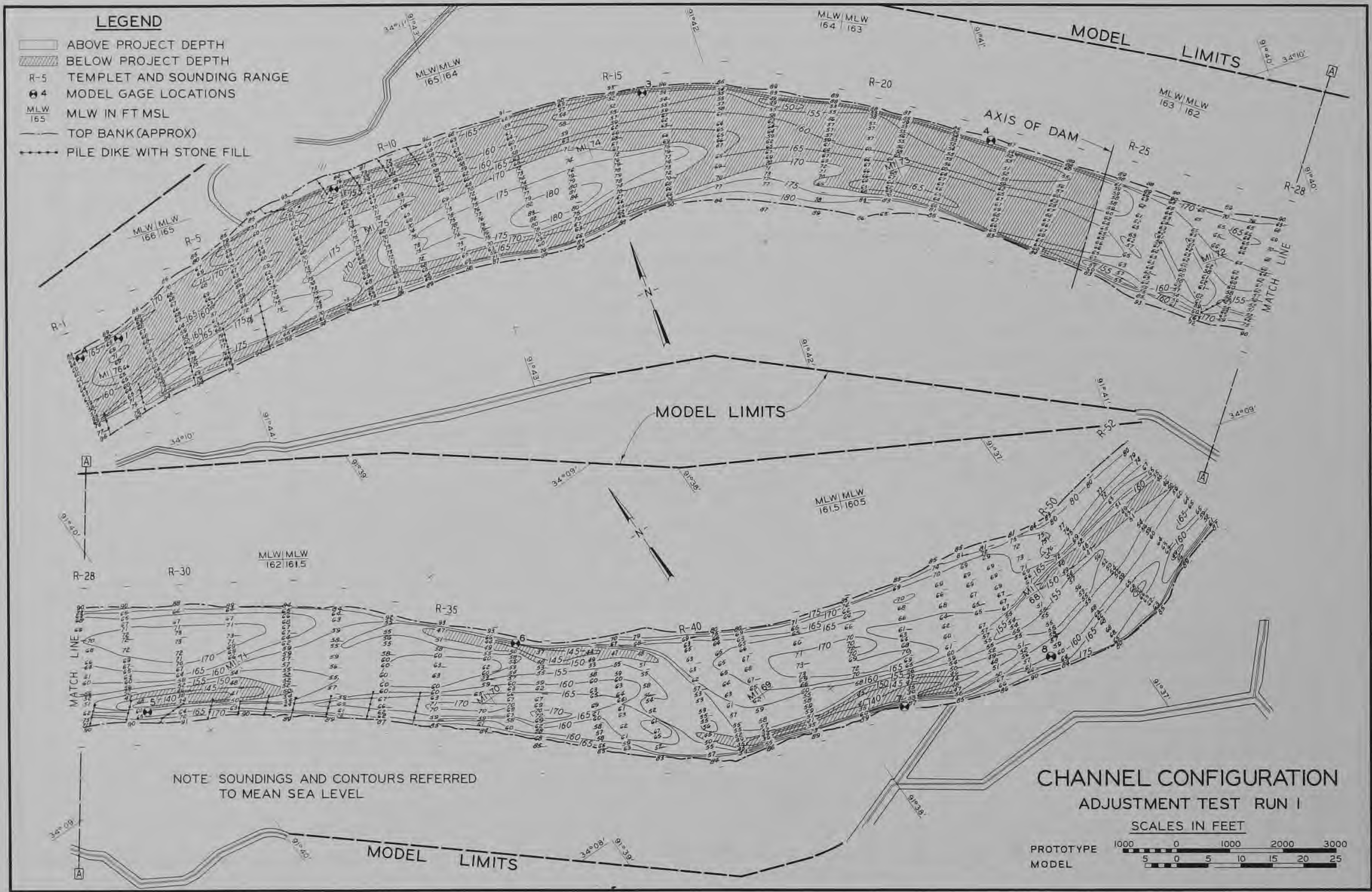
NOTE: PINE BLUFF GAGE ZERO EL 181.1 FT MSL

STAGE HYDROGRAPH



STAGE-DISCHARGE CURVE

PLATE 4


LEGEND
ABOVE PROJECT DEPTH
BELOW PROJECT DEPTH
R-5 TEMPLET AND SOUNDING RANGE
4 MODEL GAGE LOCATIONS
MLW 165 MLW IN FT MSL
TOP BANK (APPROX)
PILE DIKE WITH STONE FILL
MODEL LIMITS
AXIS OF DAM
MATCH LINE
NOTE: SOUNDINGS AND CONTOURS REFERRED TO MEAN SEA LEVEL
CHANNEL CONFIGURATION
ADJUSTMENT TEST RUN I
SCALES IN FEET
PROTOTYPE 1000 0 1000 2000 3000
MODEL 5 0 5 10 15 20 25

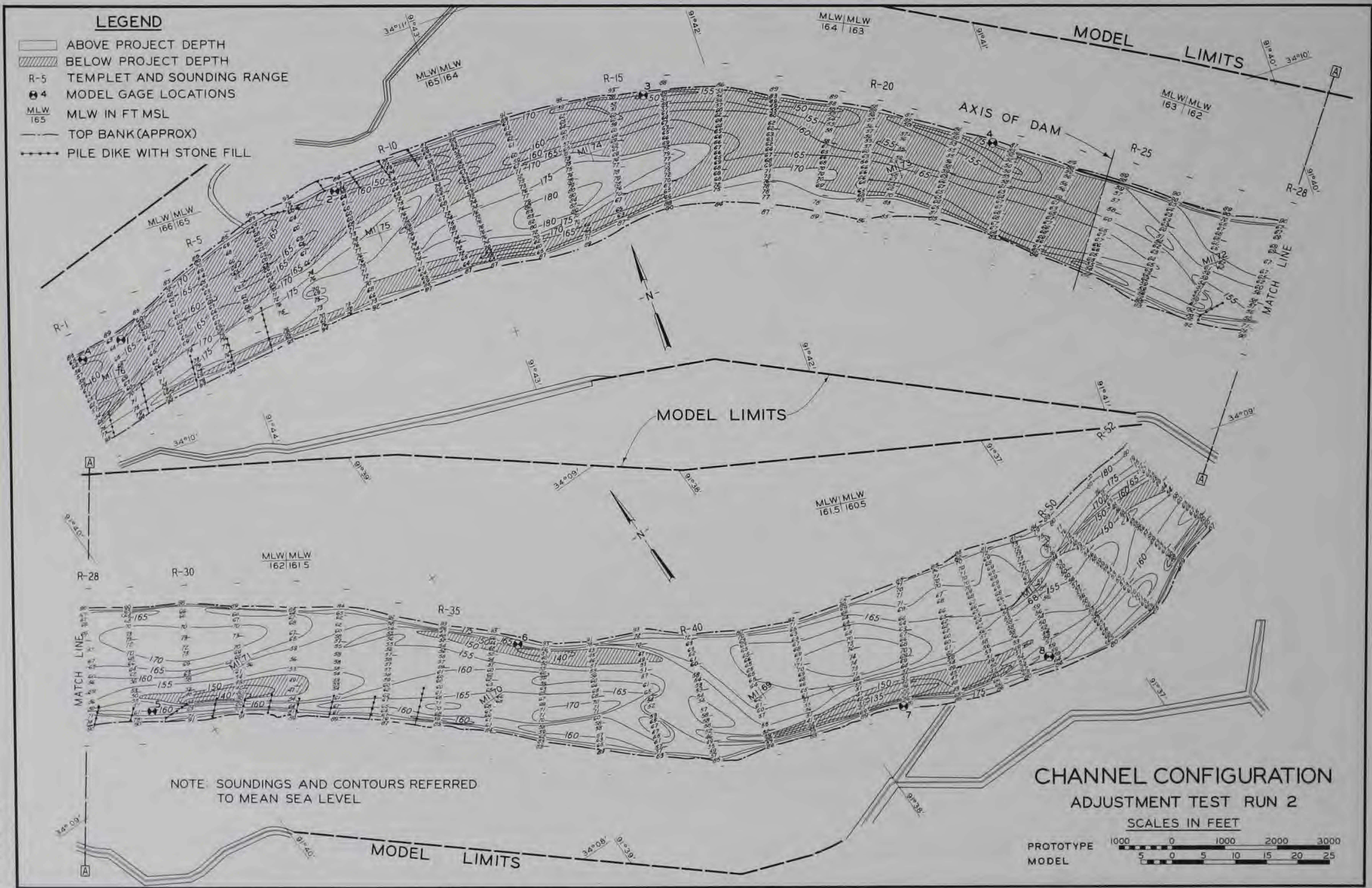
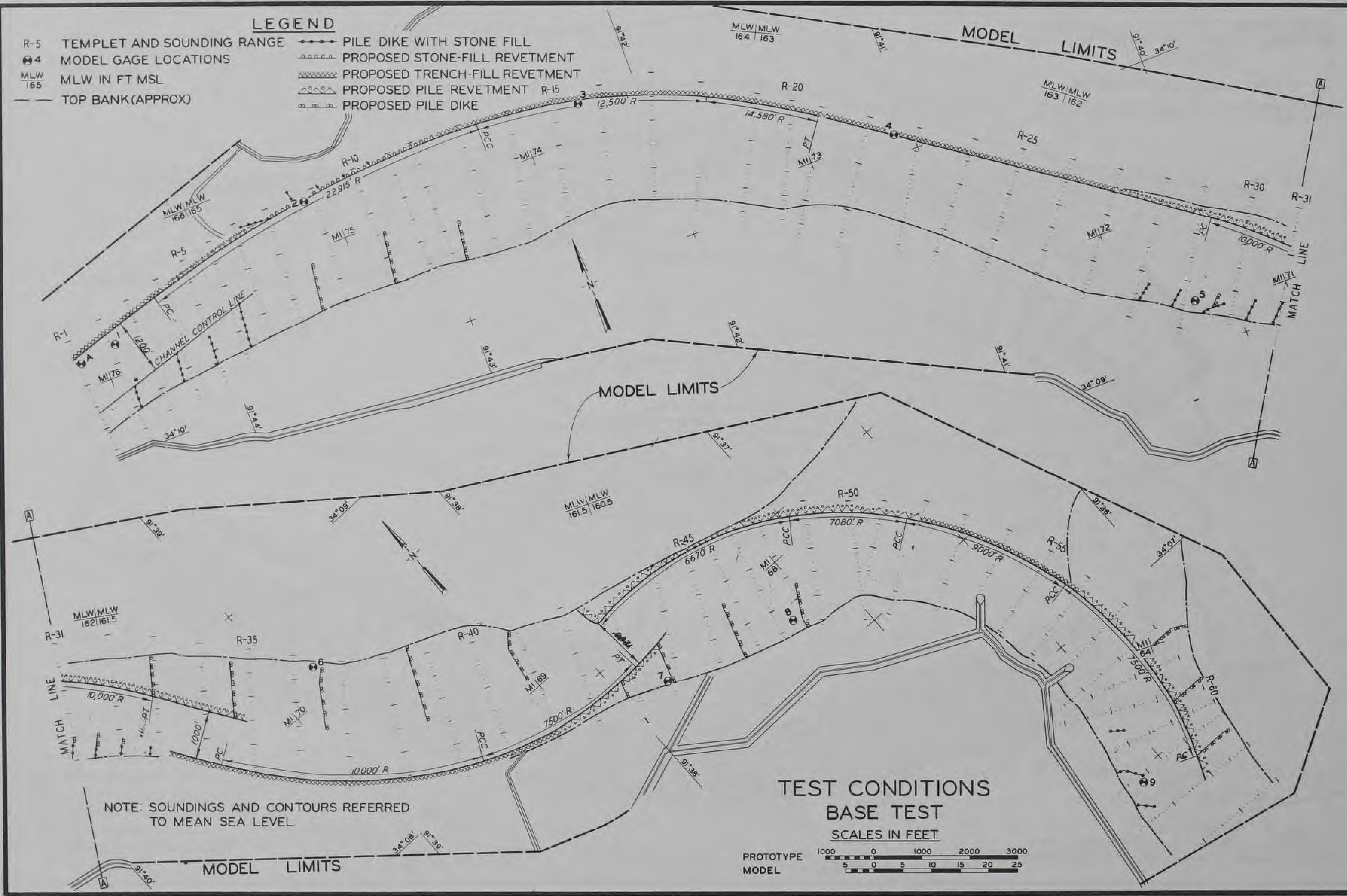

LEGEND
ABOVE PROJECT DEPTH
BELOW PROJECT DEPTH
R-5 TEMPLET AND SOUNDING RANGE
4 MODEL GAGE LOCATIONS
MLW 165 MLW IN FT MSL
TOP BANK (APPROX)
PILE DIKE WITH STONE FILL
MODEL LIMITS
AXIS OF DAM
MATCH LINE
NOTE: SOUNDINGS AND CONTOURS REFERRED TO MEAN SEA LEVEL
CHANNEL CONFIGURATION
ADJUSTMENT TEST RUN 2
SCALES IN FEET
PROTOTYPE 1000 0 1000 2000 3000
MODEL 5 0 5 10 15 20 25

PLATE 5

PLATE 6


LEGEND
R-5 TEMPLET AND SOUNDING RANGE
4 MODEL GAGE LOCATIONS
MLW / 165 MLW IN FT MSL
TOP BANK (APPROX)
PILE DIKE WITH STONE FILL
PROPOSED STONE-FILL REVETMENT
PROPOSED TRENCH-FILL REVETMENT
PROPOSED PILE REVETMENT
PROPOSED PILE DIKE
MODEL LIMITS
MODEL LIMITS
MODEL LIMITS
CHANNEL CONTROL LINE
MATCH LINE
MATCH LINE
12,500' R
14,580' R
22,915' R
10,000' R
10,000' R
7500' R
6670' R
7080' R
9000' R
7500' R
NOTE: SOUNDINGS AND CONTOURS REFERRED TO MEAN SEA LEVEL
TEST CONDITIONS
BASE TEST
SCALES IN FEET
PROTOTYPE 1000 0 1000 2000 3000
MODEL 5 0 5 10 15 20 25

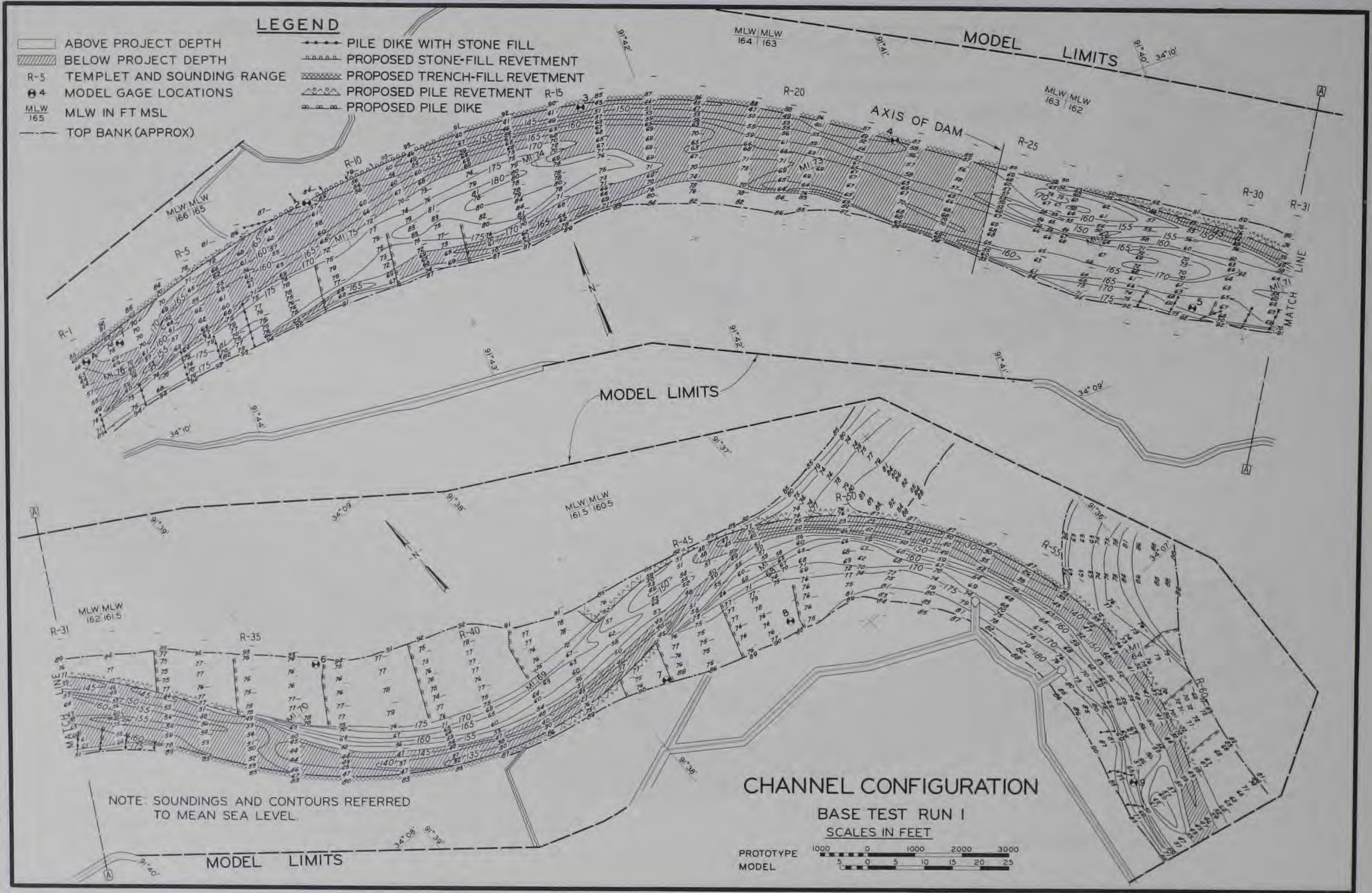
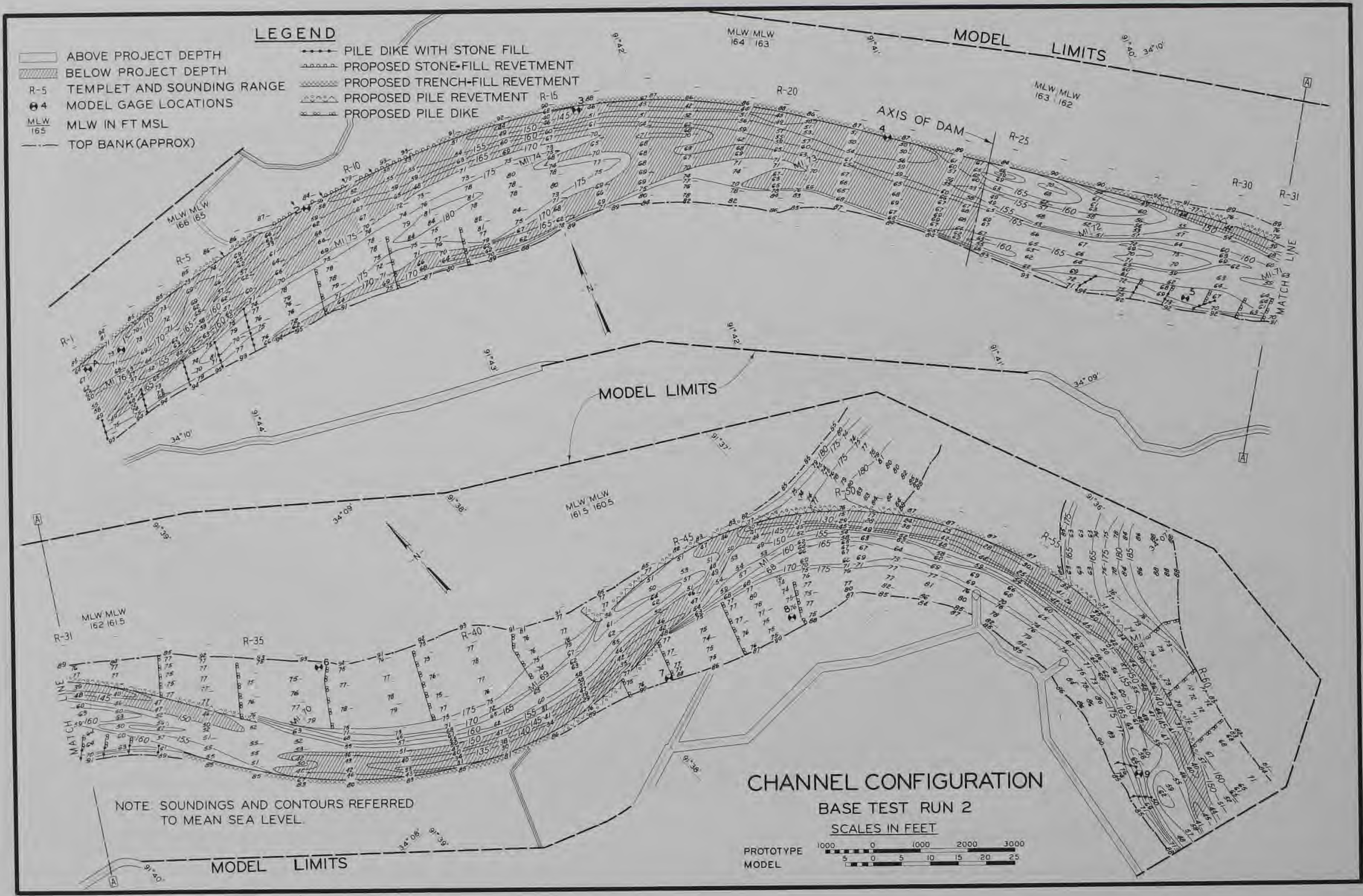

LEGEND
ABOVE PROJECT DEPTH
BELOW PROJECT DEPTH
R-5 TEMPLET AND SOUNDING RANGE
4 MODEL GAGE LOCATIONS
MLW 165 MLW IN FT MSL
TOP BANK (APPROX)
PILE DIKE WITH STONE FILL
PROPOSED STONE-FILL REVETMENT
PROPOSED TRENCH-FILL REVETMENT
PROPOSED PILE REVETMENT
PROPOSED PILE DIKE
MODEL LIMITS
AXIS OF DAM
MATCH LINE
NOTE: SOUNDINGS AND CONTOURS REFERRED TO MEAN SEA LEVEL.
CHANNEL CONFIGURATION
BASE TEST RUN 1
SCALES IN FEET
PROTOTYPE
MODEL

PLATE 7

PLATE 8


LEGEND
ABOVE PROJECT DEPTH
BELOW PROJECT DEPTH
R-5 TEMPLET AND SOUNDING RANGE
4 MODEL GAGE LOCATIONS
MLW/165 MLW IN FT MSL
TOP BANK (APPROX)
PILE DIKE WITH STONE FILL
PROPOSED STONE-FILL REVETMENT
PROPOSED TRENCH-FILL REVETMENT
PROPOSED PILE REVETMENT
PROPOSED PILE DIKE
MODEL LIMITS
AXIS OF DAM
MATCH LINE
NOTE: SOUNDINGS AND CONTOURS REFERRED TO MEAN SEA LEVEL.
CHANNEL CONFIGURATION
BASE TEST RUN 2
SCALES IN FEET
PROTOTYPE 1000 0 1000 2000 3000
MODEL 5 0 5 10 15 20 25

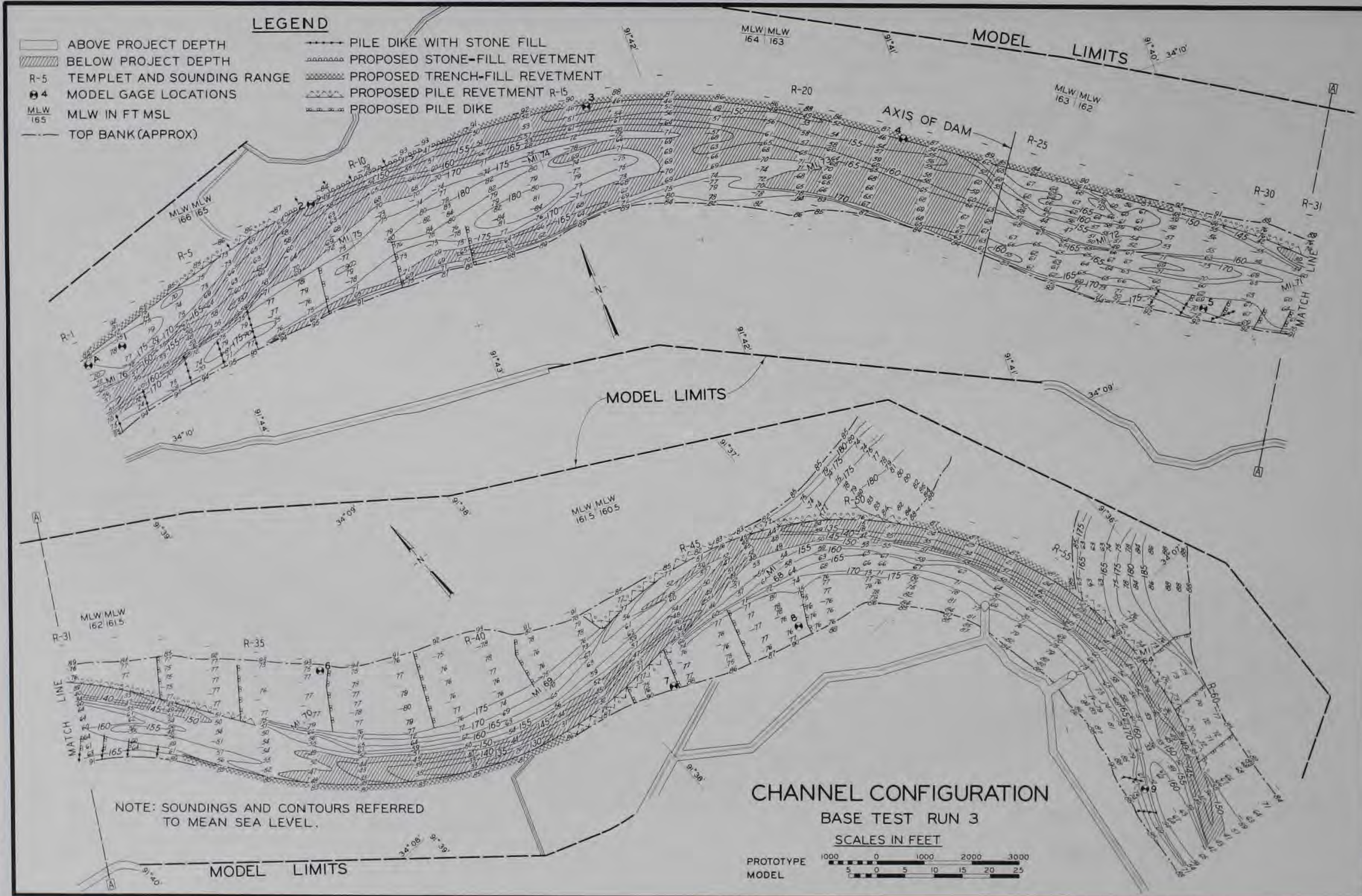
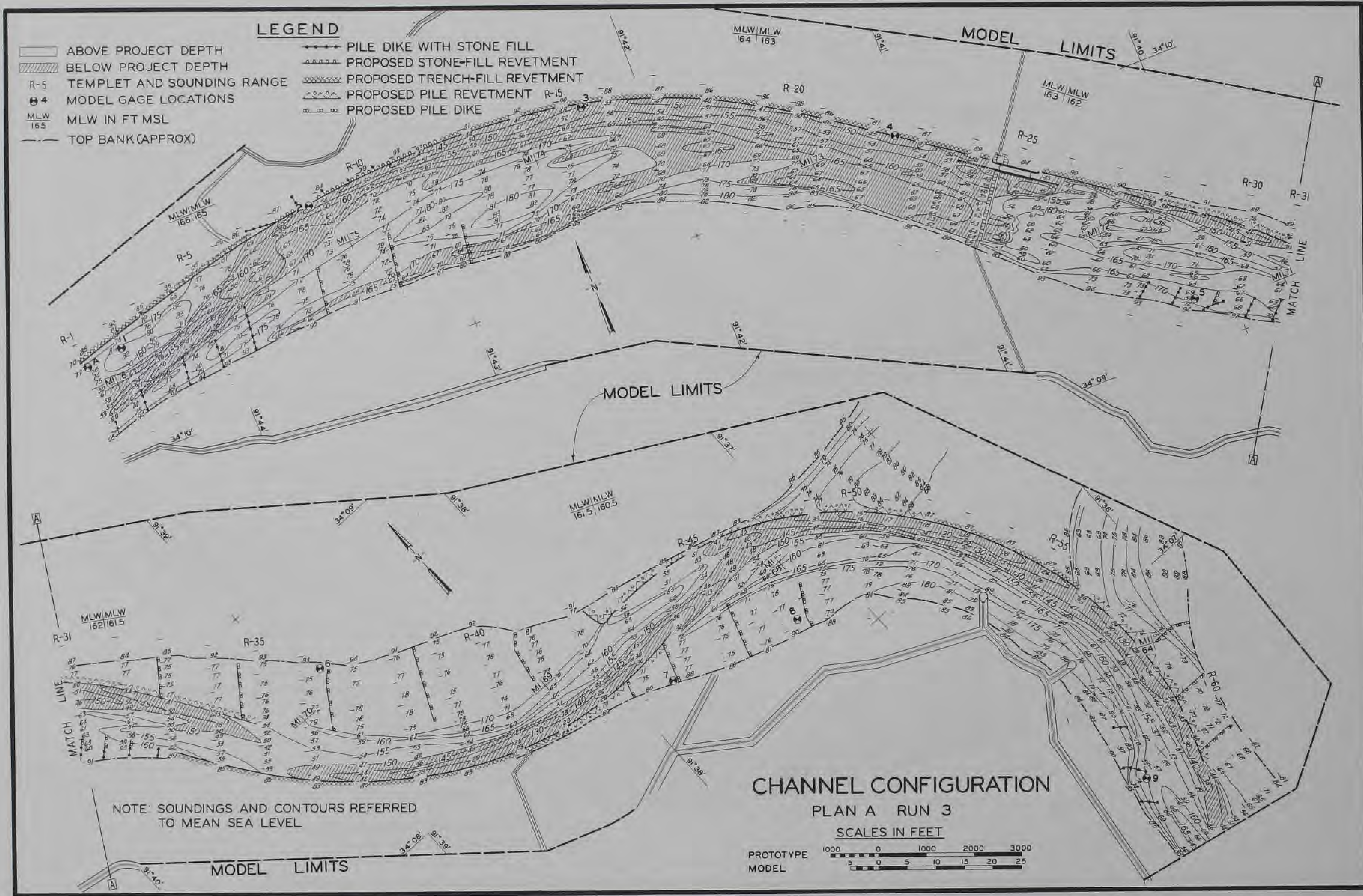

LEGEND
ABOVE PROJECT DEPTH
BELOW PROJECT DEPTH
R-5 TEMPLET AND SOUNDING RANGE
4 MODEL GAGE LOCATIONS
MLW 165 MLW IN FT MSL
TOP BANK (APPROX)
PILE DIKE WITH STONE FILL
PROPOSED STONE-FILL REVETMENT
PROPOSED TRENCH-FILL REVETMENT
PROPOSED PILE REVETMENT
PROPOSED PILE DIKE
MODEL LIMITS
AXIS OF DAM
MATCH LINE
NOTE: SOUNDINGS AND CONTOURS REFERRED TO MEAN SEA LEVEL.
CHANNEL CONFIGURATION
BASE TEST RUN 3
SCALES IN FEET
PROTOTYPE 1000 0 1000 2000 3000
MODEL 5 0 5 10 15 20 25

PLATE 9

PLATE 10


LEGEND
ABOVE PROJECT DEPTH
BELOW PROJECT DEPTH
R-5 TEMPLET AND SOUNDING RANGE
4 MODEL GAGE LOCATIONS
MLW / 165 MLW IN FT MSL
TOP BANK (APPROX)
PILE DIKE WITH STONE FILL
PROPOSED STONE-FILL REVETMENT
PROPOSED TRENCH-FILL REVETMENT
PROPOSED PILE REVETMENT
PROPOSED PILE DIKE
MODEL LIMITS
MODEL LIMITS
MODEL LIMITS
MATCH LINE
MATCH LINE
NOTE: SOUNDINGS AND CONTOURS REFERRED TO MEAN SEA LEVEL
CHANNEL CONFIGURATION
PLAN A RUN 3
SCALES IN FEET
PROTOTYPE 1000 0 1000 2000 3000
MODEL 5 0 5 10 15 20 25

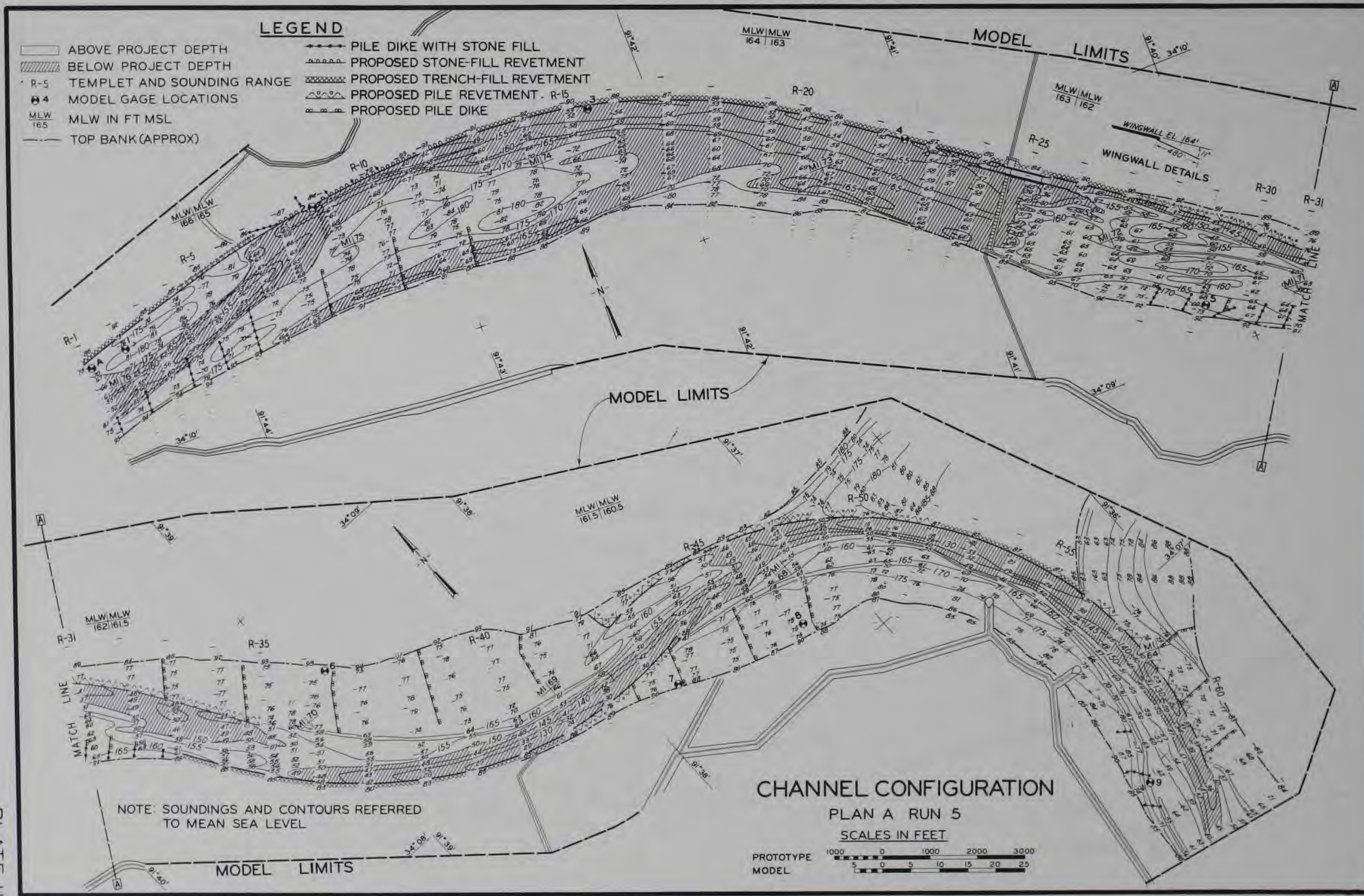
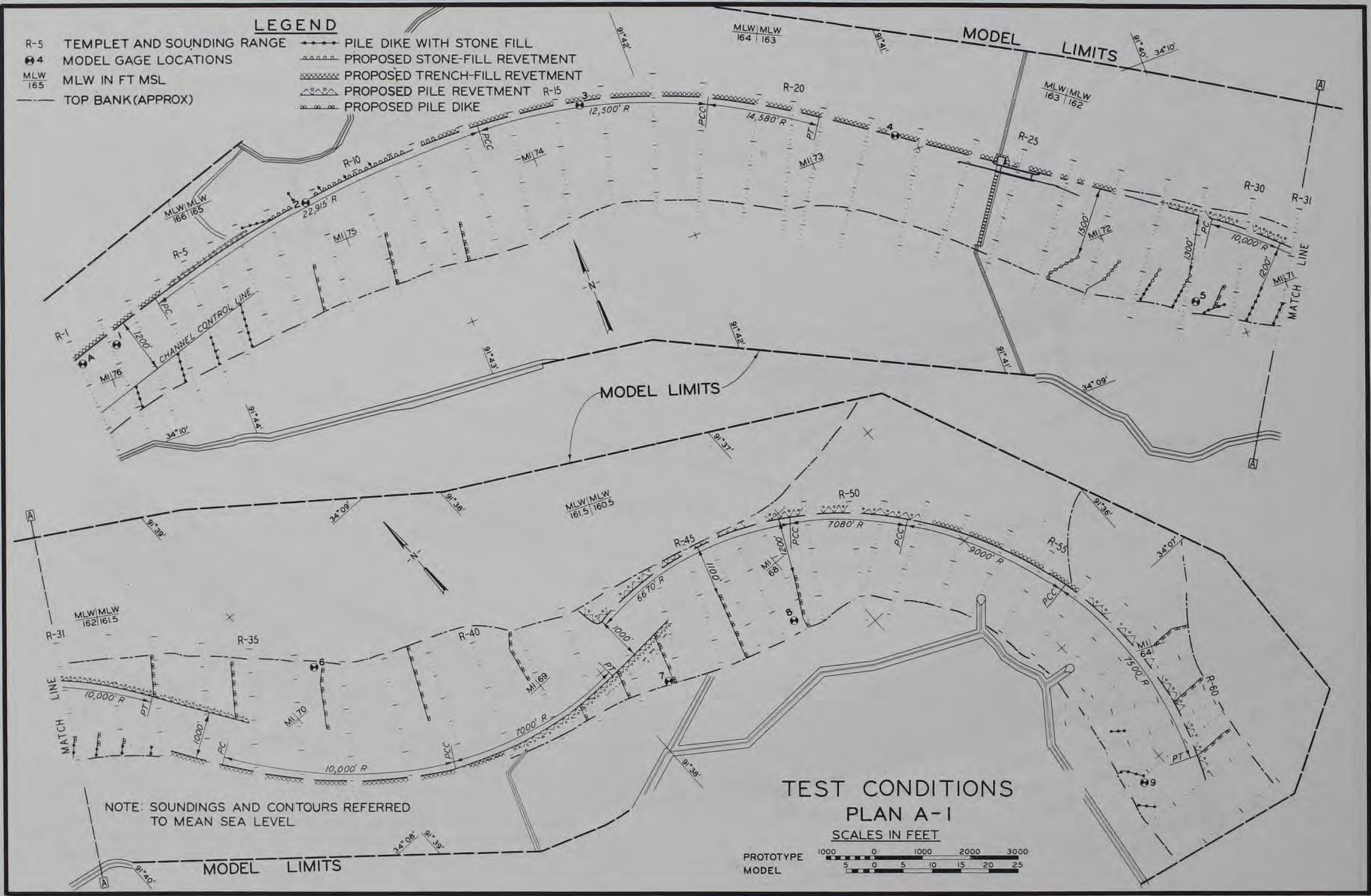
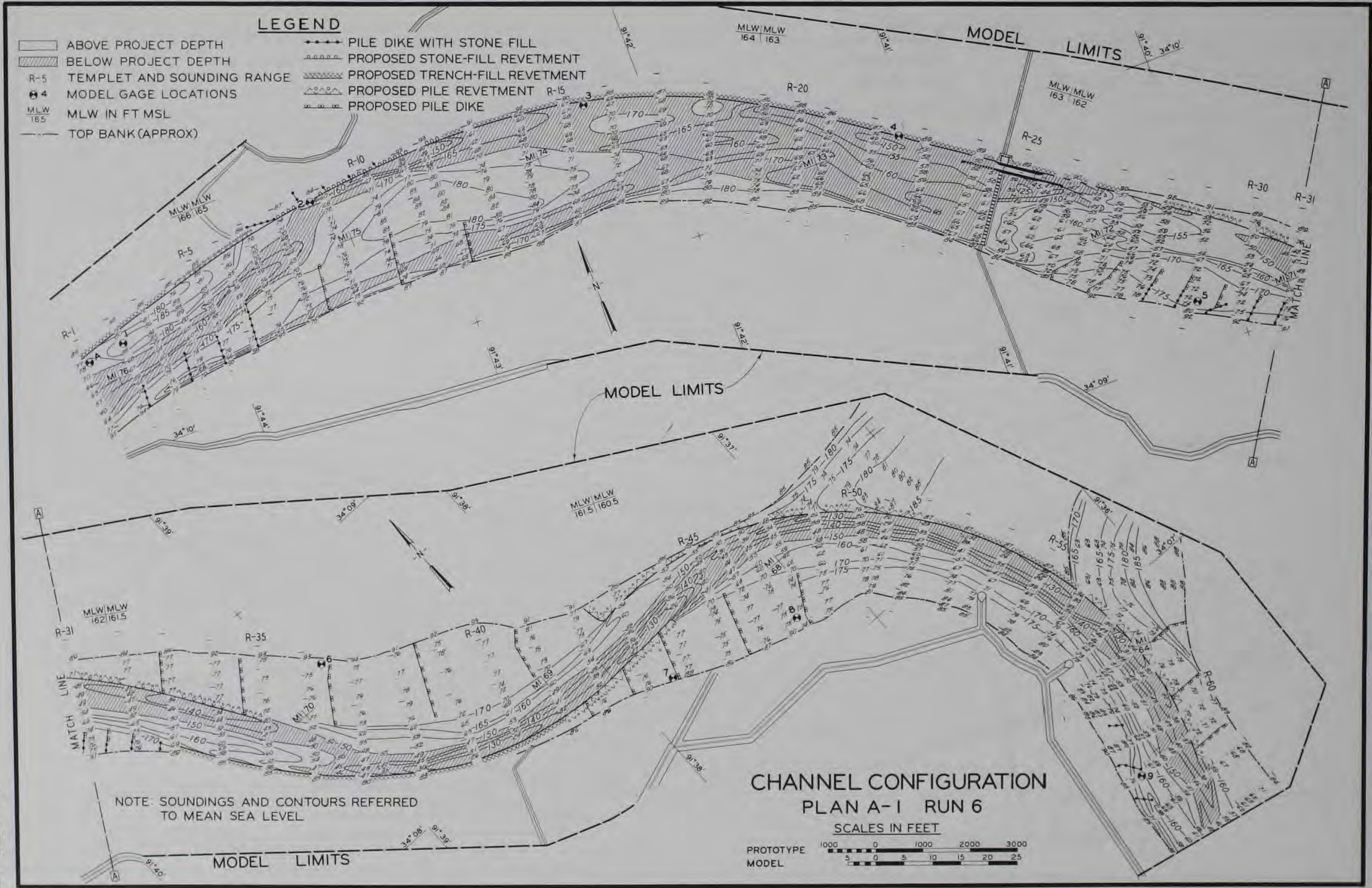
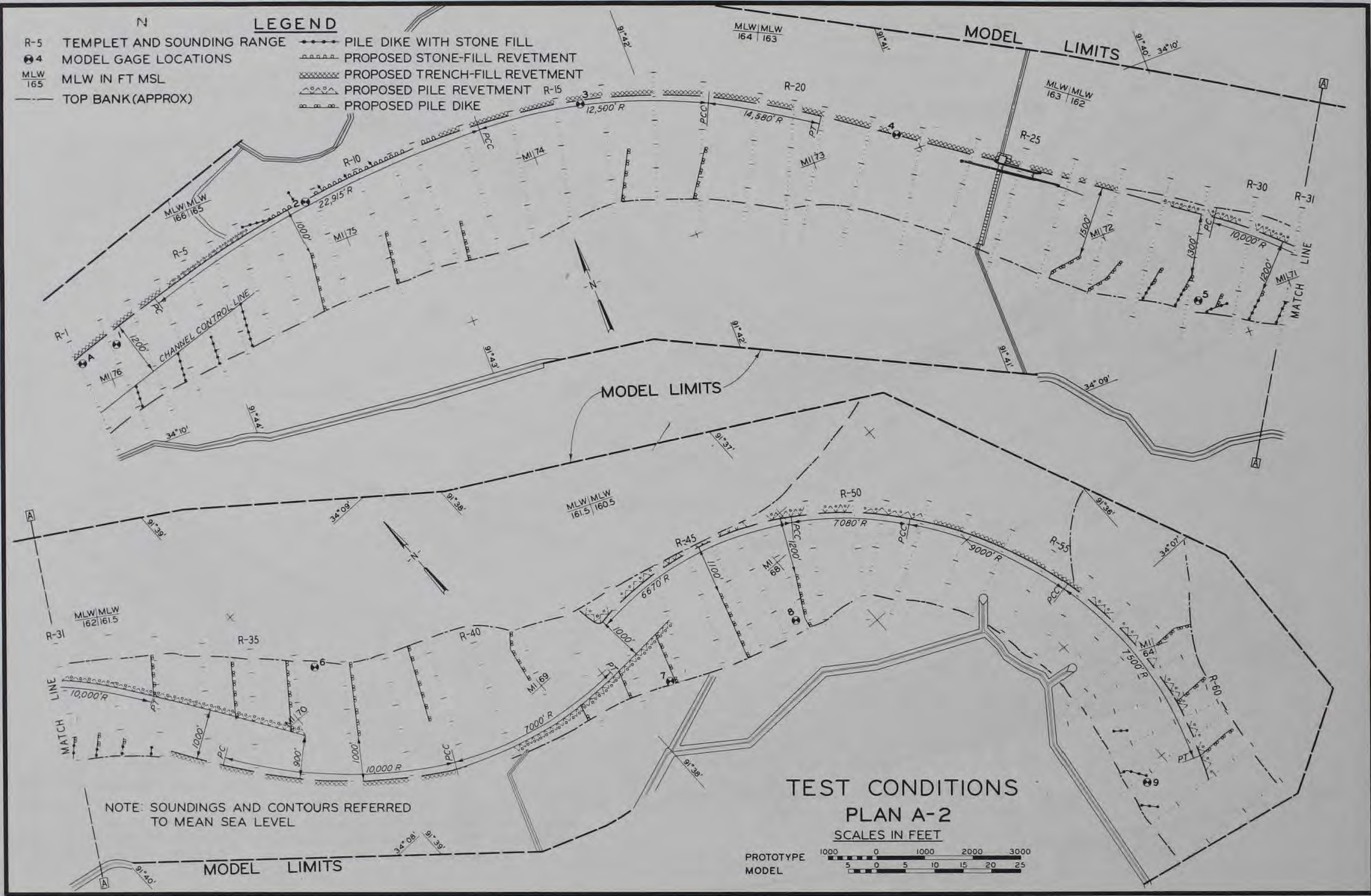

LEGEND
ABOVE PROJECT DEPTH
BELOW PROJECT DEPTH
R-5 TEMPLET AND SOUNDING RANGE
4 MODEL GAGE LOCATIONS
MLW 165 MLW IN FT MSL
TOP BANK (APPROX)
PILE DIKE WITH STONE FILL
PROPOSED STONE-FILL REVETMENT
PROPOSED TRENCH-FILL REVETMENT
PROPOSED PILE REVETMENT
PROPOSED PILE DIKE
MODEL LIMITS
WINGWALL EL 164'
480'
WINGWALL DETAILS
MATCH LINE
NOTE: SOUNDINGS AND CONTOURS REFERRED TO MEAN SEA LEVEL
CHANNEL CONFIGURATION
PLAN A RUN 5
SCALES IN FEET
PROTOTYPE 1000 0 1000 2000 3000
MODEL 5 0 5 10 15 20 25

PLATE 11



TEST CONDITIONS
PLAN A-1


LEGEND
ABOVE PROJECT DEPTH
BELOW PROJECT DEPTH
R-5 TEMPLET AND SOUNDING RANGE
4 MODEL GAGE LOCATIONS
MLW 165 MLW IN FT MSL
TOP BANK (APPROX)
PILE DIKE WITH STONE FILL
PROPOSED STONE-FILL REVETMENT
PROPOSED TRENCH-FILL REVETMENT
PROPOSED PILE REVETMENT
PROPOSED PILE DIKE
MODEL LIMITS
MODEL LIMITS
MODEL LIMITS
MODEL LIMITS
MATCH LINE
MATCH LINE
N
N
NOTE: SOUNDINGS AND CONTOURS REFERRED TO MEAN SEA LEVEL
CHANNEL CONFIGURATION
PLAN A-1 RUN 6
SCALES IN FEET
PROTOTYPE 1000 0 1000 2000 3000
MODEL 5 0 5 10 15 20 25


LEGEND
R-5 TEMPLET AND SOUNDING RANGE
4 MODEL GAGE LOCATIONS
MLW/165 MLW IN FT MSL
TOP BANK (APPROX)
PILE DIKE WITH STONE FILL
PROPOSED STONE-FILL REVETMENT
PROPOSED TRENCH-FILL REVETMENT
PROPOSED PILE REVETMENT
PROPOSED PILE DIKE
MODEL LIMITS
CHANNEL CONTROL LINE
MATCH LINE
NOTE: SOUNDINGS AND CONTOURS REFERRED
TO MEAN SEA LEVEL
TEST CONDITIONS
PLAN A-2
SCALES IN FEET
PROTOTYPE 1000 0 1000 2000 3000
MODEL 5 0 5 10 15 20 25

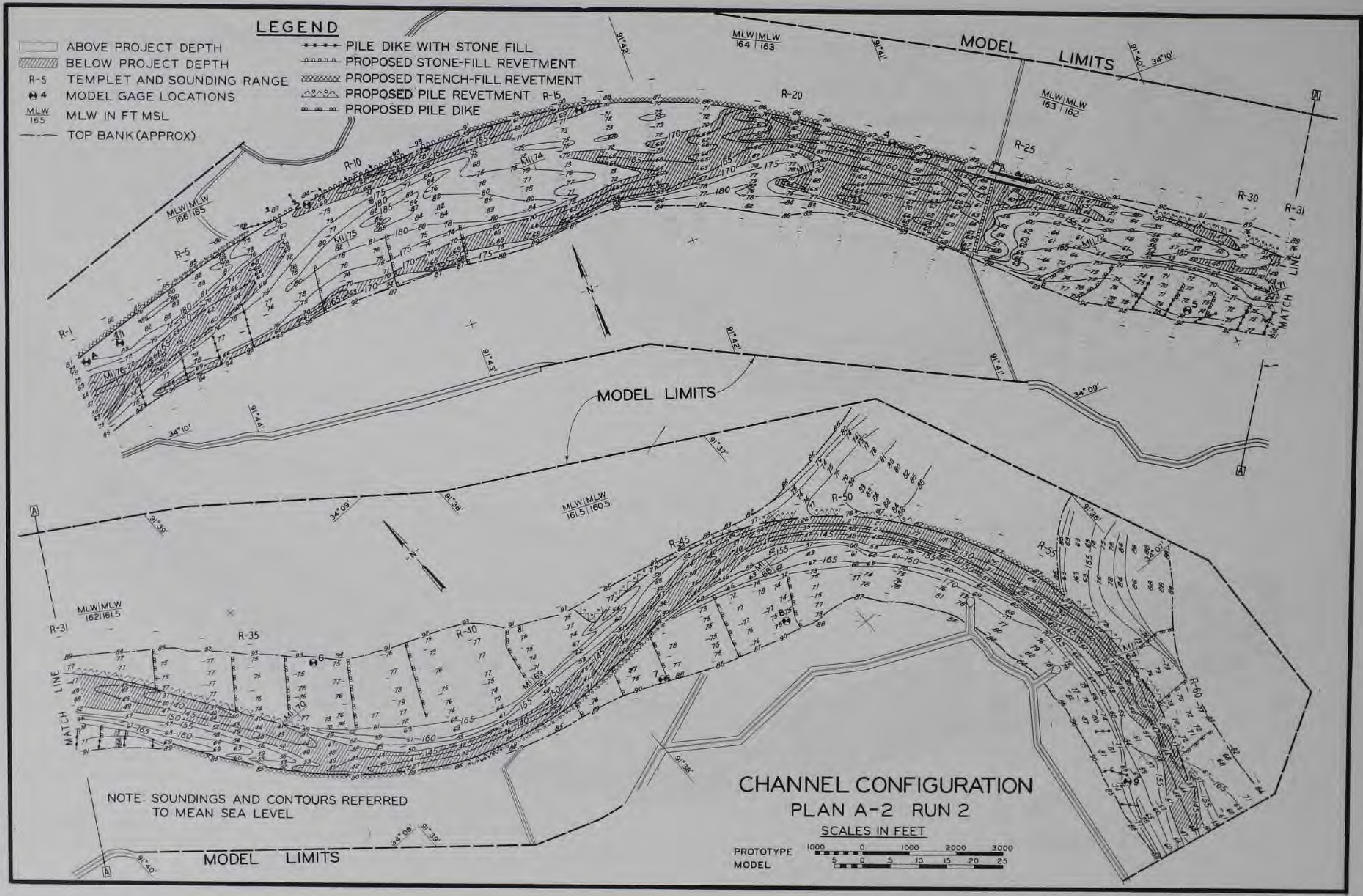
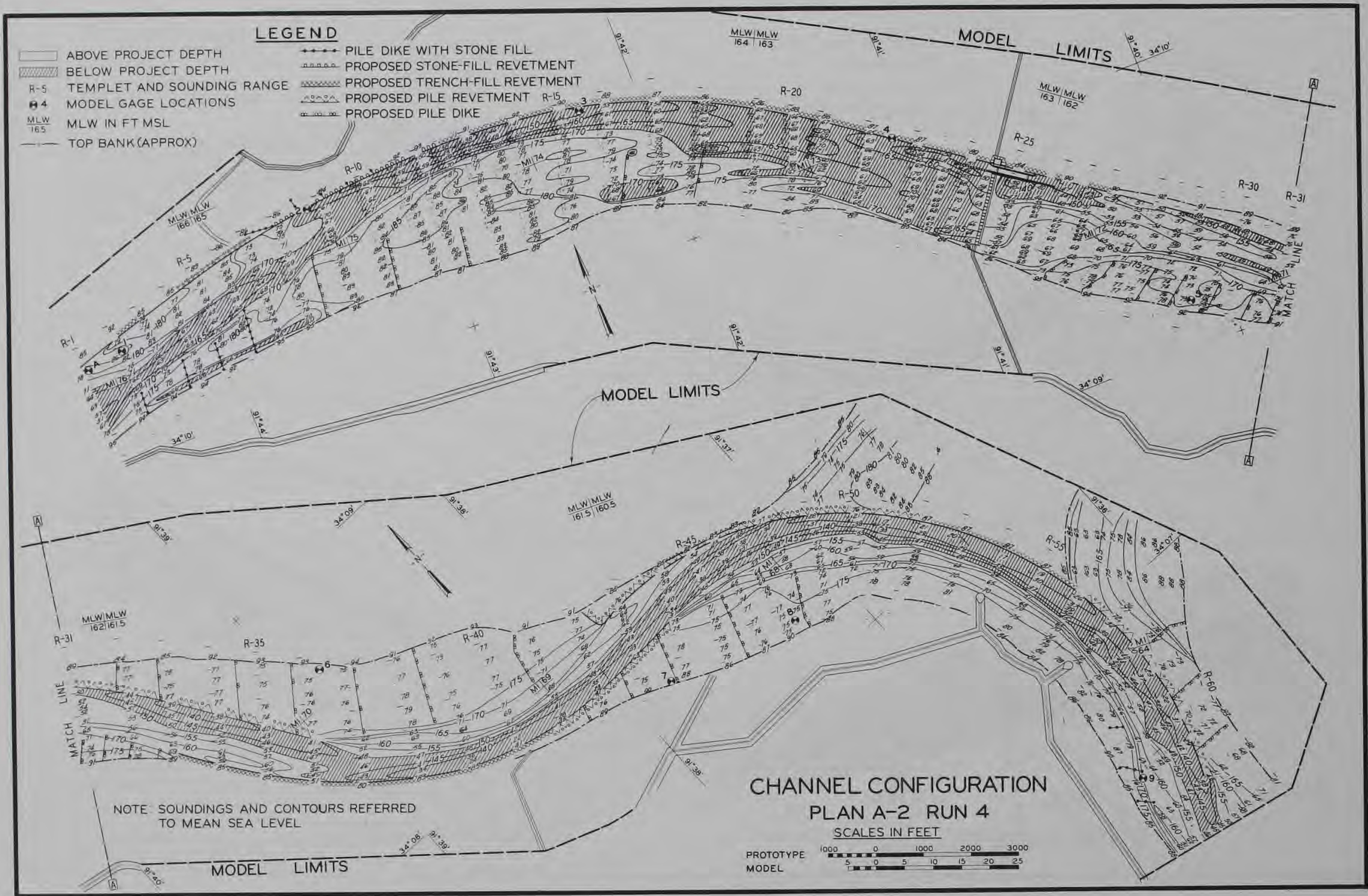

LEGEND
ABOVE PROJECT DEPTH
BELOW PROJECT DEPTH
R-5 TEMPLET AND SOUNDING RANGE
4 MODEL GAGE LOCATIONS
MLW / 165 MLW IN FT MSL
TOP BANK (APPROX)
PILE DIKE WITH STONE FILL
PROPOSED STONE-FILL REVETMENT
PROPOSED TRENCH-FILL REVETMENT
PROPOSED PILE REVETMENT
PROPOSED PILE DIKE
MODEL LIMITS
MODEL LIMITS
MODEL LIMITS
MATCH LINE
MATCH LINE
N
MI 76
MI 75
MI 74
MI 73
MI 72
MI 71
MI 70
MI 69
MI 68
MI 67
R-1
R-5
R-10
R-15
R-20
R-25
R-30
R-31
R-35
R-40
R-45
R-50
R-55
R-60
NOTE: SOUNDINGS AND CONTOURS REFERRED TO MEAN SEA LEVEL
CHANNEL CONFIGURATION
PLAN A-2 RUN 2
SCALES IN FEET
PROTOTYPE 1000 0 1000 2000 3000
MODEL 5 0 5 10 15 20 25

PLATE 15

PLATE 16



LEGEND
ABOVE PROJECT DEPTH
BELOW PROJECT DEPTH
R-5 TEMPLET AND SOUNDING RANGE
4 MODEL GAGE LOCATIONS
MLW 165 MLW IN FT MSL
TOP BANK (APPROX)
PILE DIKE WITH STONE FILL
PROPOSED STONE-FILL REVETMENT
PROPOSED TRENCH-FILL REVETMENT
PROPOSED PILE REVETMENT
PROPOSED PILE DIKE
MODEL LIMITS
MODEL LIMITS
MODEL LIMITS
MATCH LINE
MATCH LINE
R-1
R-5
R-10
R-15
R-20
R-25
R-30
R-31
R-31
R-35
R-40
R-45
R-50
R-55
R-60
MLW 166 MLW 165
MLW 164 MLW 163
MLW 163 MLW 162
MLW 162 MLW 161.5
MLW 161.5 MLW 160.5
34° 10'
34° 09'
34° 08'
34° 07'
91° 44'
91° 43'
91° 42'
91° 41'
91° 40'
91° 39'
91° 38'
91° 37'
91° 36'
N
NOTE: SOUNDINGS AND CONTOURS REFERRED TO MEAN SEA LEVEL
CHANNEL CONFIGURATION
PLAN A-2 RUN 4
SCALES IN FEET
PROTOTYPE 1000 0 1000 2000 3000
MODEL 5 0 5 10 15 20 25

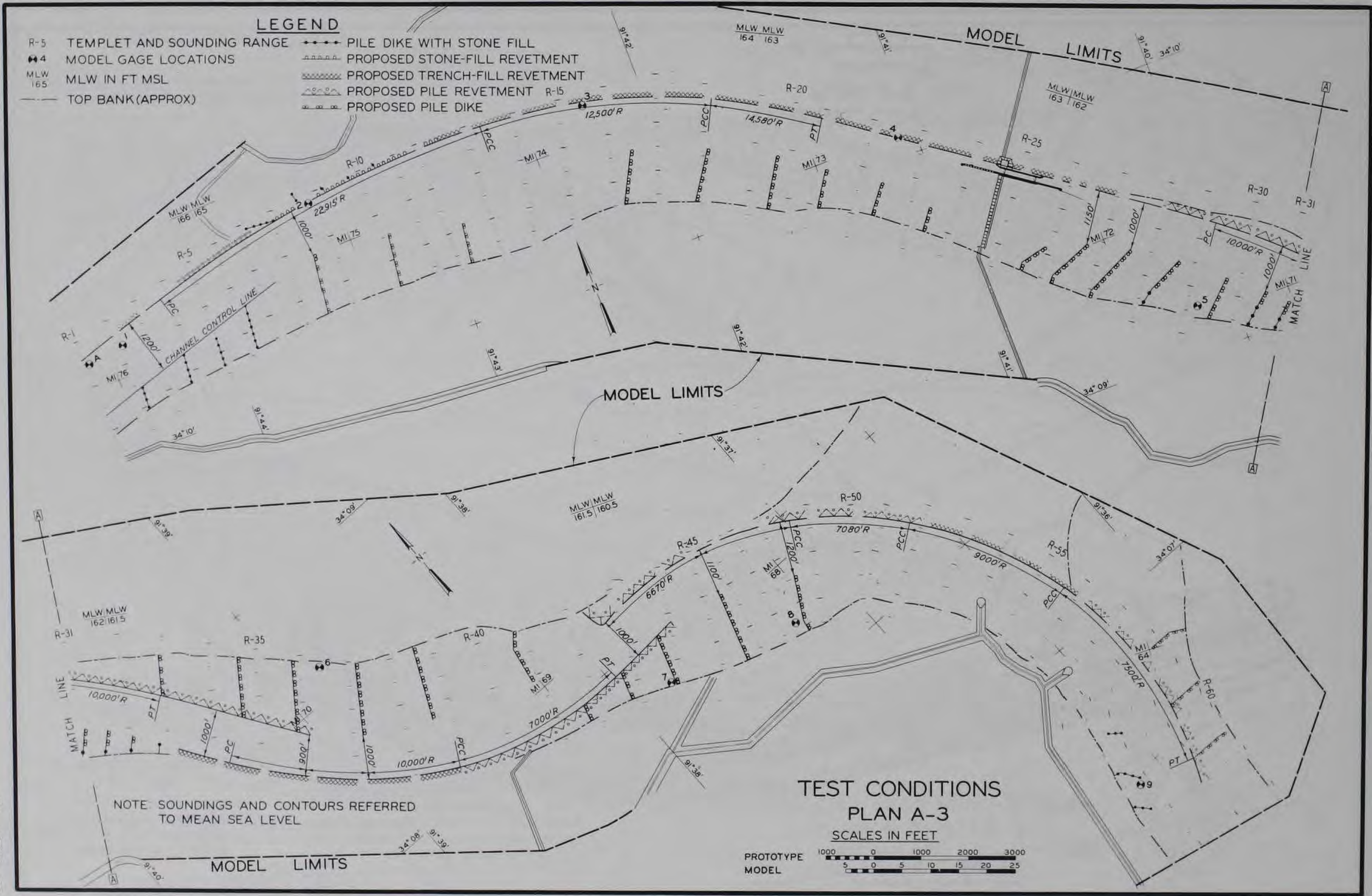
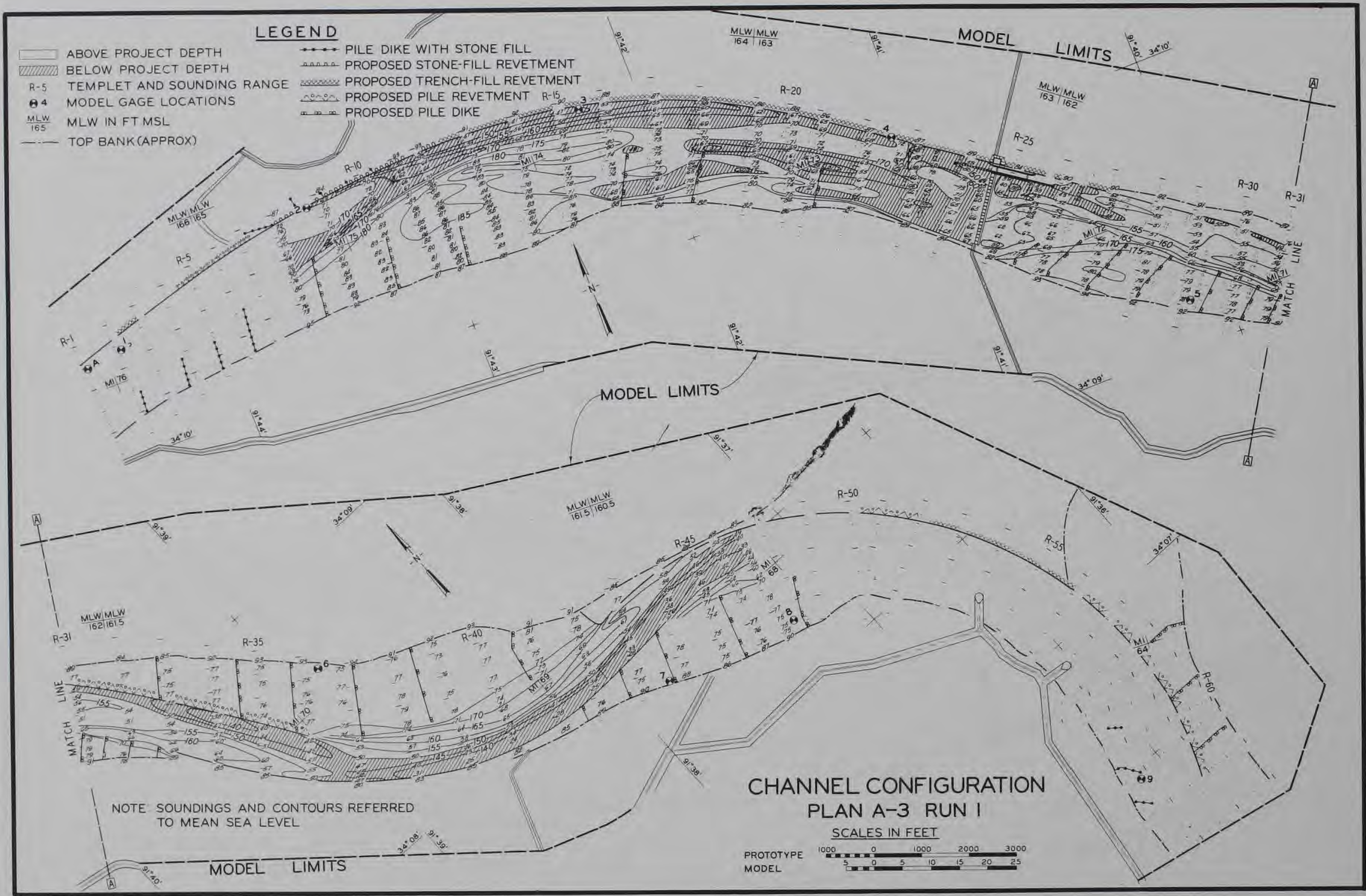
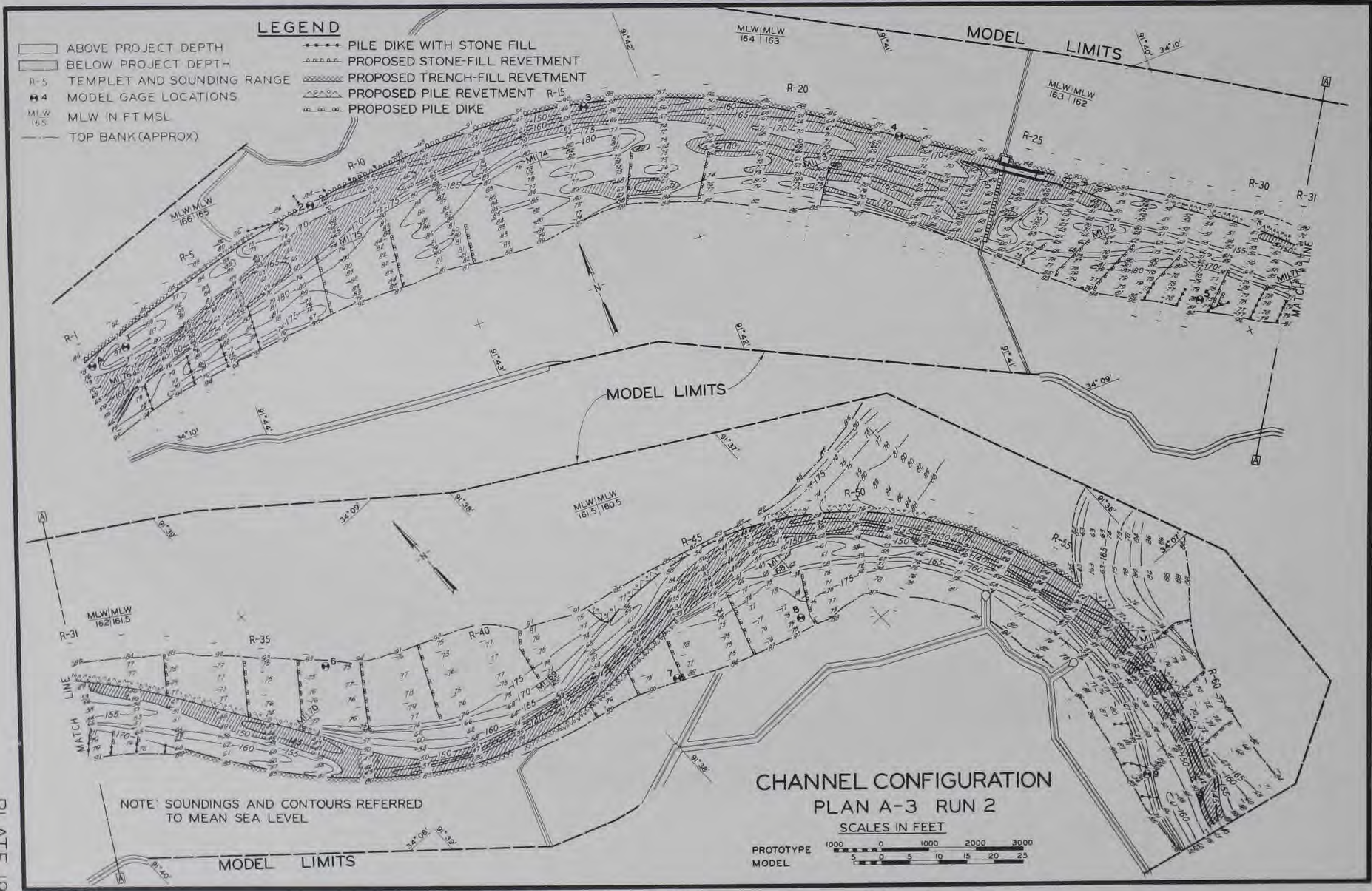

LEGEND
R-5 TEMPLET AND SOUNDING RANGE
4 MODEL GAGE LOCATIONS
MLW 165 MLW IN FT MSL
TOP BANK (APPROX)
PILE DIKE WITH STONE FILL
PROPOSED STONE-FILL REVETMENT
PROPOSED TRENCH-FILL REVETMENT
PROPOSED PILE REVETMENT
PROPOSED PILE DIKE
MODEL LIMITS
MODEL LIMITS
MODEL LIMITS
CHANNEL CONTROL LINE
MATCH LINE
MATCH LINE
NOTE: SOUNDINGS AND CONTOURS REFERRED TO MEAN SEA LEVEL
TEST CONDITIONS
PLAN A-3
SCALES IN FEET
PROTOTYPE
MODEL

PLATE 18



CHANNEL CONFIGURATION
PLAN A-3 RUN I

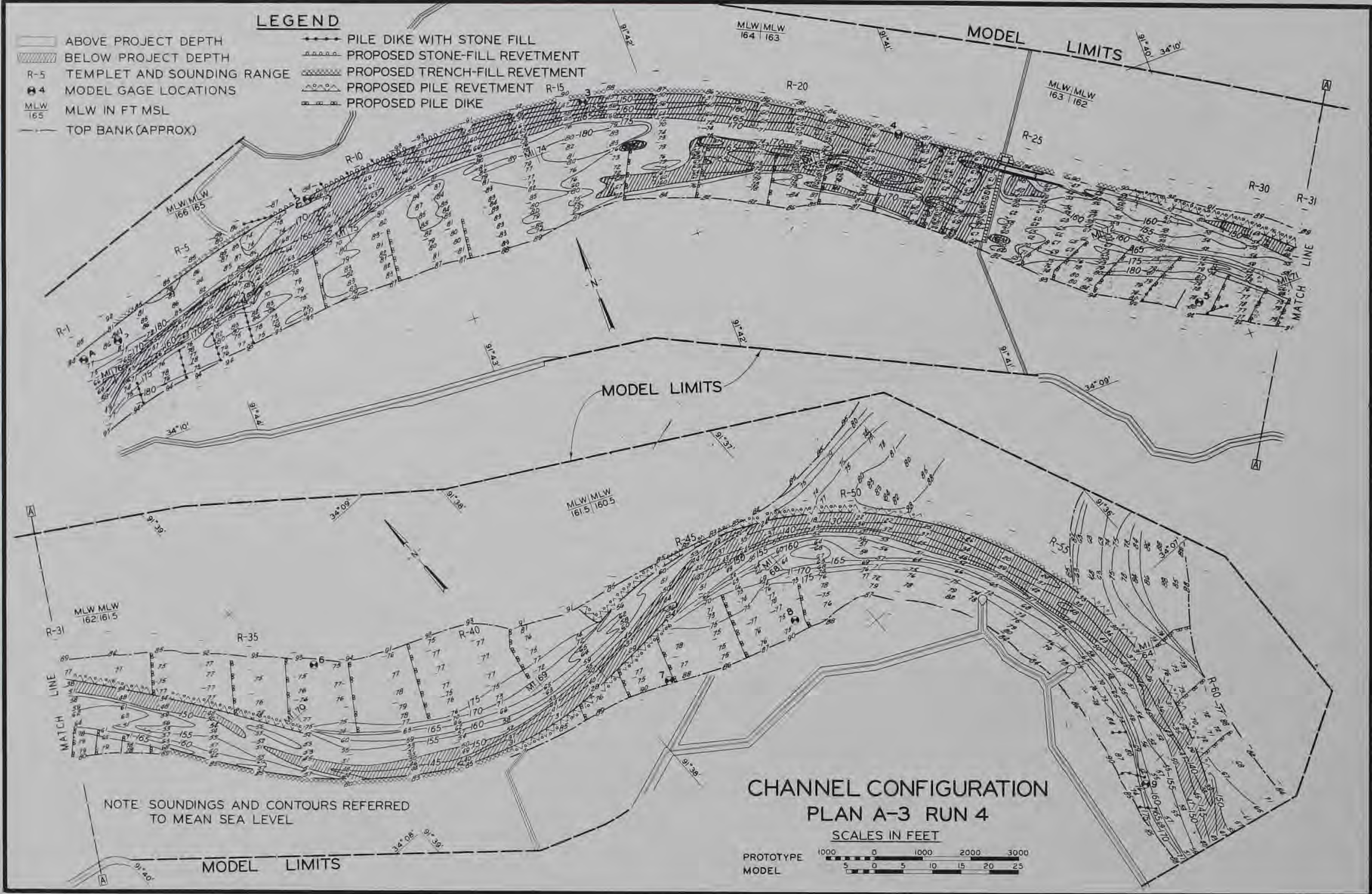
SCALES IN FEET



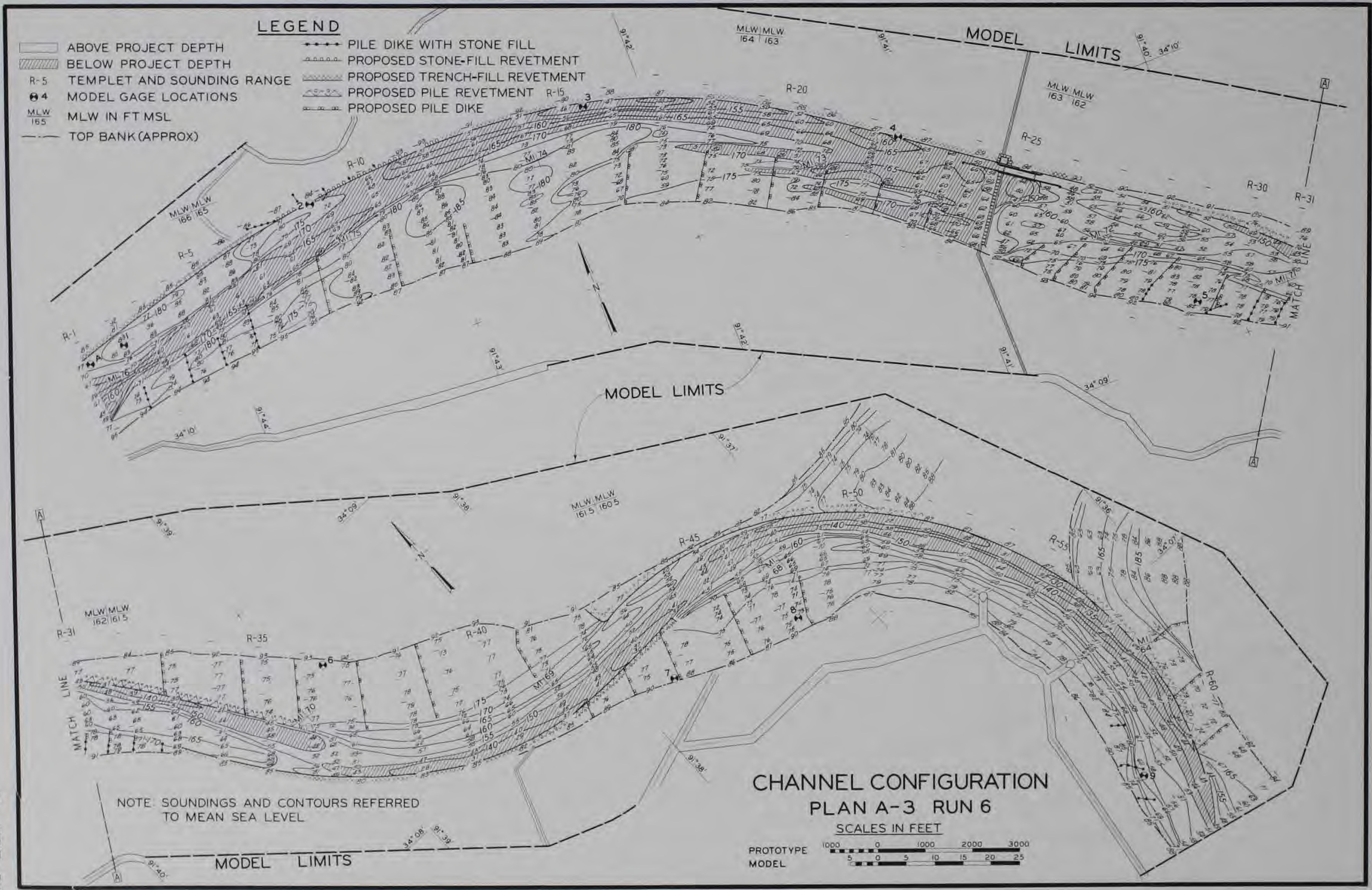
CHANNEL CONFIGURATION
PLAN A-3 RUN 2

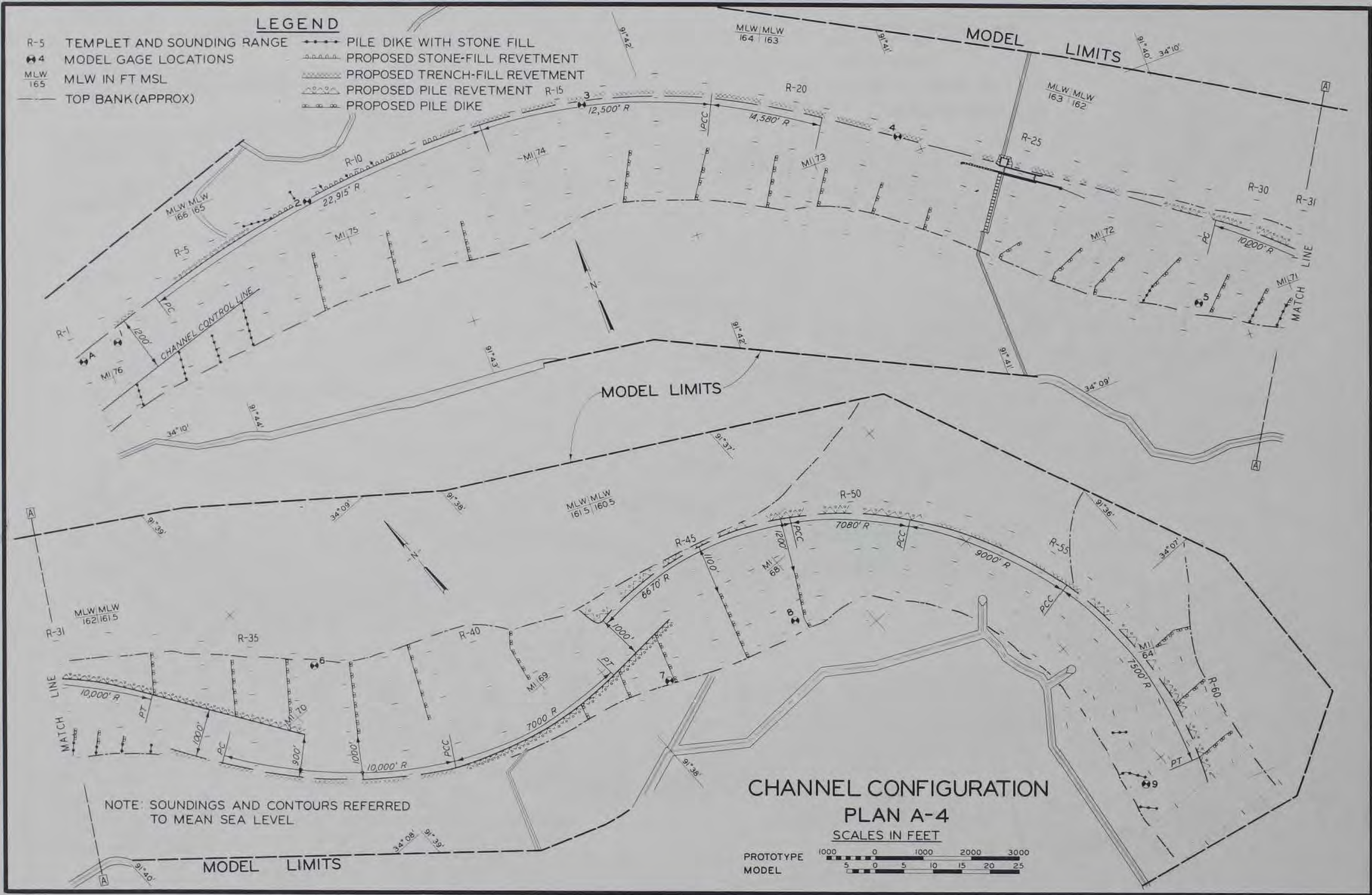
PLATE 19

PLATE 20



LEGEND
ABOVE PROJECT DEPTH
BELOW PROJECT DEPTH
R-5 TEMPLET AND SOUNDING RANGE
4 MODEL GAGE LOCATIONS
MLW / 165 MLW IN FT MSL
TOP BANK (APPROX)
PILE DIKE WITH STONE FILL
PROPOSED STONE-FILL REVETMENT
PROPOSED TRENCH-FILL REVETMENT
PROPOSED PILE REVETMENT
PROPOSED PILE DIKE
MODEL LIMITS
MATCH LINE
NOTE: SOUNDINGS AND CONTOURS REFERRED TO MEAN SEA LEVEL
CHANNEL CONFIGURATION
PLAN A-3 RUN 4
SCALES IN FEET
PROTOTYPE 1000 0 1000 2000 3000
MODEL 5 0 5 10 15 20 25


LEGEND
ABOVE PROJECT DEPTH
BELOW PROJECT DEPTH
R-5 TEMPLET AND SOUNDING RANGE
4 MODEL GAGE LOCATIONS
MLW/165 MLW IN FT MSL
TOP BANK (APPROX)
PILE DIKE WITH STONE FILL
PROPOSED STONE-FILL REVETMENT
PROPOSED TRENCH-FILL REVETMENT
PROPOSED PILE REVETMENT
PROPOSED PILE DIKE
MODEL LIMITS
MODEL LIMITS
MODEL LIMITS
MATCH LINE
MATCH LINE
NOTE: SOUNDINGS AND CONTOURS REFERRED TO MEAN SEA LEVEL
CHANNEL CONFIGURATION
PLAN A-3 RUN 6
SCALES IN FEET
PROTOTYPE 1000 0 1000 2000 3000
MODEL 5 0 5 10 15 20 25


LEGEND
R-5 TEMPLET AND SOUNDING RANGE
4 MODEL GAGE LOCATIONS
MLW/165 MLW IN FT MSL
TOP BANK (APPROX)
PILE DIKE WITH STONE FILL
PROPOSED STONE-FILL REVETMENT
PROPOSED TRENCH-FILL REVETMENT
PROPOSED PILE REVETMENT
PROPOSED PILE DIKE
MODEL LIMITS
MATCH LINE
CHANNEL CONTROL LINE
NOTE: SOUNDINGS AND CONTOURS REFERRED TO MEAN SEA LEVEL
CHANNEL CONFIGURATION
PLAN A-4
SCALES IN FEET
PROTOTYPE 1000 0 1000 2000 3000
MODEL 5 0 5 10 15 20 25
PLATE 22

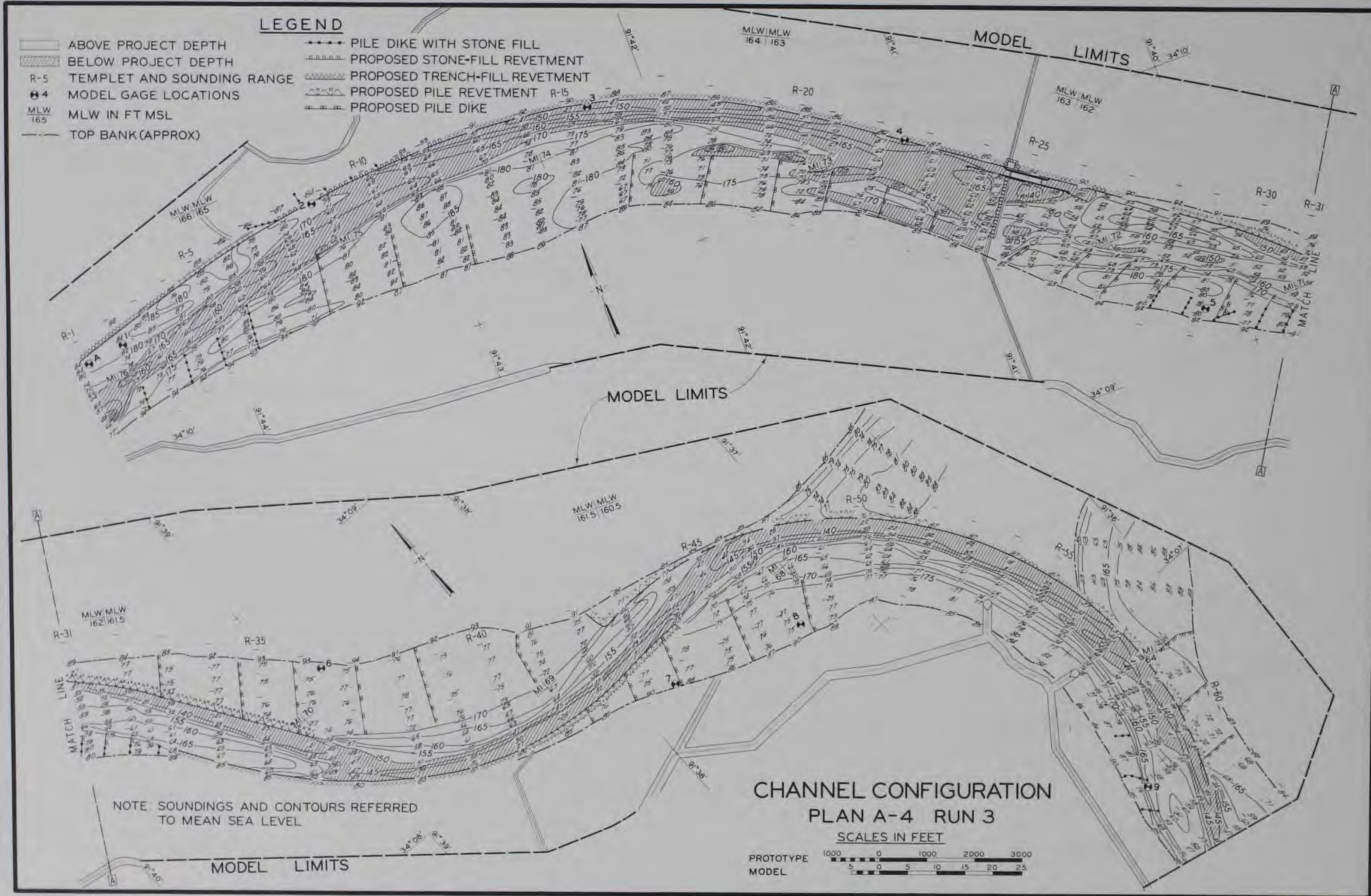
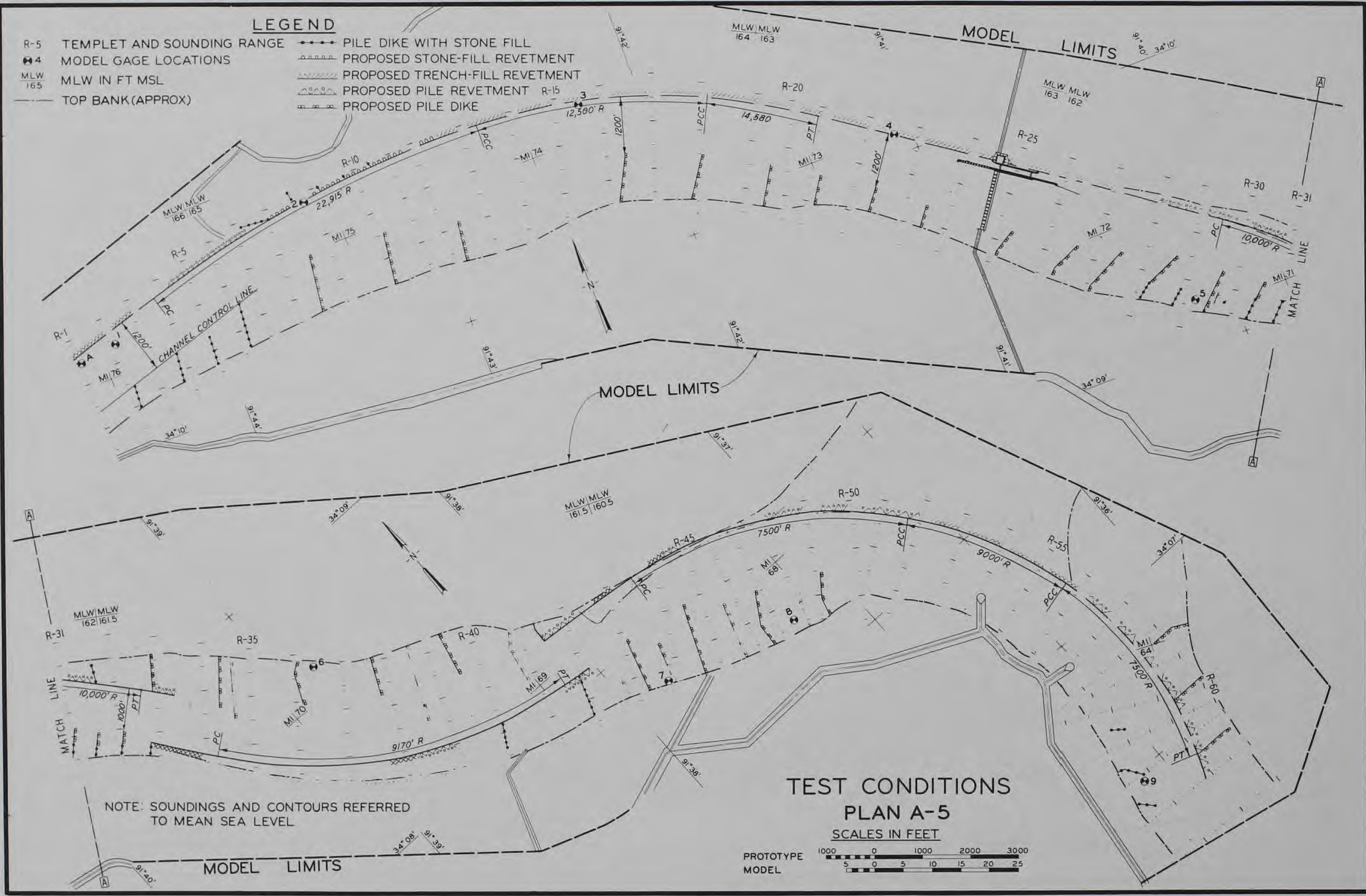
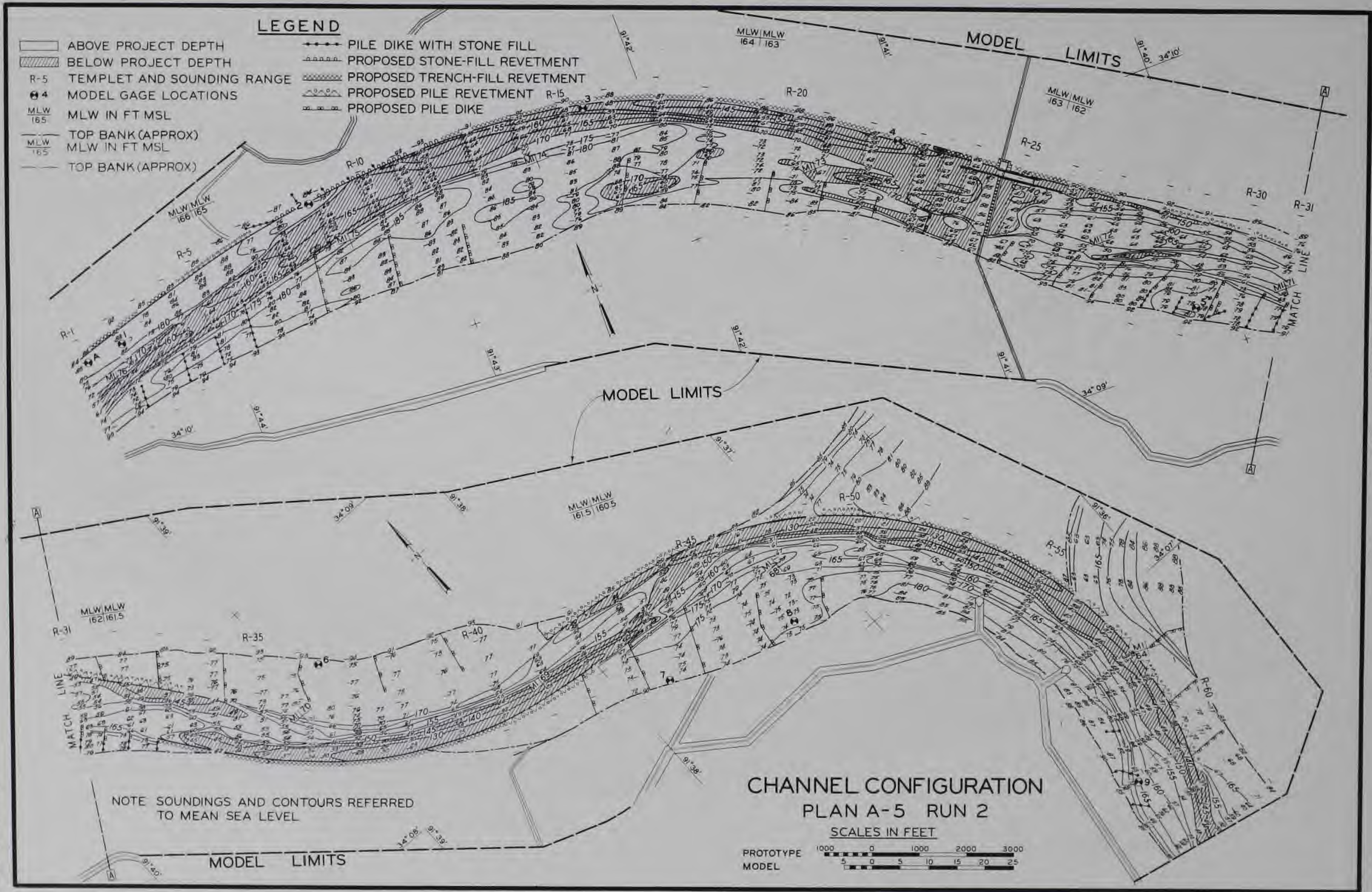

LEGEND
ABOVE PROJECT DEPTH
BELOW PROJECT DEPTH
R-5 TEMPLET AND SOUNDING RANGE
4 MODEL GAGE LOCATIONS
MLW 165 MLW IN FT MSL
TOP BANK (APPROX)
PILE DIKE WITH STONE FILL
PROPOSED STONE-FILL REVETMENT
PROPOSED TRENCH-FILL REVETMENT
PROPOSED PILE REVETMENT
PROPOSED PILE DIKE
MODEL LIMITS
MODEL LIMITS
MODEL LIMITS
MATCH LINE
MATCH LINE
NOTE: SOUNDINGS AND CONTOURS REFERRED TO MEAN SEA LEVEL
CHANNEL CONFIGURATION
PLAN A-4 RUN 3
SCALES IN FEET
PROTOTYPE 1000 0 1000 2000 3000
MODEL 5 0 5 10 15 20 25

PLATE 23

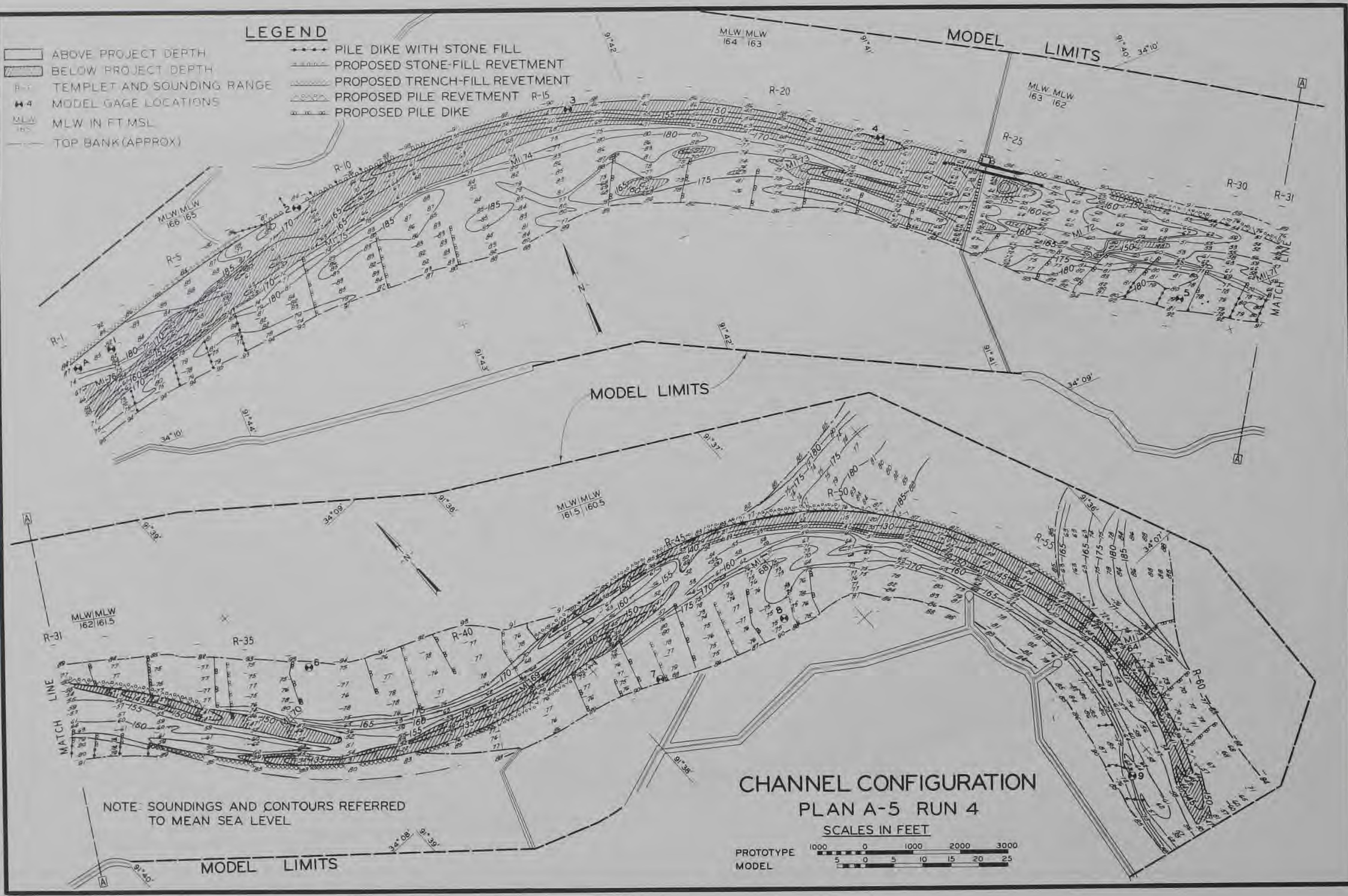

LEGEND
R-5 TEMPLET AND SOUNDING RANGE
4 MODEL GAGE LOCATIONS
MLW / 165 MLW IN FT MSL
TOP BANK (APPROX)
PILE DIKE WITH STONE FILL
PROPOSED STONE-FILL REVETMENT
PROPOSED TRENCH-FILL REVETMENT
PROPOSED PILE REVETMENT
PROPOSED PILE DIKE
MODEL LIMITS
CHANNEL CONTROL LINE
MATCH LINE
NOTE: SOUNDINGS AND CONTOURS REFERRED TO MEAN SEA LEVEL
TEST CONDITIONS
PLAN A-5
SCALES IN FEET
PROTOTYPE 1000 0 1000 2000 3000
MODEL 5 0 5 10 15 20 25

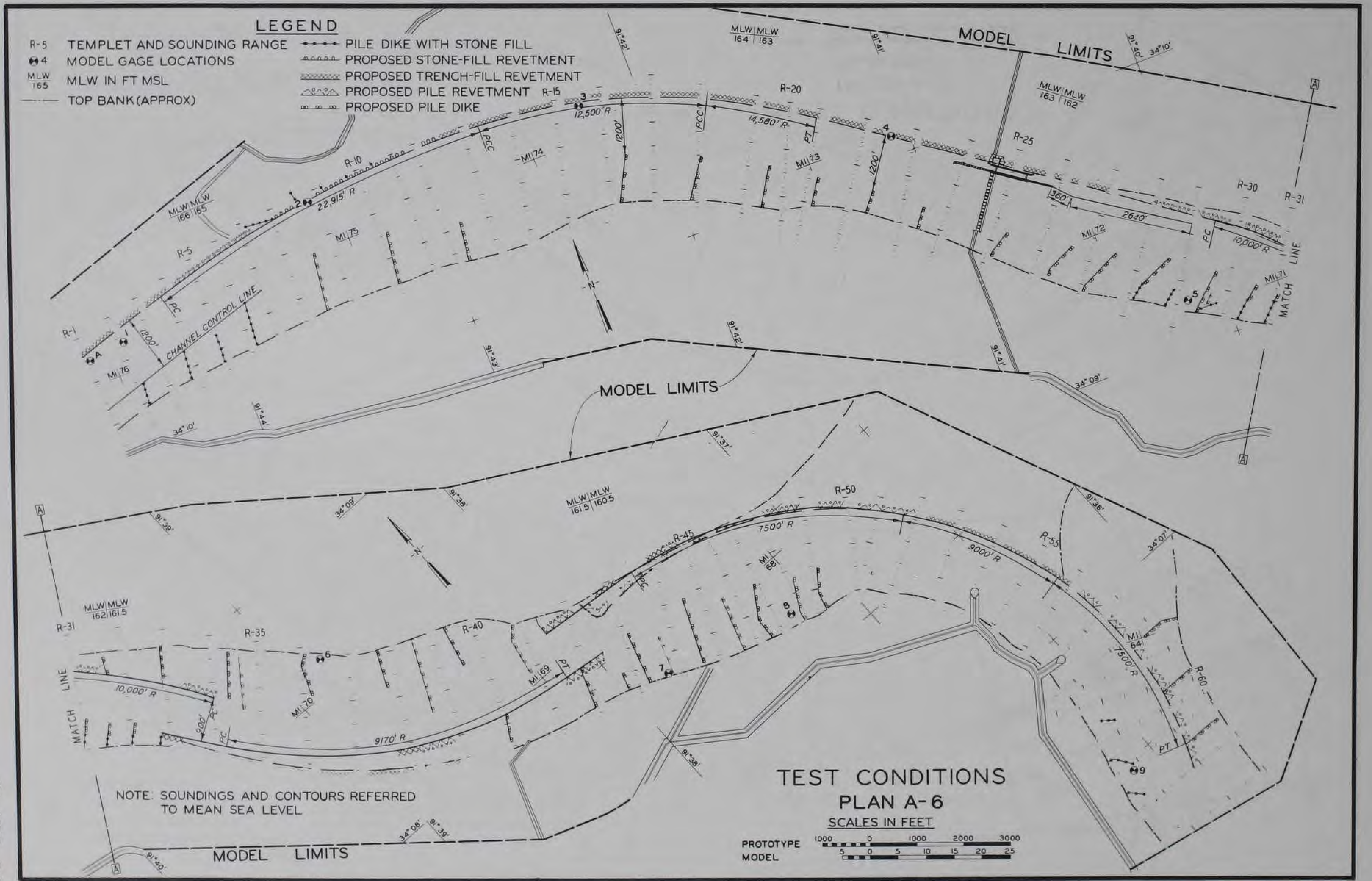


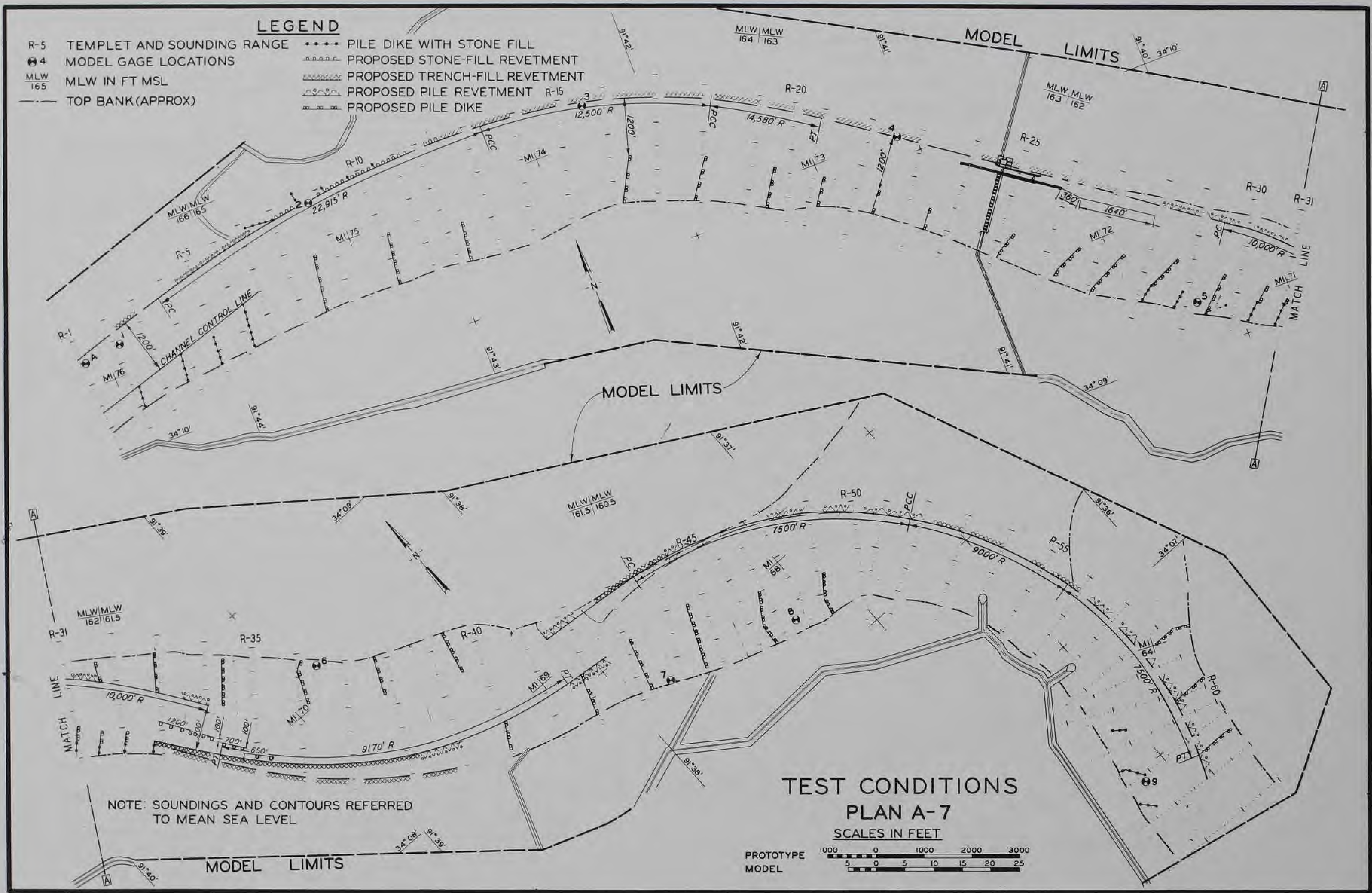
LEGEND
ABOVE PROJECT DEPTH
BELOW PROJECT DEPTH
R-5 TEMPLET AND SOUNDING RANGE
4 MODEL GAGE LOCATIONS
MLW / 165 MLW IN FT MSL
TOP BANK (APPROX)
PILE DIKE WITH STONE FILL
PROPOSED STONE-FILL REVETMENT
PROPOSED TRENCH-FILL REVETMENT
PROPOSED PILE REVETMENT
PROPOSED PILE DIKE
MODEL LIMITS
MATCH LINE
NOTE SOUNDINGS AND CONTOURS REFERRED TO MEAN SEA LEVEL
CHANNEL CONFIGURATION
PLAN A-5 RUN 2
SCALES IN FEET
PROTOTYPE
MODEL

PLATE 25

PLATE 26


LEGEND
ABOVE PROJECT DEPTH
BELOW PROJECT DEPTH
TEMPLET AND SOUNDING RANGE
MODEL GAGE LOCATIONS
MLW IN FT MSL
TOP BANK (APPROX)
PILE DIKE WITH STONE FILL
PROPOSED STONE-FILL REVETMENT
PROPOSED TRENCH-FILL REVETMENT
PROPOSED PILE REVETMENT
PROPOSED PILE DIKE
MODEL LIMITS
MATCH LINE
NOTE: SOUNDINGS AND CONTOURS REFERRED TO MEAN SEA LEVEL
CHANNEL CONFIGURATION
PLAN A-5 RUN 4
SCALES IN FEET
PROTOTYPE 1000 0 1000 2000 3000
MODEL 5 0 5 10 15 20 25


LEGEND
R-5 TEMPLET AND SOUNDING RANGE
4 MODEL GAGE LOCATIONS
MLW 165 MLW IN FT MSL
TOP BANK (APPROX)
PILE DIKE WITH STONE FILL
PROPOSED STONE-FILL REVETMENT
PROPOSED TRENCH-FILL REVETMENT
PROPOSED PILE REVETMENT
PROPOSED PILE DIKE
MODEL LIMITS
MODEL LIMITS
MODEL LIMITS
CHANNEL CONTROL LINE
MATCH LINE
MATCH LINE
22,915' R
12,500' R
14,580' R
10,000' R
10,000' R
9170' R
7500' R
9000' R
7500' R
NOTE: SOUNDINGS AND CONTOURS REFERRED
TO MEAN SEA LEVEL
TEST CONDITIONS
PLAN A-6
SCALES IN FEET
PROTOTYPE
MODEL


LEGEND
R-5 TEMPLET AND SOUNDING RANGE
4 MODEL GAGE LOCATIONS
MLW/165 MLW IN FT MSL
TOP BANK (APPROX)
PILE DIKE WITH STONE FILL
PROPOSED STONE-FILL REVETMENT
PROPOSED TRENCH-FILL REVETMENT
PROPOSED PILE REVETMENT
PROPOSED PILE DIKE
MODEL LIMITS
MODEL LIMITS
MODEL LIMITS
CHANNEL CONTROL LINE
MATCH LINE
MATCH LINE
12,500' R
14,580' R
22,915' R
10,000' R
10,000' R
7500' R
9000' R
9170' R
7500' R
NOTE: SOUNDINGS AND CONTOURS REFERRED TO MEAN SEA LEVEL
TEST CONDITIONS
PLAN A-7
SCALES IN FEET
PROTOTYPE 1000 0 1000 2000 3000
MODEL 5 0 5 10 15 20 25

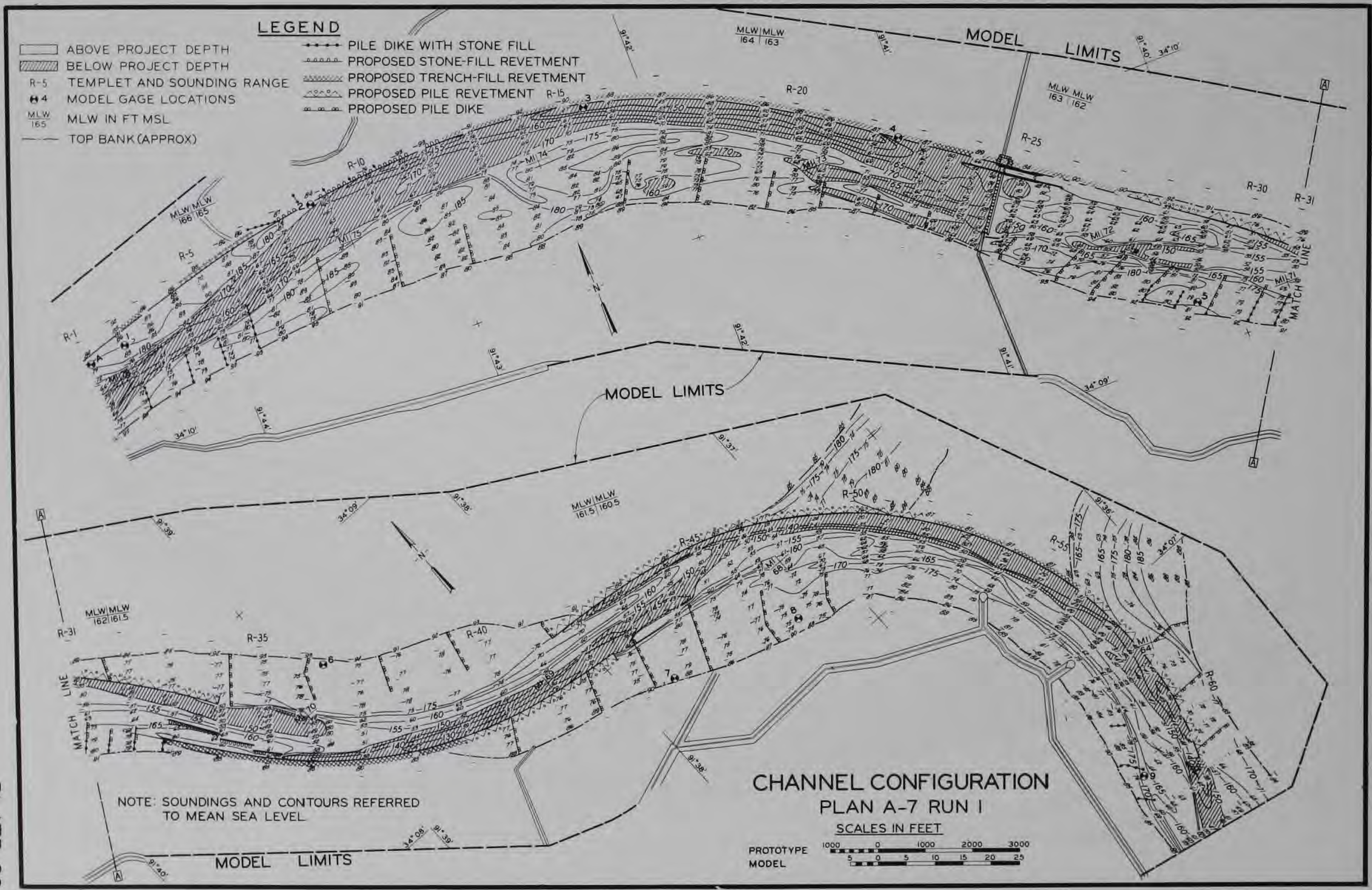
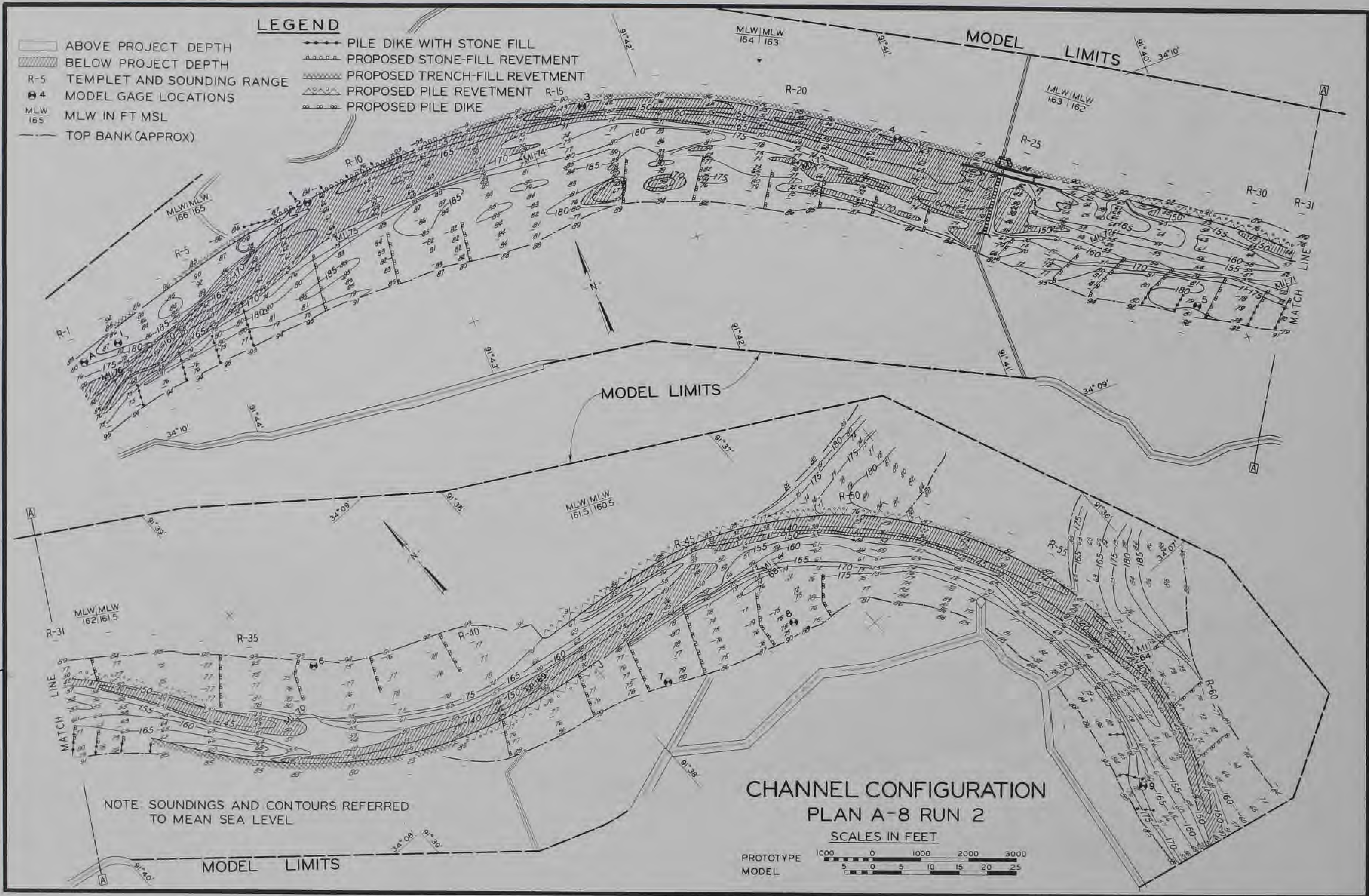

LEGEND
ABOVE PROJECT DEPTH
BELOW PROJECT DEPTH
R-5 TEMPLET AND SOUNDING RANGE
4 MODEL GAGE LOCATIONS
MLW 165 MLW IN FT MSL
TOP BANK (APPROX)
PILE DIKE WITH STONE FILL
PROPOSED STONE-FILL REVETMENT
PROPOSED TRENCH-FILL REVETMENT
PROPOSED PILE REVETMENT
PROPOSED PILE DIKE
MODEL LIMITS
MODEL LIMITS
MODEL LIMITS
MATCH LINE
MATCH LINE
NOTE: SOUNDINGS AND CONTOURS REFERRED TO MEAN SEA LEVEL.
CHANNEL CONFIGURATION
PLAN A-7 RUN I
SCALES IN FEET
PROTOTYPE 1000 0 1000 2000 3000
MODEL 5 0 5 10 15 20 25

PLATE 29



CHANNEL CONFIGURATION
PLAN A-8 RUN 2

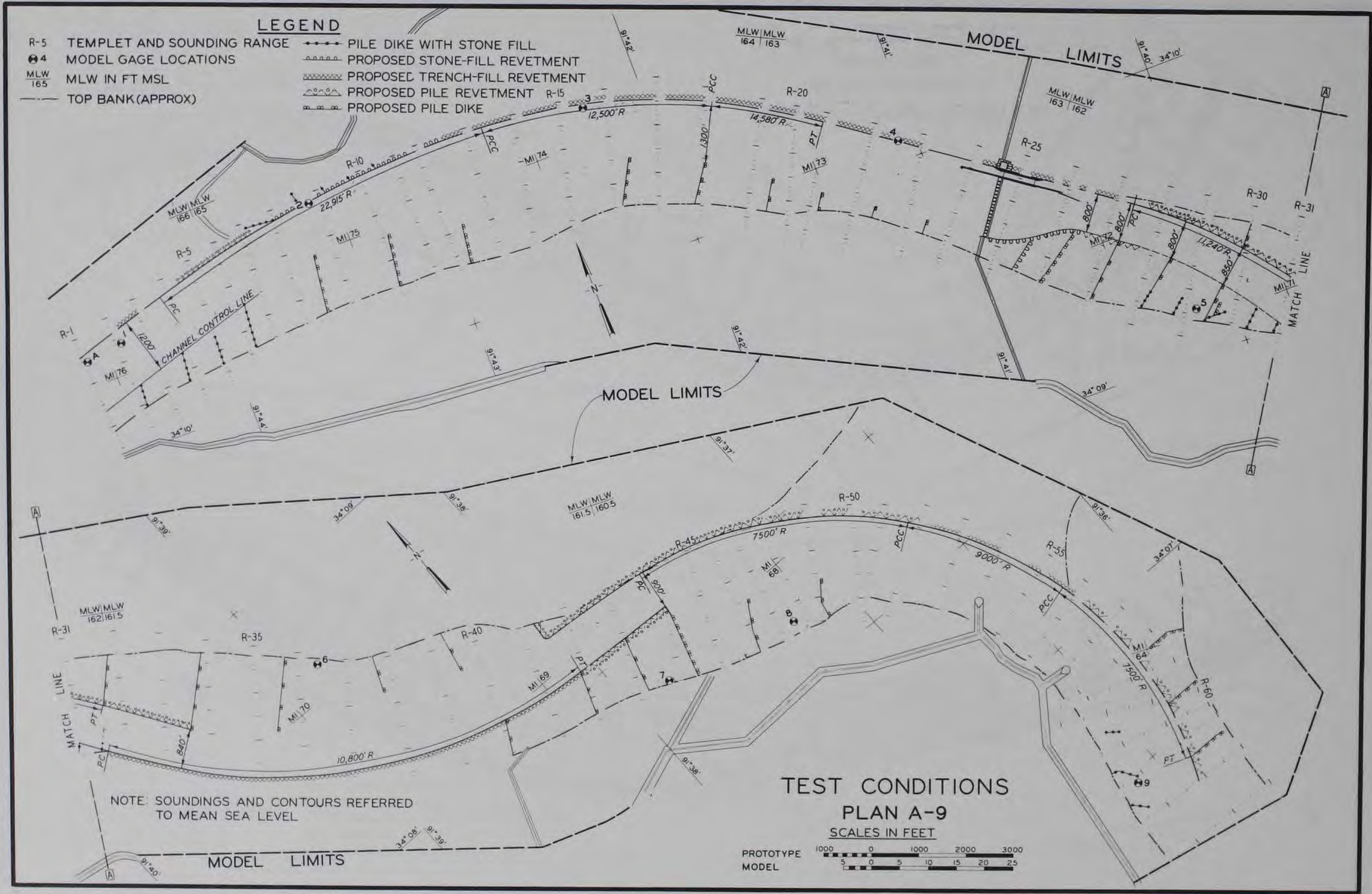
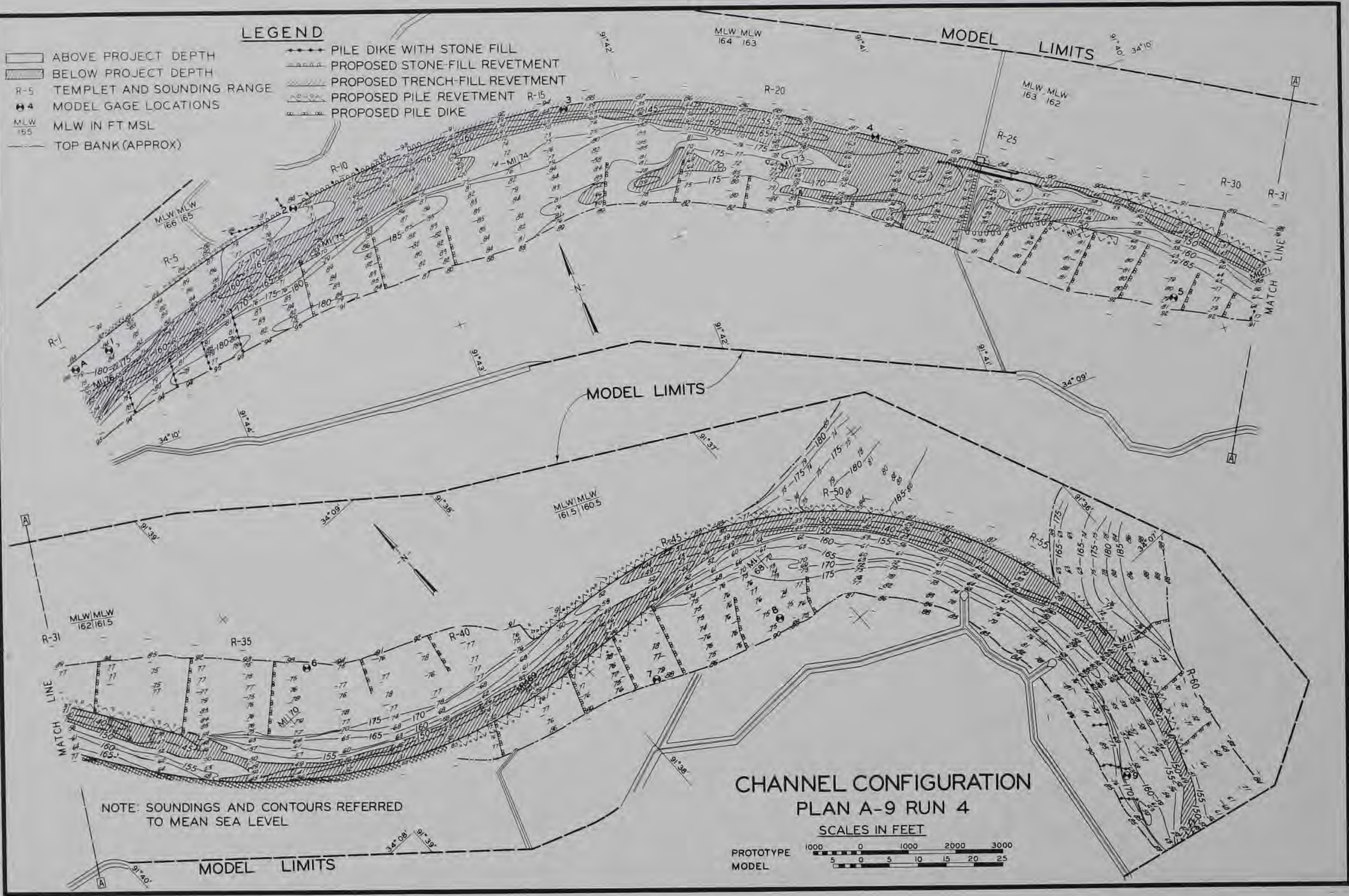

LEGEND
R-5 TEMPLET AND SOUNDING RANGE
4 MODEL GAGE LOCATIONS
MLW/165 MLW IN FT MSL
TOP BANK (APPROX)
PILE DIKE WITH STONE FILL
PROPOSED STONE-FILL REVETMENT
PROPOSED TRENCH-FILL REVETMENT
PROPOSED PILE REVETMENT
PROPOSED PILE DIKE
MODEL LIMITS
MODEL LIMITS
MODEL LIMITS
CHANNEL CONTROL LINE
MATCH LINE
MATCH LINE
N
N
R-1
R-5
R-10
R-15
R-20
R-25
R-30
R-31
R-31
R-35
R-40
R-45
R-50
R-55
R-60
22,915' R
12,500' R
14,580' R
11,240' R
7500' R
9000' R
7500' R
10,800' R
NOTE: SOUNDINGS AND CONTOURS REFERRED
TO MEAN SEA LEVEL
TEST CONDITIONS
PLAN A-9
SCALES IN FEET
PROTOTYPE 1000 0 1000 2000 3000
MODEL 5 0 5 10 15 20 25

PLATE 31



CHANNEL CONFIGURATION
PLAN A-9 RUN 4

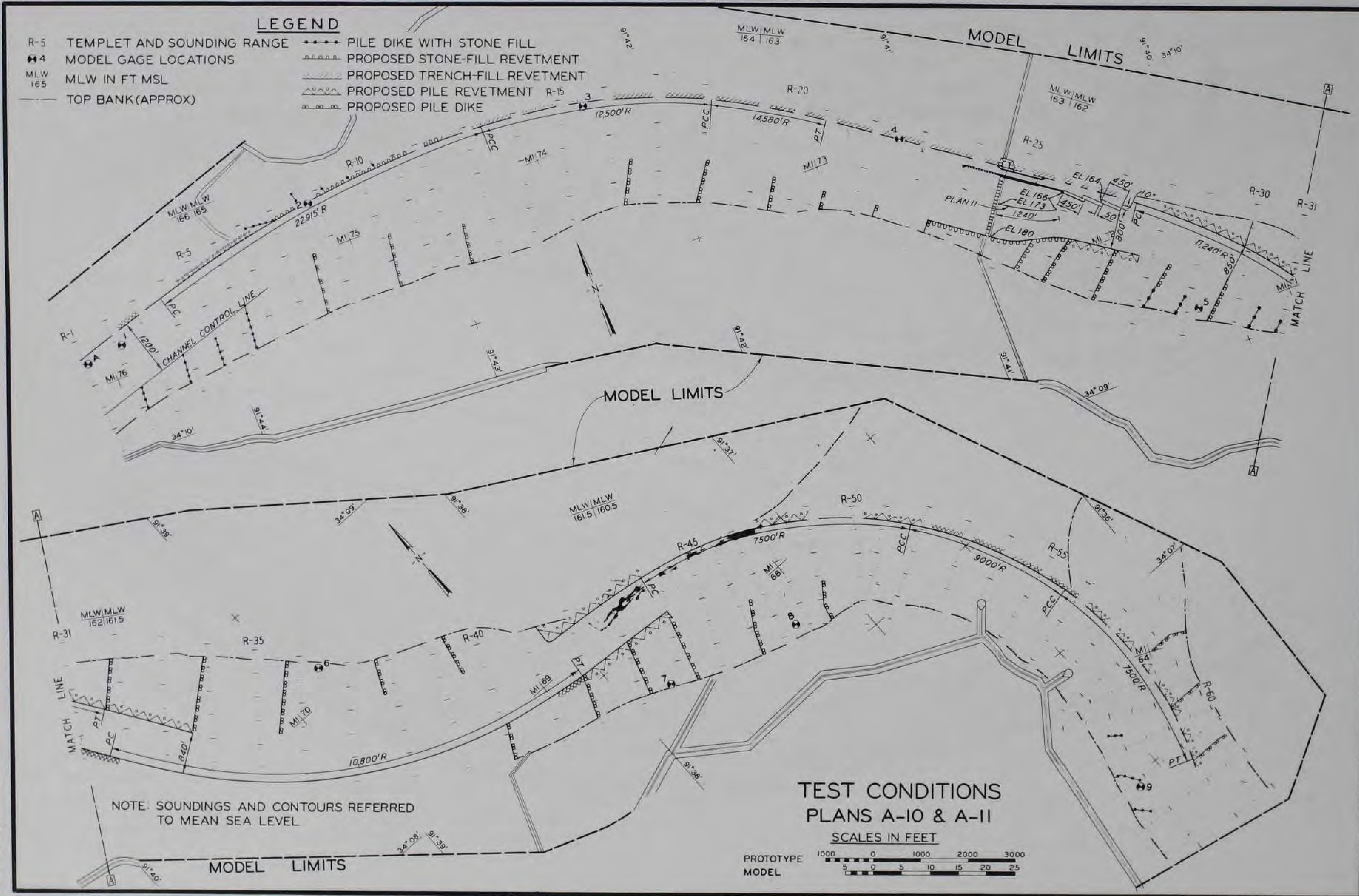
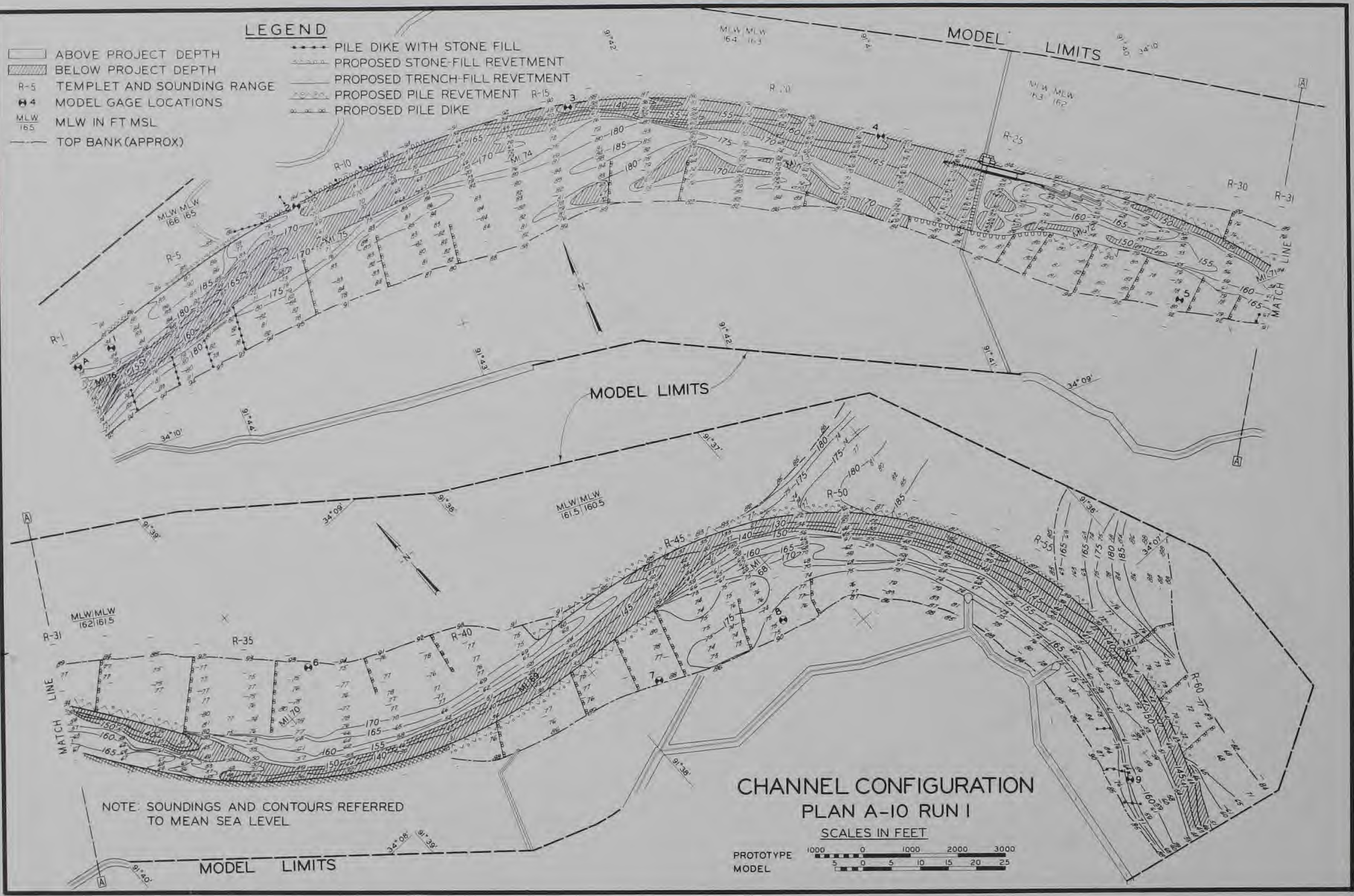
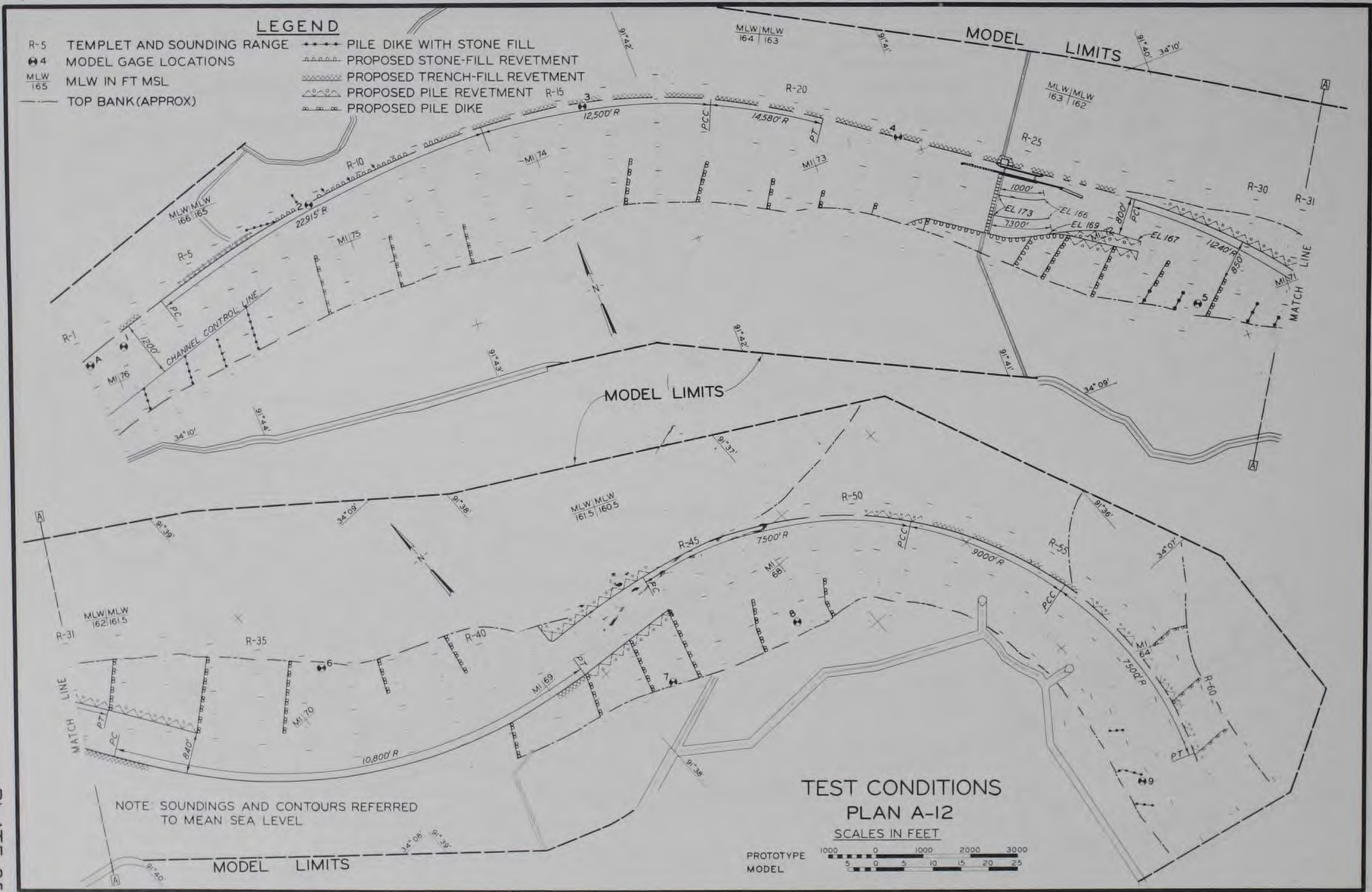
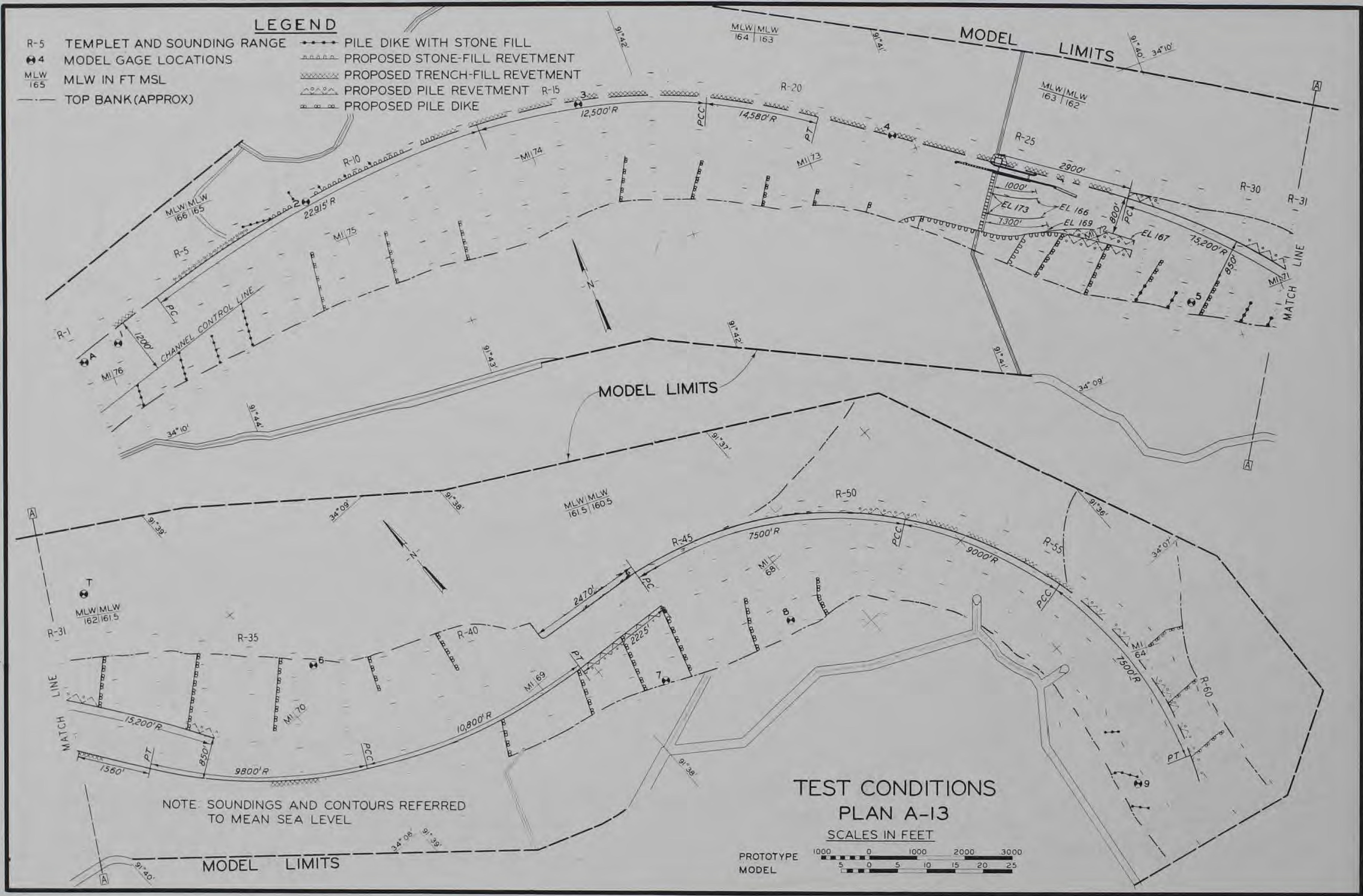

LEGEND
R-5 TEMPLET AND SOUNDING RANGE
4 MODEL GAGE LOCATIONS
MLW 165 MLW IN FT MSL
TOP BANK (APPROX)
PILE DIKE WITH STONE FILL
PROPOSED STONE-FILL REVETMENT
PROPOSED TRENCH-FILL REVETMENT
PROPOSED PILE REVETMENT
PROPOSED PILE DIKE
MODEL LIMITS
MODEL LIMITS
MODEL LIMITS
CHANNEL CONTROL LINE
MATCH LINE
MATCH LINE
PLAN II
12,500'R
14,580'R
22,915'R
11,240'R
7500'R
9000'R
10,800'R
NOTE: SOUNDINGS AND CONTOURS REFERRED TO MEAN SEA LEVEL
TEST CONDITIONS
PLANS A-10 & A-11
SCALES IN FEET
PROTOTYPE 1000 0 1000 2000 3000
MODEL 5 0 5 10 15 20 25

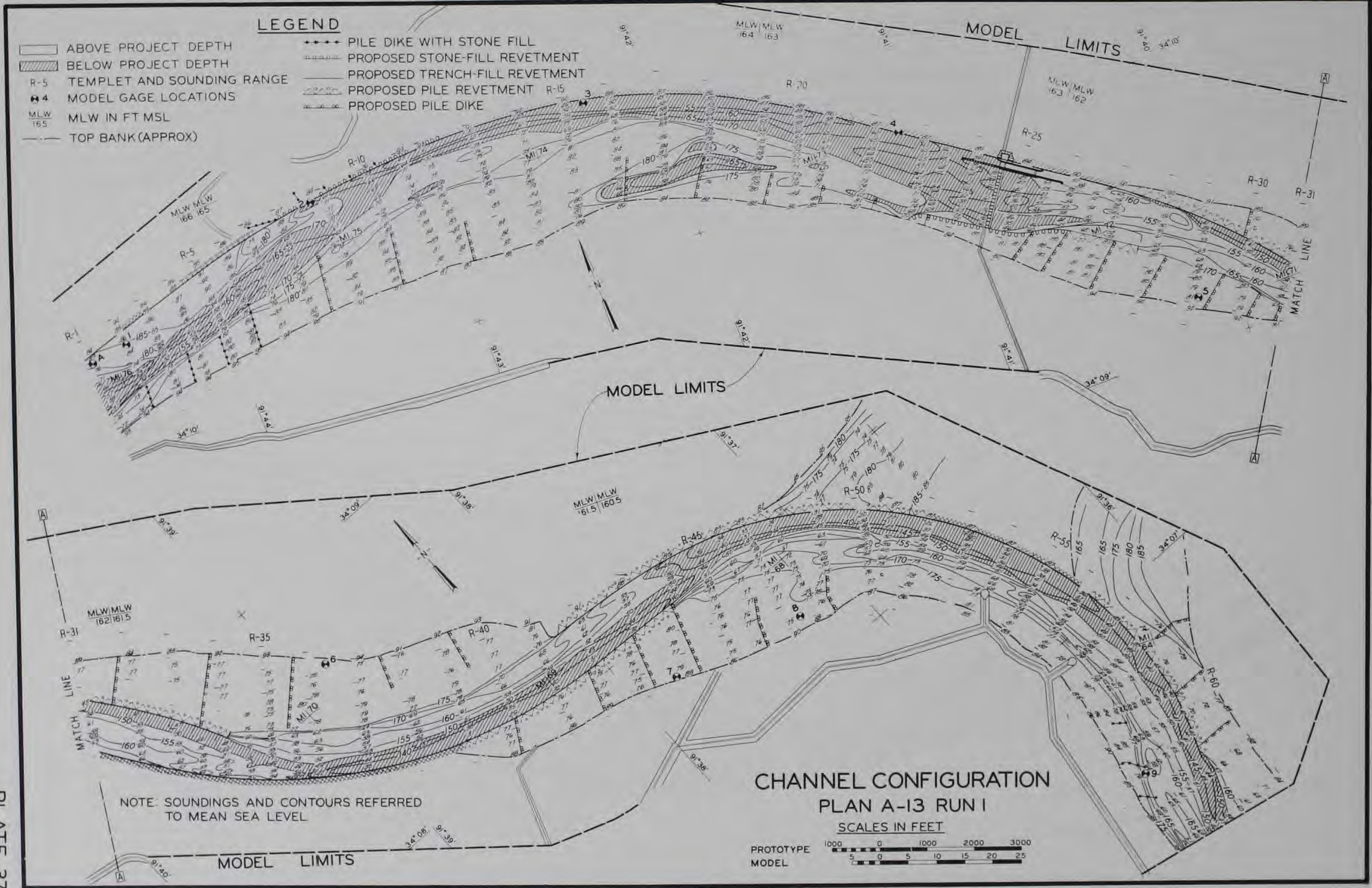
PLATE 33



CHANNEL CONFIGURATION
PLAN A-10 RUN 1

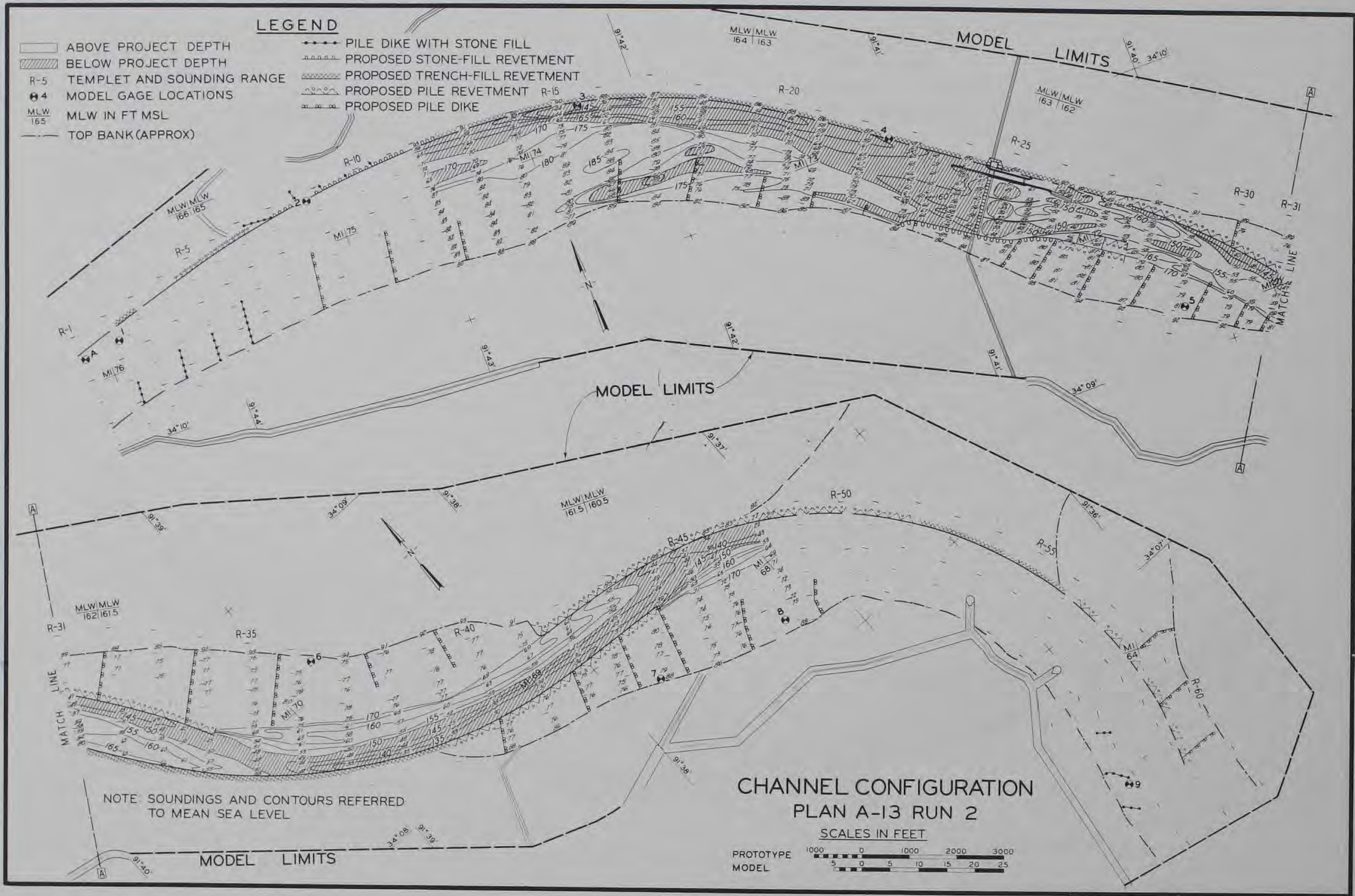

LEGEND
R-5 TEMPLET AND SOUNDING RANGE
4 MODEL GAGE LOCATIONS
MLW/165 MLW IN FT MSL
TOP BANK (APPROX)
PILE DIKE WITH STONE FILL
PROPOSED STONE-FILL REVETMENT
PROPOSED TRENCH-FILL REVETMENT
PROPOSED PILE REVETMENT
PROPOSED PILE DIKE
MODEL LIMITS
MODEL LIMITS
MODEL LIMITS
CHANNEL CONTROL LINE
MATCH LINE
MATCH LINE
12,500' R
14,580' R
22,915' R
1240' R
7500' R
9000' R
7500' R
10,800' R
EL 173
EL 166
EL 169
EL 167
NOTE: SOUNDINGS AND CONTOURS REFERRED TO MEAN SEA LEVEL
TEST CONDITIONS
PLAN A-12
SCALES IN FEET
PROTOTYPE 1000 0 1000 2000 3000
MODEL 5 0 5 10 15 20 25


LEGEND
R-5 TEMPLET AND SOUNDING RANGE
4 MODEL GAGE LOCATIONS
MLW/165 MLW IN FT MSL
TOP BANK (APPROX)
PILE DIKE WITH STONE FILL
PROPOSED STONE-FILL REVETMENT
PROPOSED TRENCH-FILL REVETMENT
PROPOSED PILE REVETMENT
PROPOSED PILE DIKE
MODEL LIMITS
CHANNEL CONTROL LINE
MATCH LINE
NOTE: SOUNDINGS AND CONTOURS REFERRED TO MEAN SEA LEVEL
TEST CONDITIONS
PLAN A-13
SCALES IN FEET
PROTOTYPE 1000 0 1000 2000 3000
MODEL 5 0 5 10 15 20 25

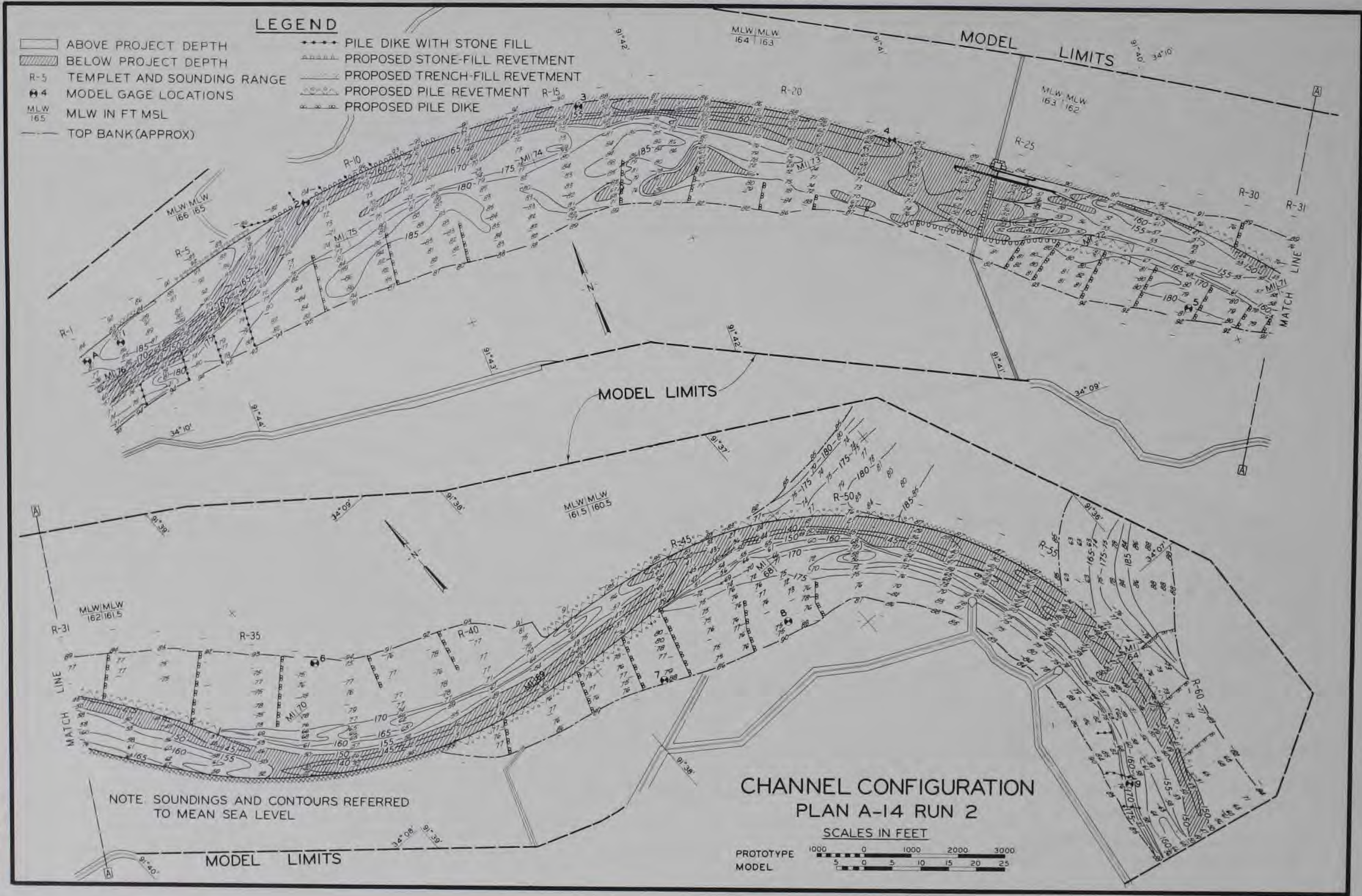


CHANNEL CONFIGURATION
PLAN A-13 RUN I

PLATE 38

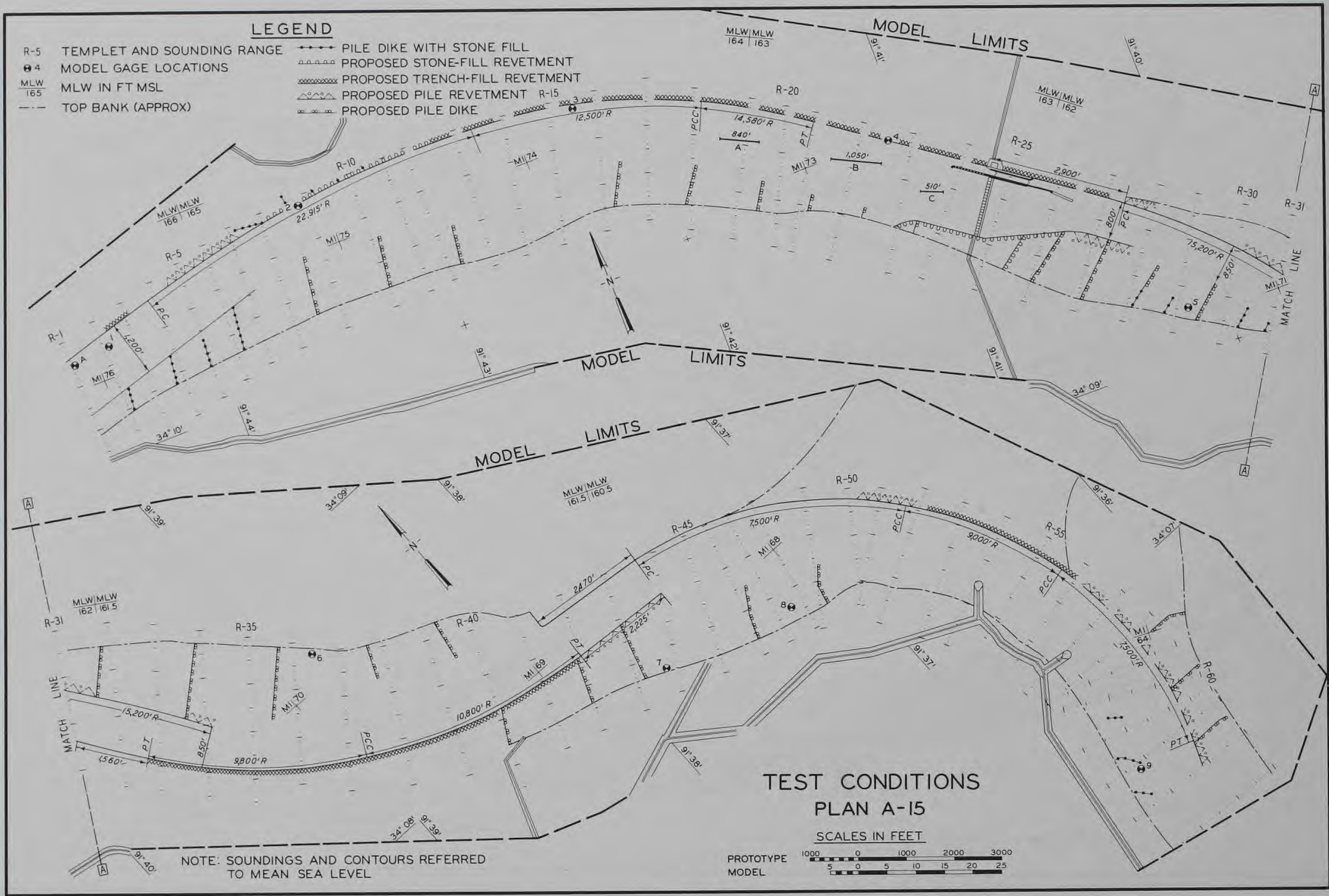


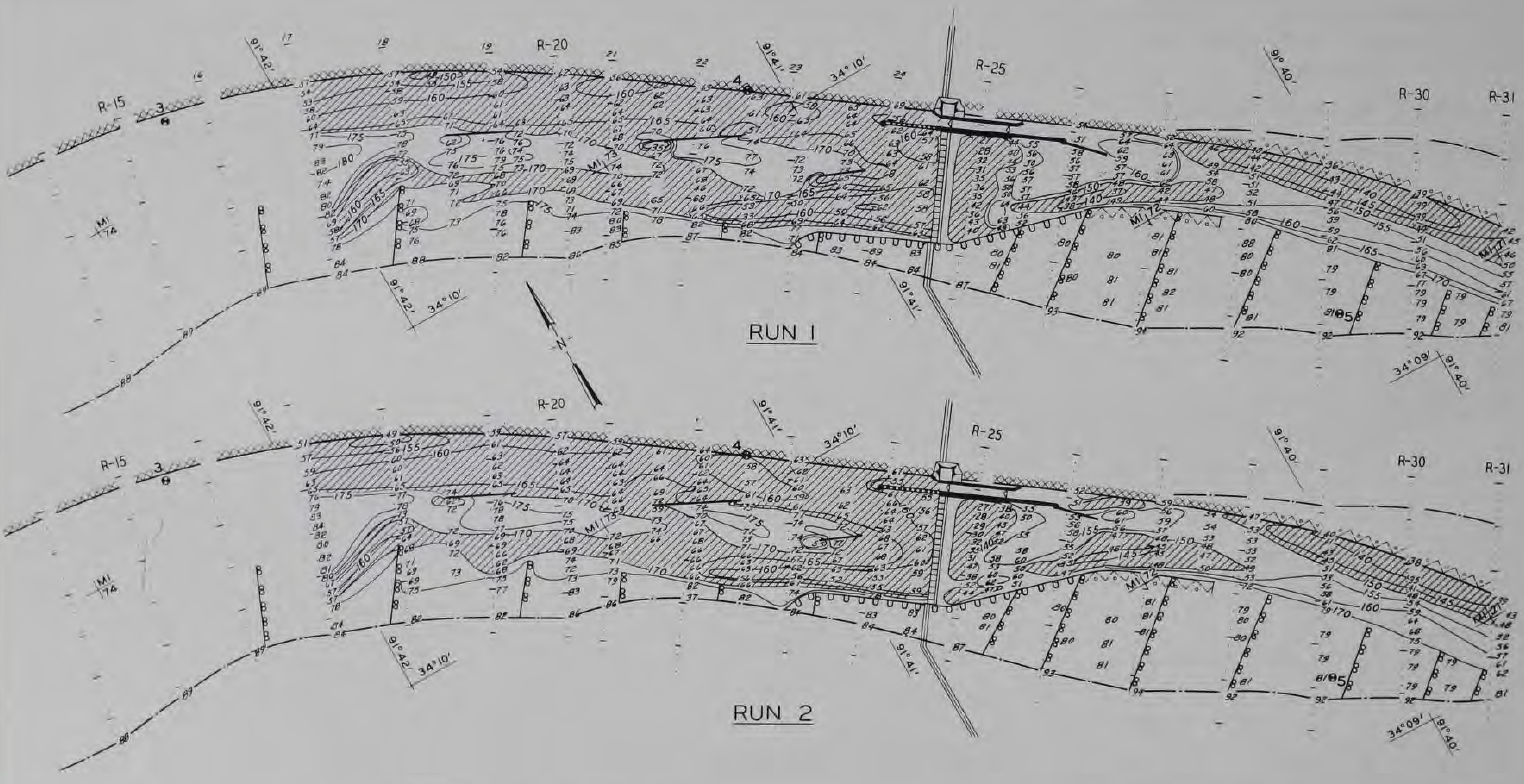
LEGEND
ABOVE PROJECT DEPTH
BELOW PROJECT DEPTH
R-5 TEMPLET AND SOUNDING RANGE
4 MODEL GAGE LOCATIONS
MLW 165 MLW IN FT MSL
TOP BANK (APPROX)
PILE DIKE WITH STONE FILL
PROPOSED STONE-FILL REVETMENT
PROPOSED TRENCH-FILL REVETMENT
PROPOSED PILE REVETMENT
PROPOSED PILE DIKE
MODEL LIMITS
MATCH LINE
NOTE: SOUNDINGS AND CONTOURS REFERRED TO MEAN SEA LEVEL
CHANNEL CONFIGURATION
PLAN A-13 RUN 2
SCALES IN FEET
PROTOTYPE 1000 0 1000 2000 3000
MODEL 5 0 5 10 15 20 25



CHANNEL CONFIGURATION
PLAN A-14 RUN 2

PLATE 40


LEGEND
R-5 TEMPLET AND SOUNDING RANGE
4 MODEL GAGE LOCATIONS
MLW 165 MLW IN FT MSL
TOP BANK (APPROX)
PILE DIKE WITH STONE FILL
PROPOSED STONE-FILL REVETMENT
PROPOSED TRENCH-FILL REVETMENT
PROPOSED PILE REVETMENT
PROPOSED PILE DIKE
MODEL LIMITS
MLW MLW 164 163
MLW MLW 163 162
MLW MLW 166 165
MLW MLW 161.5 160.5
MLW MLW 162 161.5
R-1
R-5
R-10
R-15
R-20
R-25
R-30
R-31
R-35
R-40
R-45
R-50
R-55
R-60
12,500' R
14,580' R
22,915' R
15,200' R
7,500' R
9,000' R
10,800' R
9,800' R
2,900'
2,470'
2,225'
1,200'
1,560'
850'
800'
840' A
1,050' B
510' C
PC
PCC
PT
MI 76
MI 75
MI 74
MI 73
MI 71
MI 70
MI 69
MI 68
MI 64
91° 44'
91° 43'
91° 42'
91° 41'
91° 40'
91° 39'
91° 38'
91° 37'
91° 36'
34° 10'
34° 09'
34° 08'
34° 07'
N
MATCH LINE
A
NOTE: SOUNDINGS AND CONTOURS REFERRED TO MEAN SEA LEVEL
TEST CONDITIONS
PLAN A-15
SCALES IN FEET
PROTOTYPE 1000 0 1000 2000 3000
MODEL 5 0 5 10 15 20 25



LEGEND

- ABOVE PROJECT DEPTH
- BELOW PROJECT DEPTH
- R-5 TEMPLET AND SOUNDING RANGE
- 4 MODEL GAGE LOCATIONS
- MLW/165 MLW IN FT MSL
- TOP BANK (APPROX)
- PILE DIKE WITH STONE FILL
- PROPOSED STONE-FILL REVETMENT
- PROPOSED TRENCH-FILL REVETMENT
- PROPOSED PILE REVETMENT
- PROPOSED PILE DIKE

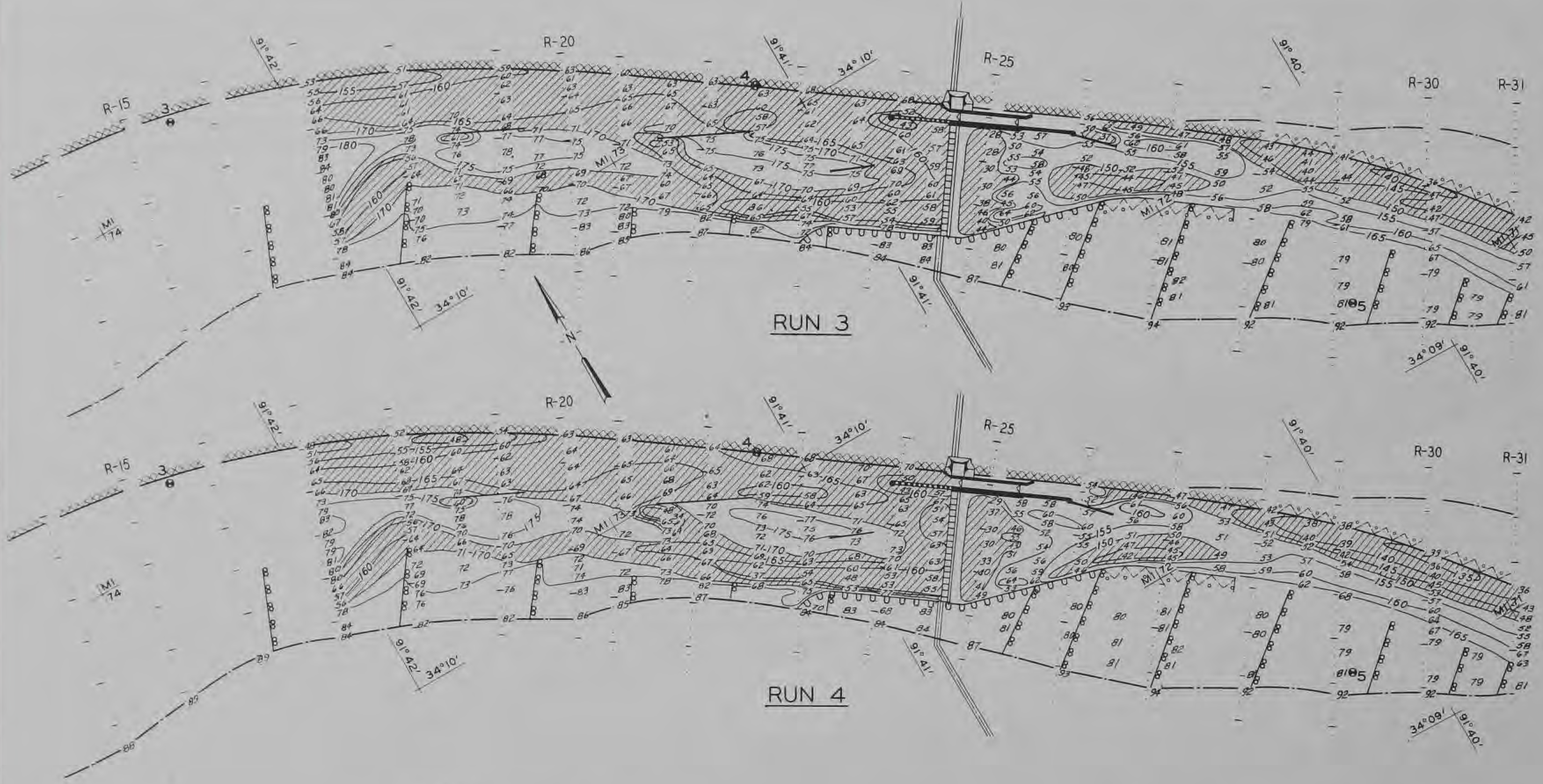
CHANNEL CONFIGURATION

PLAN A-15, RUNS 1 AND 2

SCALES IN FEET

PROTOTYPE 1000 0 1000 2000

MODEL 5 0 5 10 15



LEGEND

- ABOVE PROJECT DEPTH
- BELOW PROJECT DEPTH
- R-5 TEMPLET AND SOUNDING RANGE
- 4 MODEL GAGE LOCATIONS
- MLW/165 MLW IN FT MSL
- TOP BANK (APPROX)
- PILE DIKE WITH STONE FILL
- PROPOSED STONE-FILL REVETMENT
- PROPOSED TRENCH-FILL REVETMENT
- PROPOSED PILE REVETMENT
- PROPOSED PILE DIKE

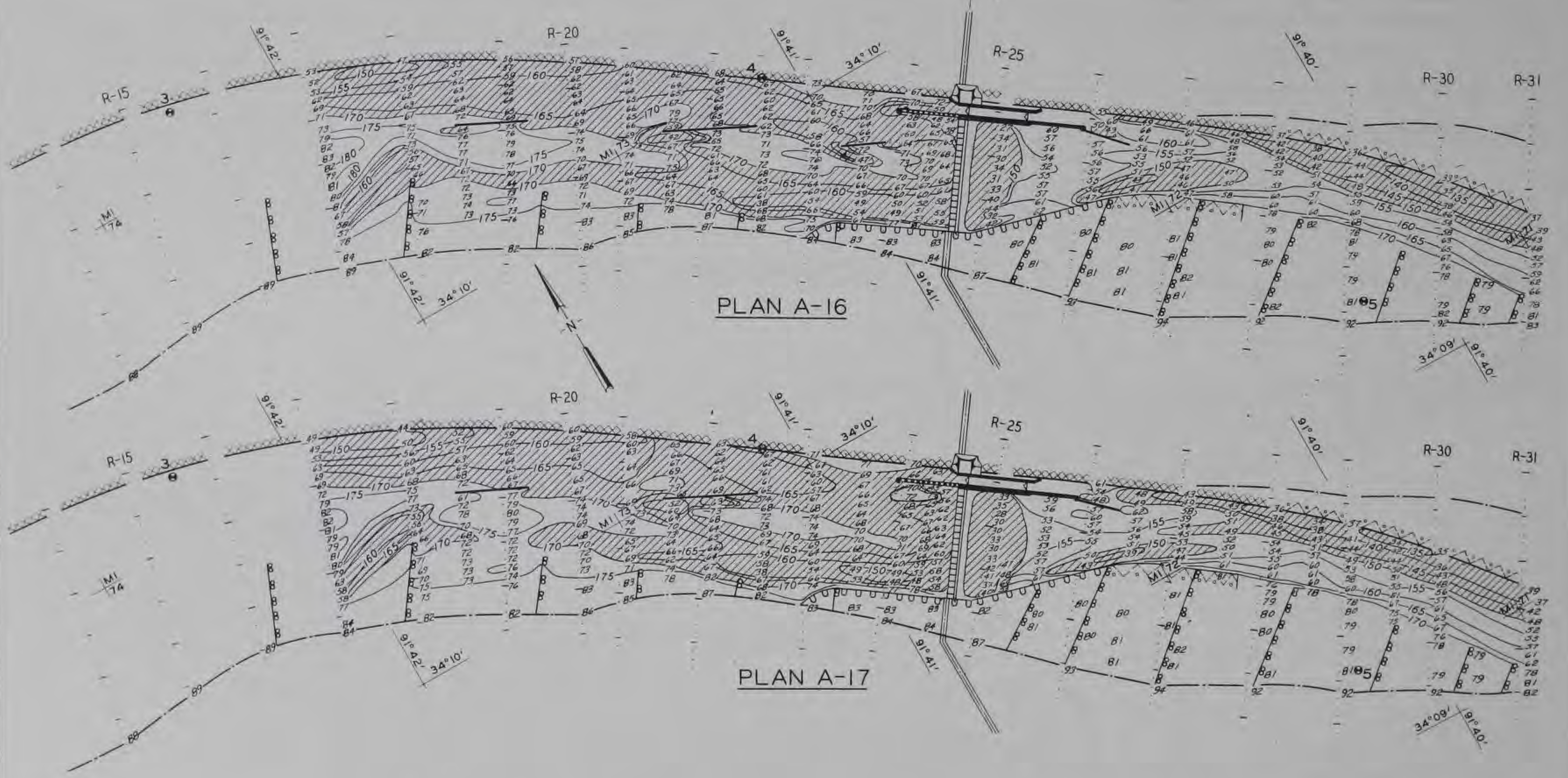
CHANNEL CONFIGURATION

PLAN A-15, RUNS 3 AND 4

SCALES IN FEET

PROTOTYPE 1000 0 1000 2000

MODEL 5 0 5 10 15



LEGEND

ABOVE PROJECT DEPTH
BELOW PROJECT DEPTH
R-5 TEMPLET AND SOUNDING RANGE
4 MODEL GAGE LOCATIONS
$\frac{\text{MLW}}{165}$ MLW IN FT MSL
TOP BANK (APPROX)
PILE DIKE WITH STONE FILL
PROPOSED STONE-FILL REVETMENT
PROPOSED TRENCH-FILL REVETMENT
PROPOSED PILE REVETMENT
PROPOSED PILE DIKE

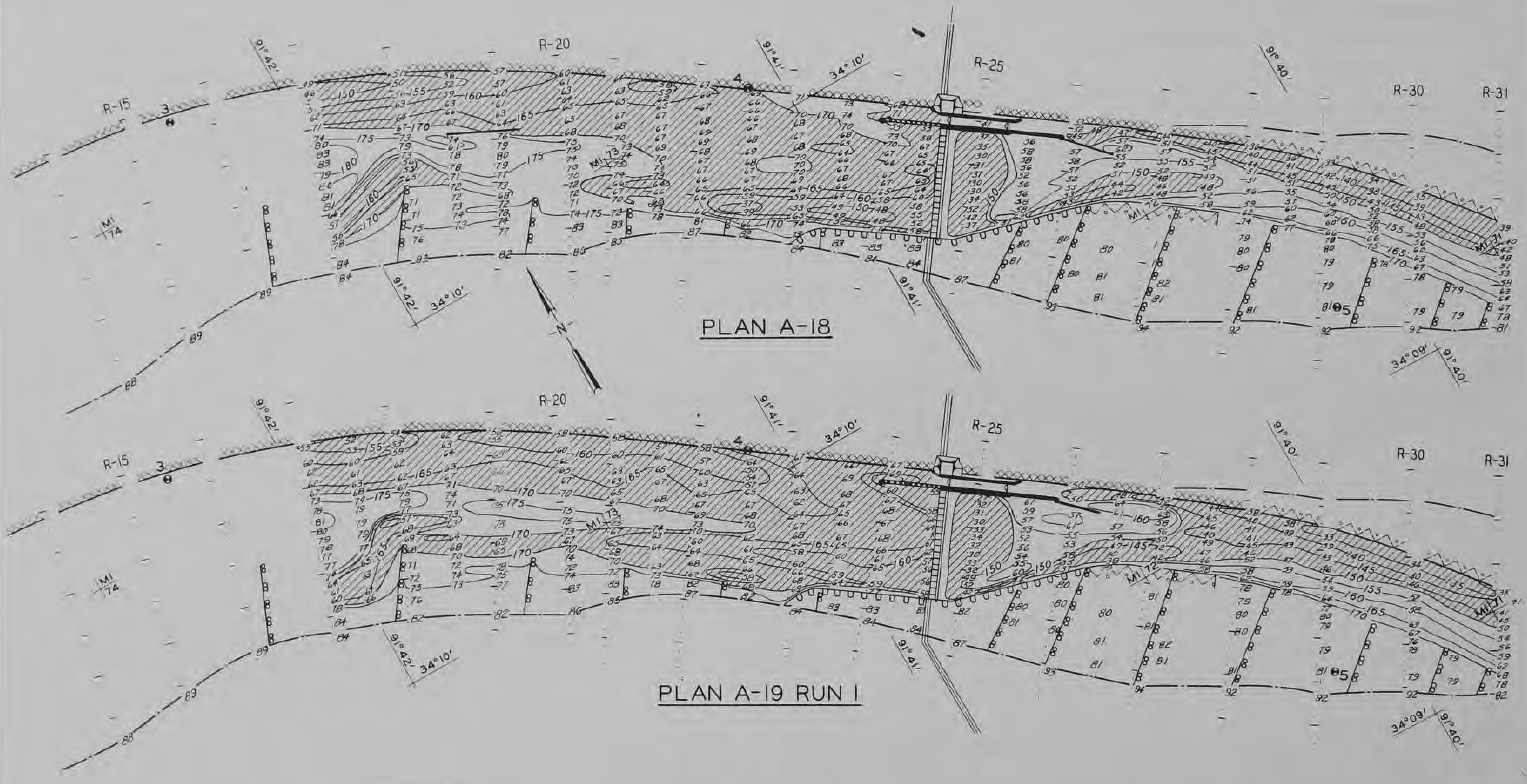
CHANNEL CONFIGURATION

PLANS A-16 AND A-17

SCALES IN FEET

PROTOTYPE 1000 0 1000 2000

MODEL 5 0 5 10 15



LEGEND

ABOVE PROJECT DEPTH
BELOW PROJECT DEPTH
R-5 TEMPLET AND SOUNDING RANGE
4 MODEL GAGE LOCATIONS
MLW / 165 MLW IN FT MSL
TOP BANK (APPROX)
PILE DIKE WITH STONE FILL
PROPOSED STONE-FILL REVETMENT
PROPOSED TRENCH-FILL REVETMENT
PROPOSED PILE REVETMENT
PROPOSED PILE DIKE

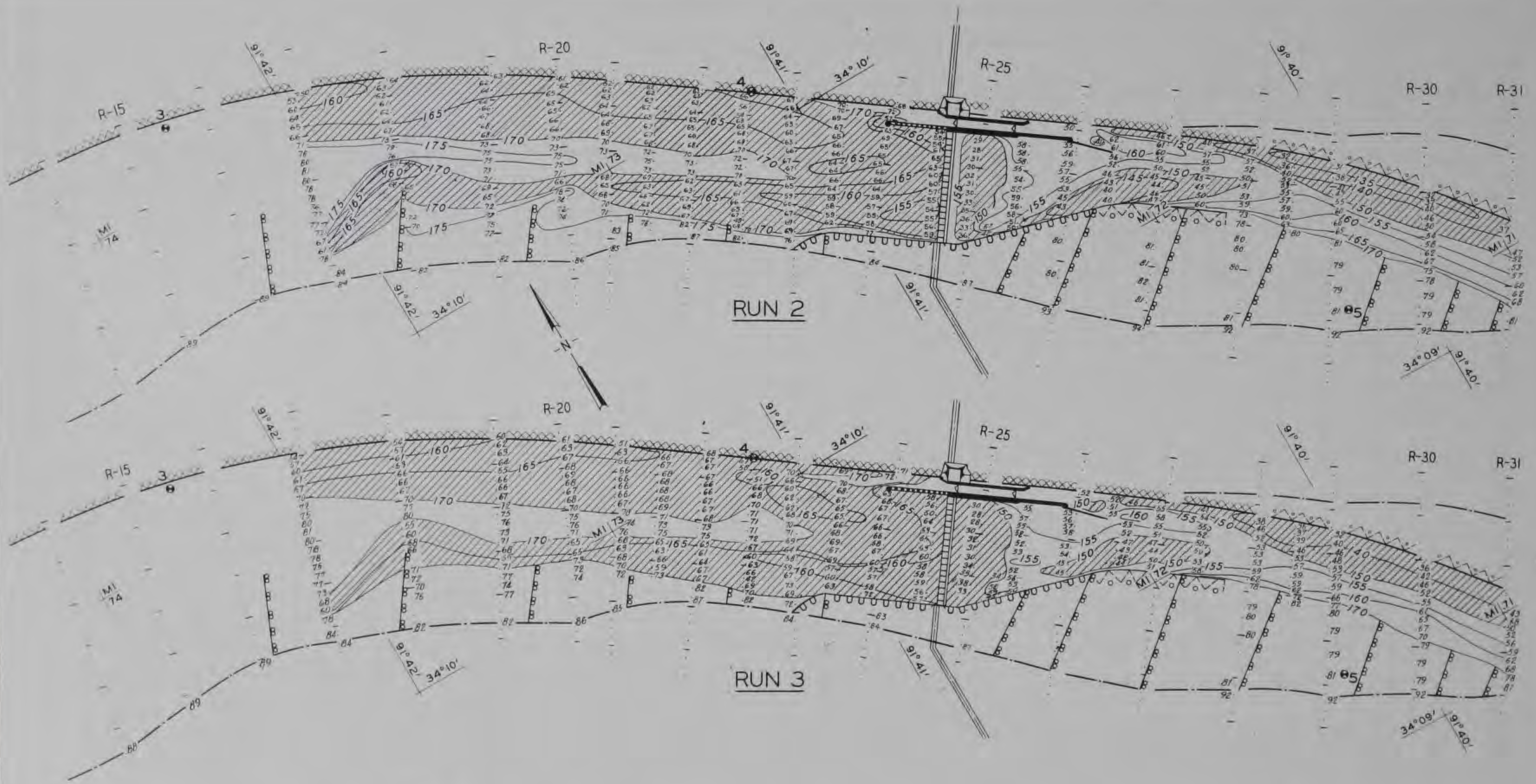
CHANNEL CONFIGURATION

PLANS A-18 AND A-19

SCALES IN FEET

PROTOTYPE 1000 0 1000 2000

MODEL 5 0 5 10 15



LEGEND

	ABOVE PROJECT DEPTH		PILE DIKE WITH STONE FILL
	BELOW PROJECT DEPTH		PROPOSED STONE-FILL REVETMENT
R-5	TEMPLET AND SOUNDING RANGE		PROPOSED TRENCH-FILL REVETMENT
⊕ 4	MODEL GAGE LOCATIONS		PROPOSED PILE REVETMENT
MLW 165	MLW IN FT MSL		PROPOSED PILE DIKE
---	TOP BANK (APPROX)		

CHANNEL CONFIGURATION

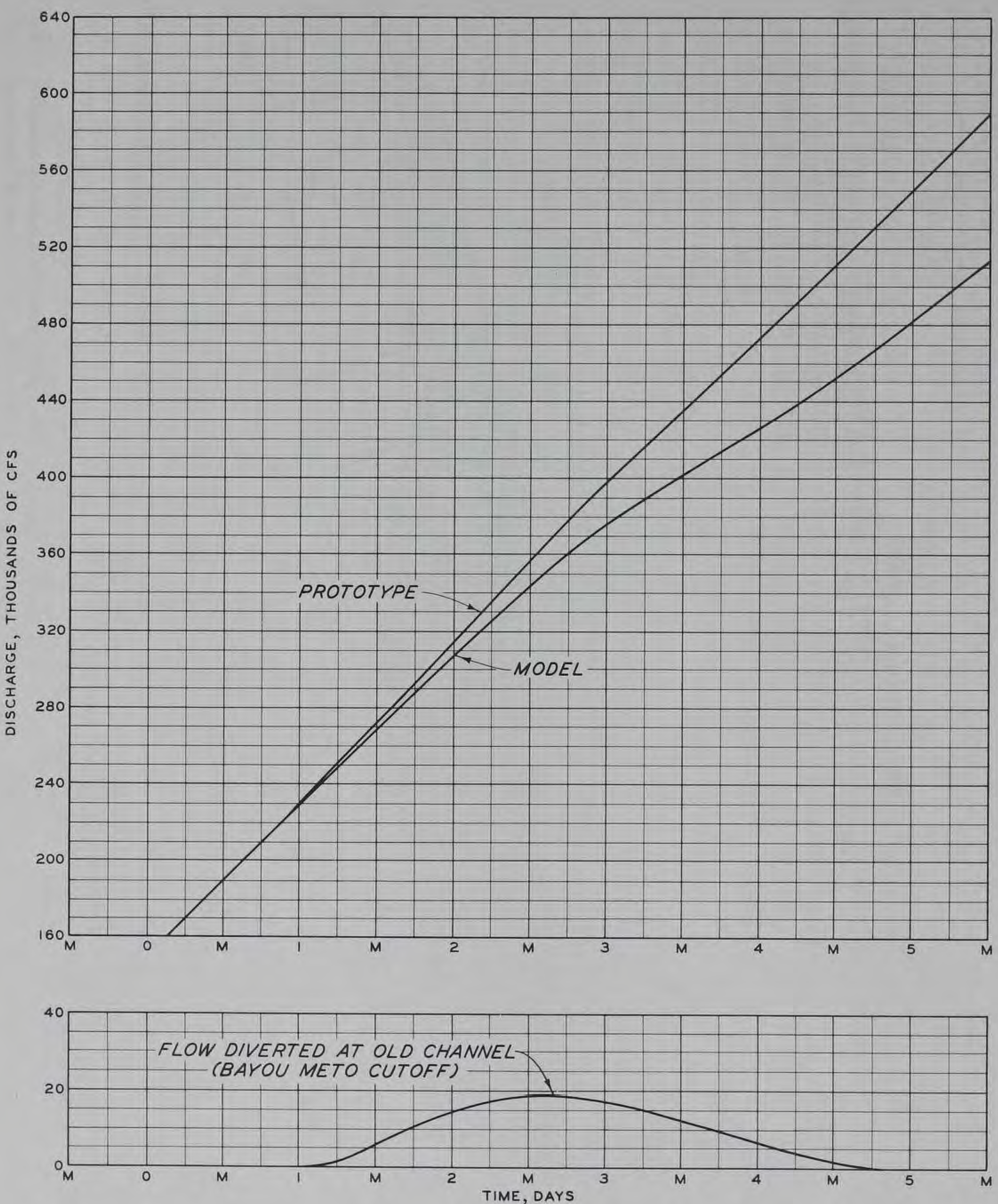
PLAN A-19, RUNS 2 AND 3

SCALES IN FEET

PROTOTYPE 1000 0 1000 2000

MODEL 5 0 5 10 15

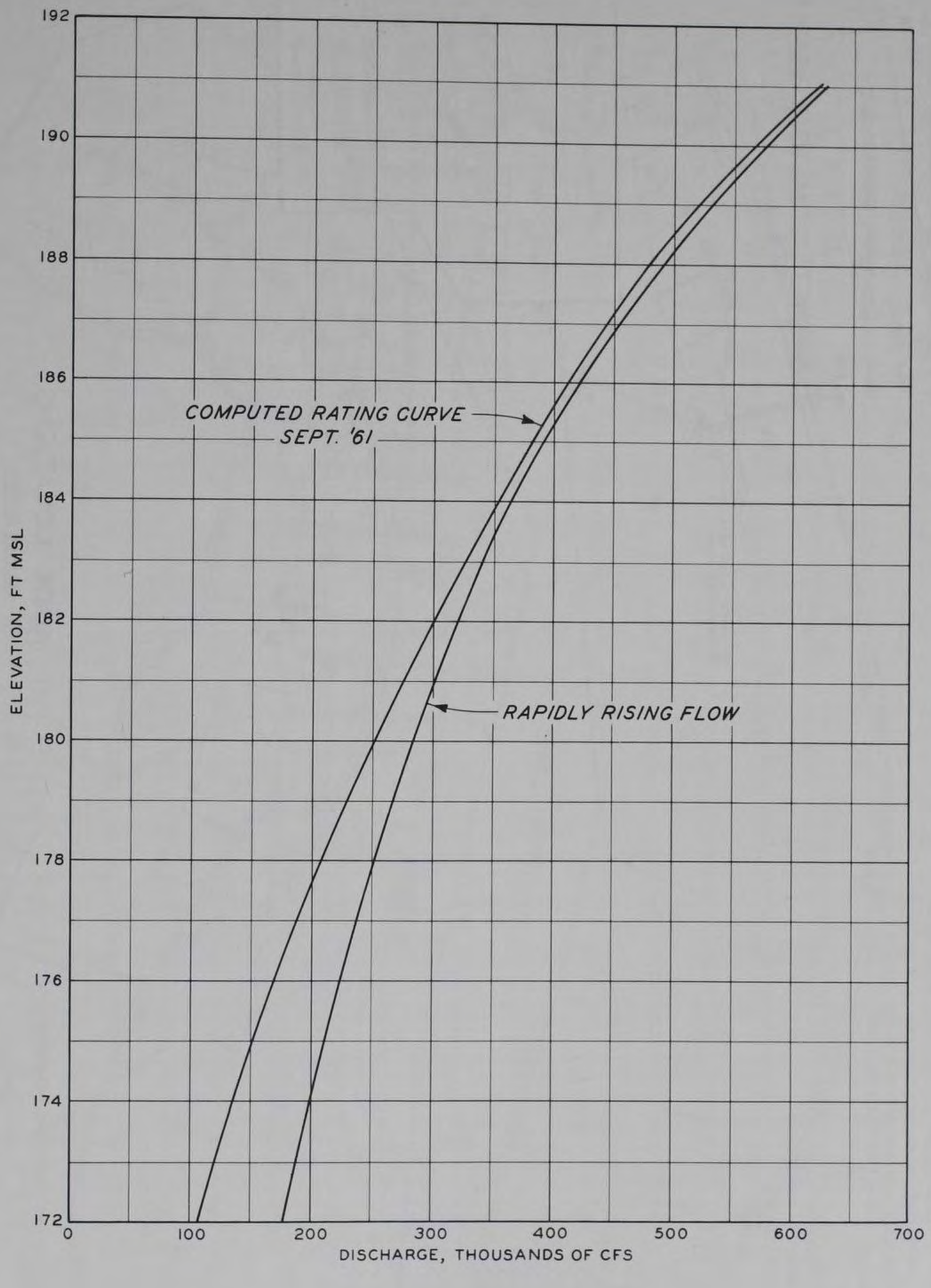
PLATE 45



NOTE: MODEL INFLOW REPRESENTS THE ESTIMATED MODIFICATION OF PROTOTYPE DISCHARGES REQUIRED TO COMPENSATE FOR OVERBANK AREAS NOT REPRODUCED IN MODEL.

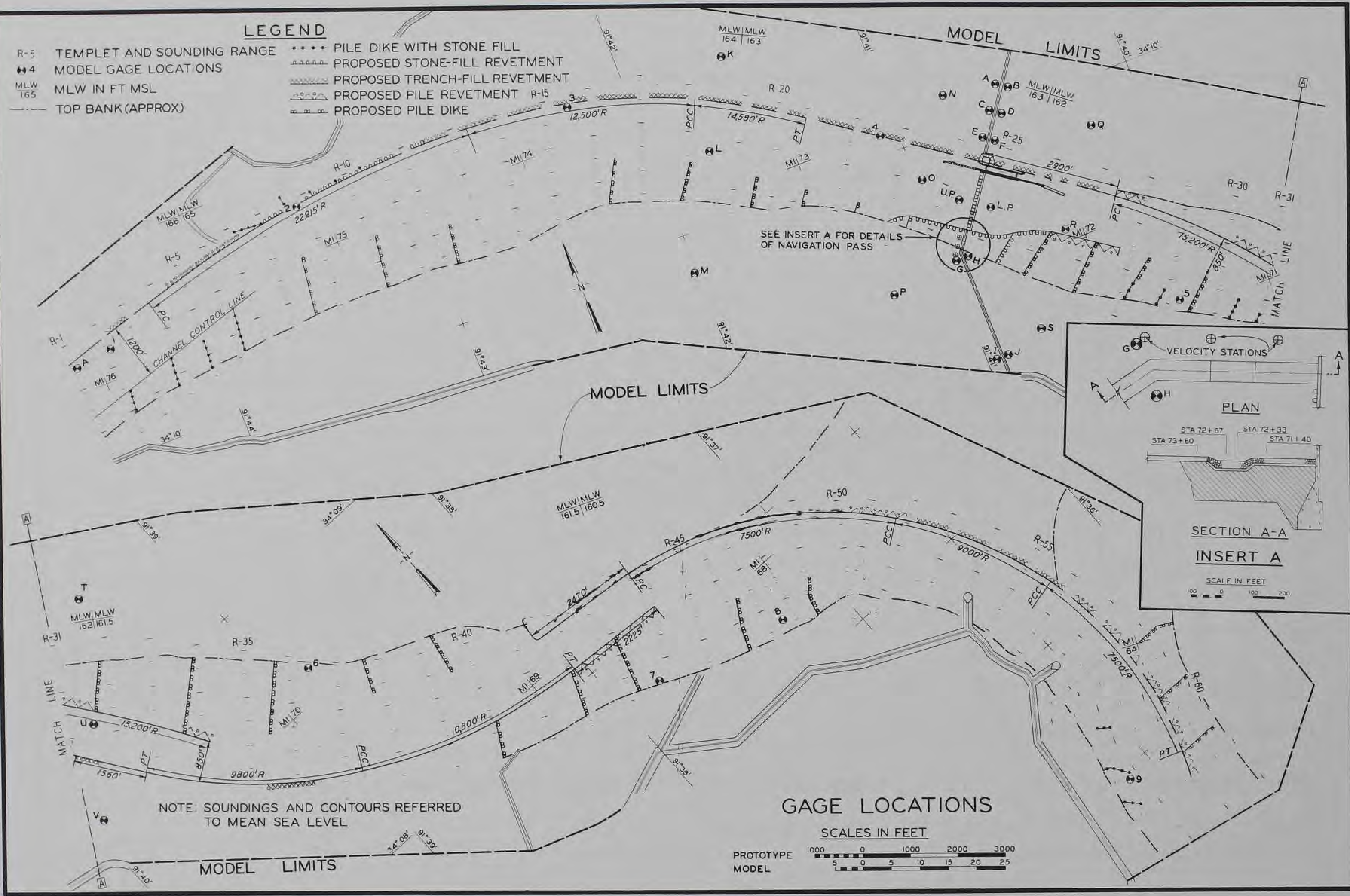
HYDROGRAPH IS MAY 1943, SECOND CREST, WHICH HAD THE HIGHEST RATE OF CHANGE OF FLOW, AT THESE DISCHARGES, IN PERIOD 1927 TO DATE.

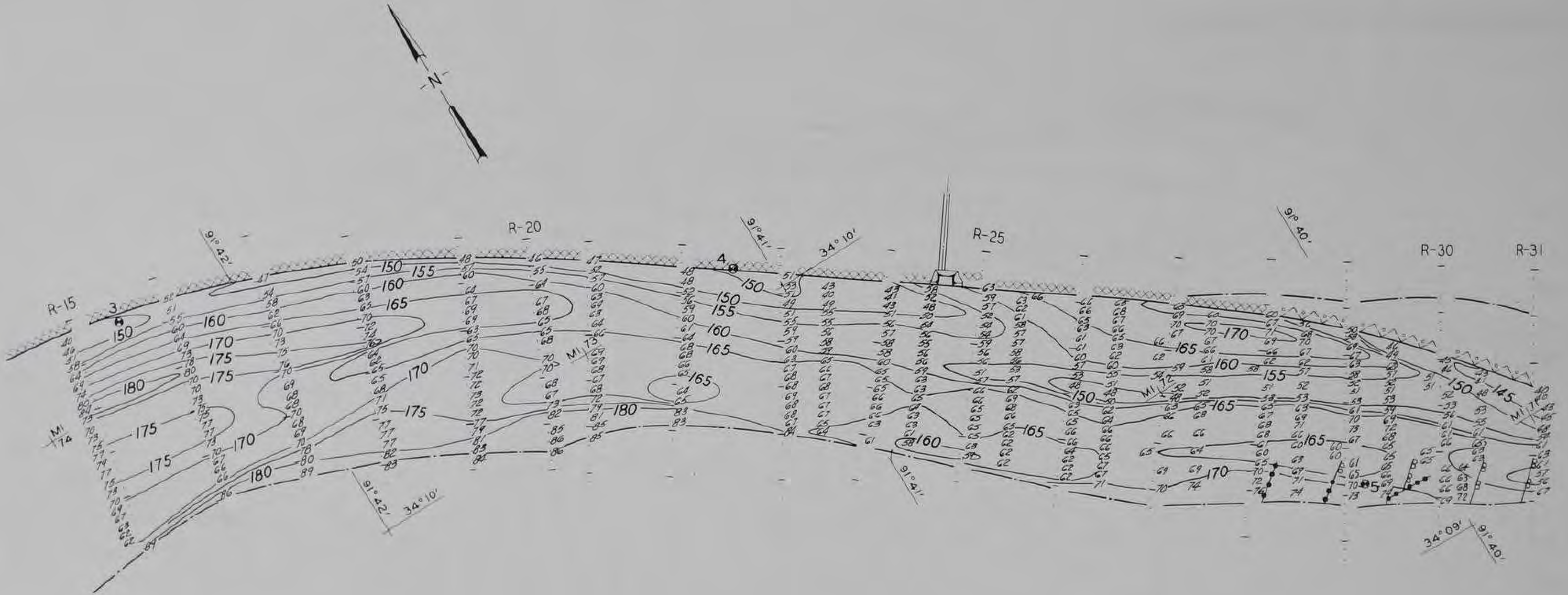
SWELLHEAD TESTS
RAPID-RISE HYDROGRAPH



SWELLHEAD TESTS
RAPID-RISE RATING CURVE
MILE 63.5

PLATE 48


LEGEND
R-5 TEMPLET AND SOUNDING RANGE
4 MODEL GAGE LOCATIONS
MLW 165 MLW IN FT MSL
TOP BANK (APPROX)
PILE DIKE WITH STONE FILL
PROPOSED STONE-FILL REVETMENT
PROPOSED TRENCH-FILL REVETMENT
PROPOSED PILE REVETMENT
PROPOSED PILE DIKE
MODEL LIMITS
MODEL LIMITS
MODEL LIMITS
SEE INSERT A FOR DETAILS OF NAVIGATION PASS
CHANNEL CONTROL LINE
MATCH LINE
MATCH LINE
VELOCITY STATIONS
PLAN
STA 72+67
STA 72+33
STA 73+60
STA 71+40
SECTION A-A
INSERT A
SCALE IN FEET
NOTE: SOUNDINGS AND CONTOURS REFERRED TO MEAN SEA LEVEL
GAGE LOCATIONS
SCALES IN FEET
PROTOTYPE
MODEL



ABOVE PROJECT DEPTH

BELOW PROJECT DEPTH

R-15 TEMPLET AND SOUNDING RANGE

4 MODEL GAGE LOCATIONS

MLW / 165 MLW IN FT MSL

TOP BANK (APPROX)

PROPOSED TRENCH-FILL REVETMENT

PILE DIKE WITH STONE-FILL

PROPOSED PILE DIKE

CHANNEL CONFIGURATION

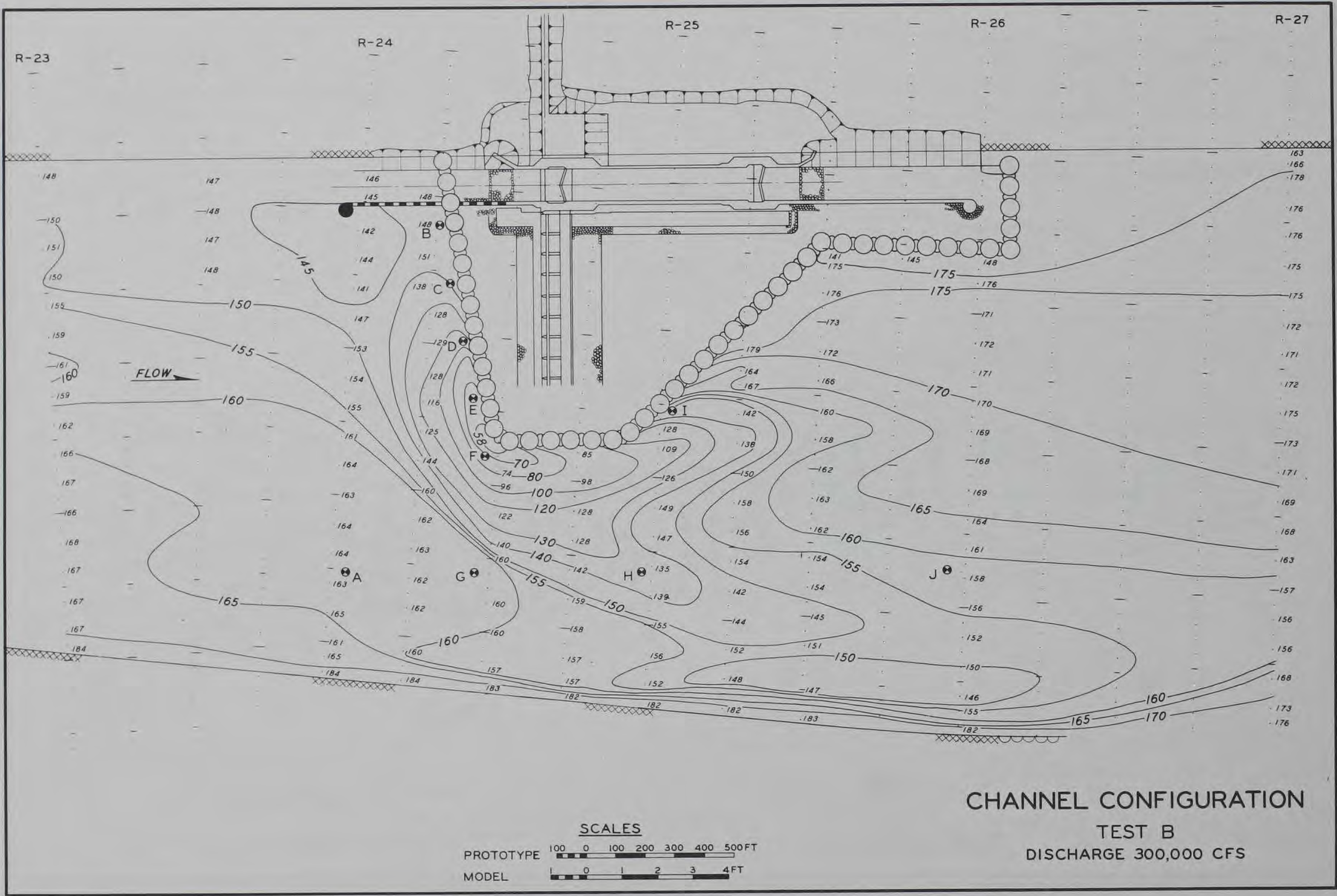
TEST A

DISCHARGE 300,000 CFS

SCALES IN FEET

PROTOTYPE 1000 0 1000 2000

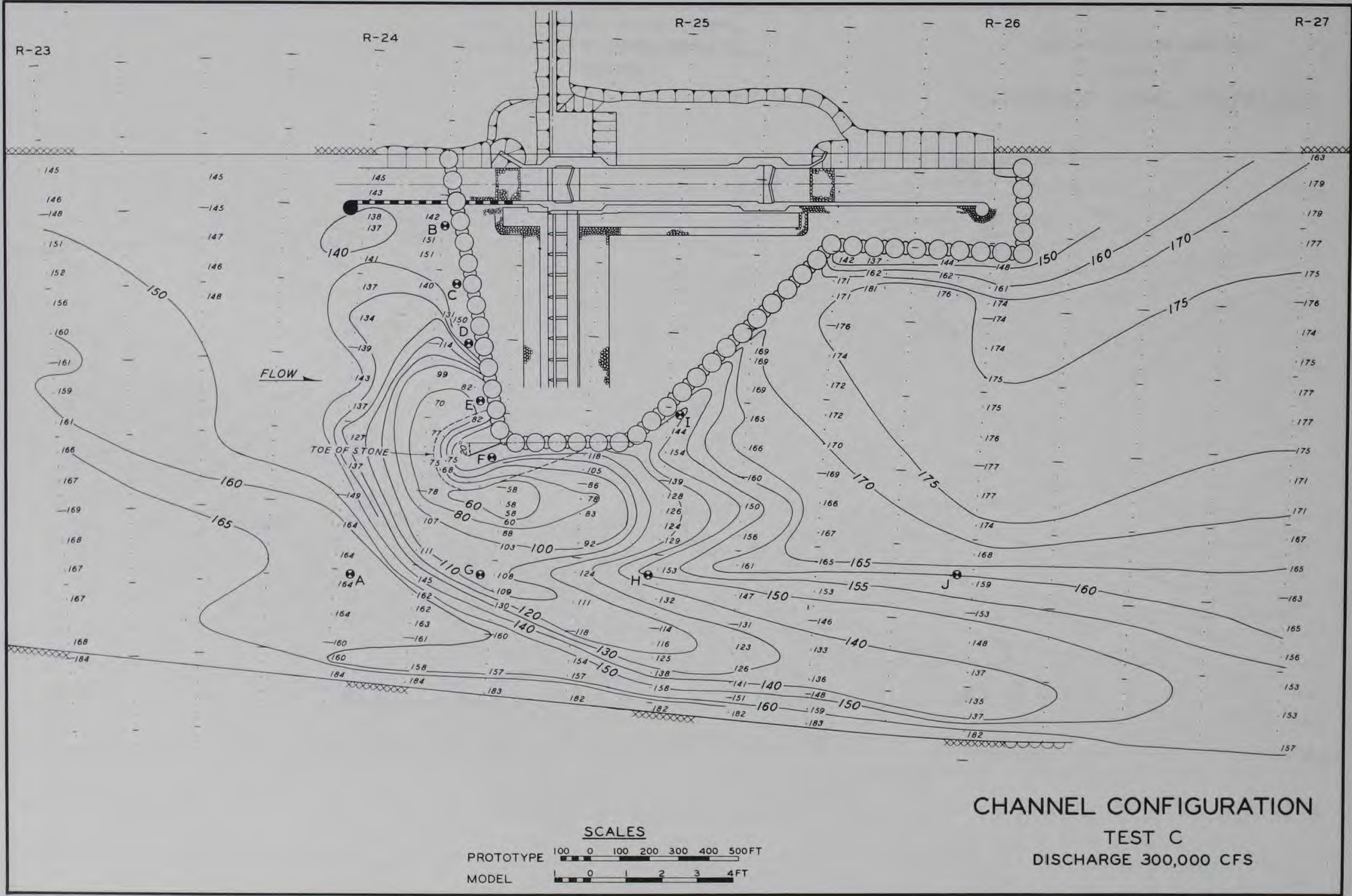
MODEL 5 0 5 10 15



CHANNEL CONFIGURATION

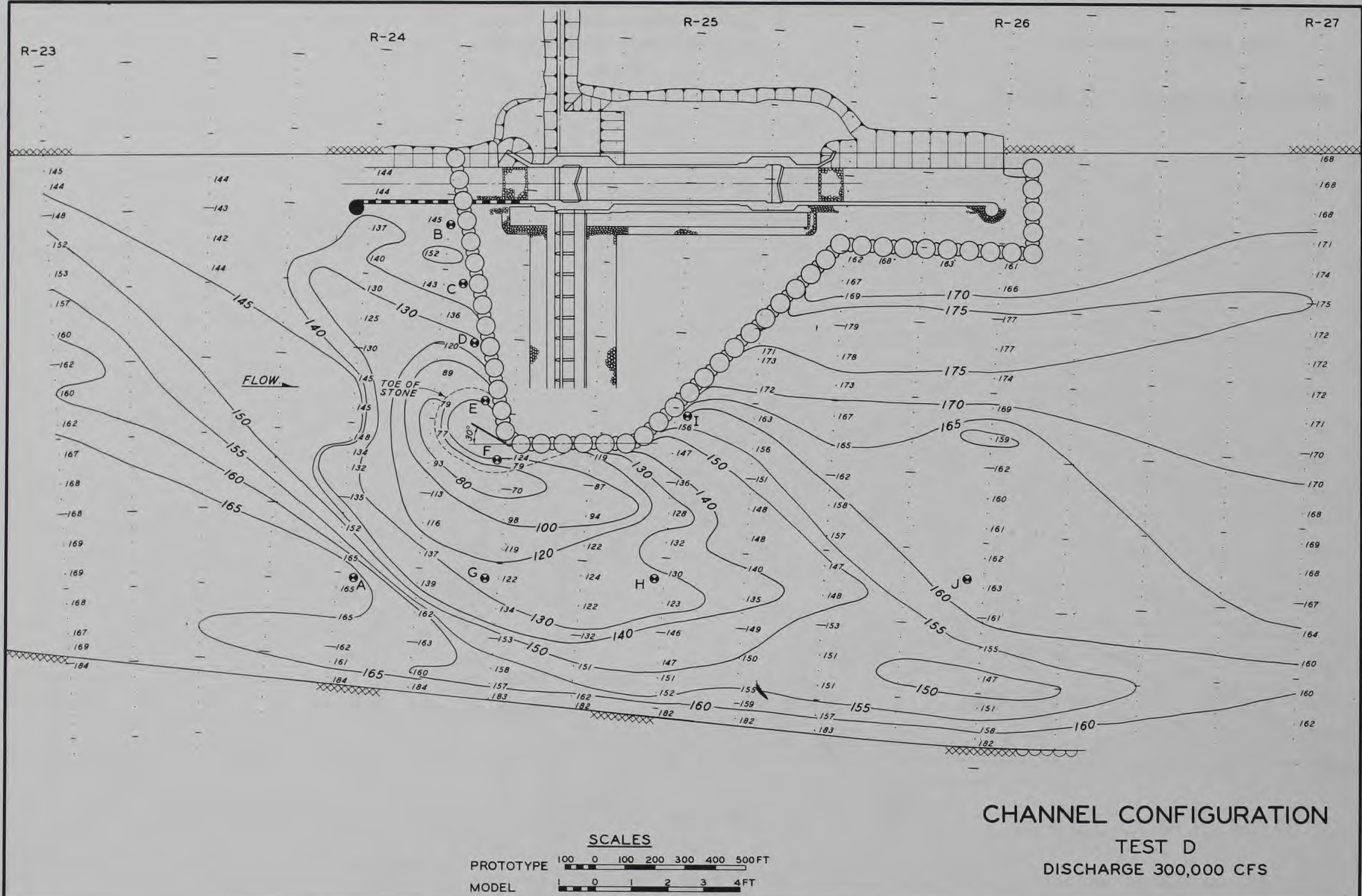
TEST B

DISCHARGE 300,000 CFS

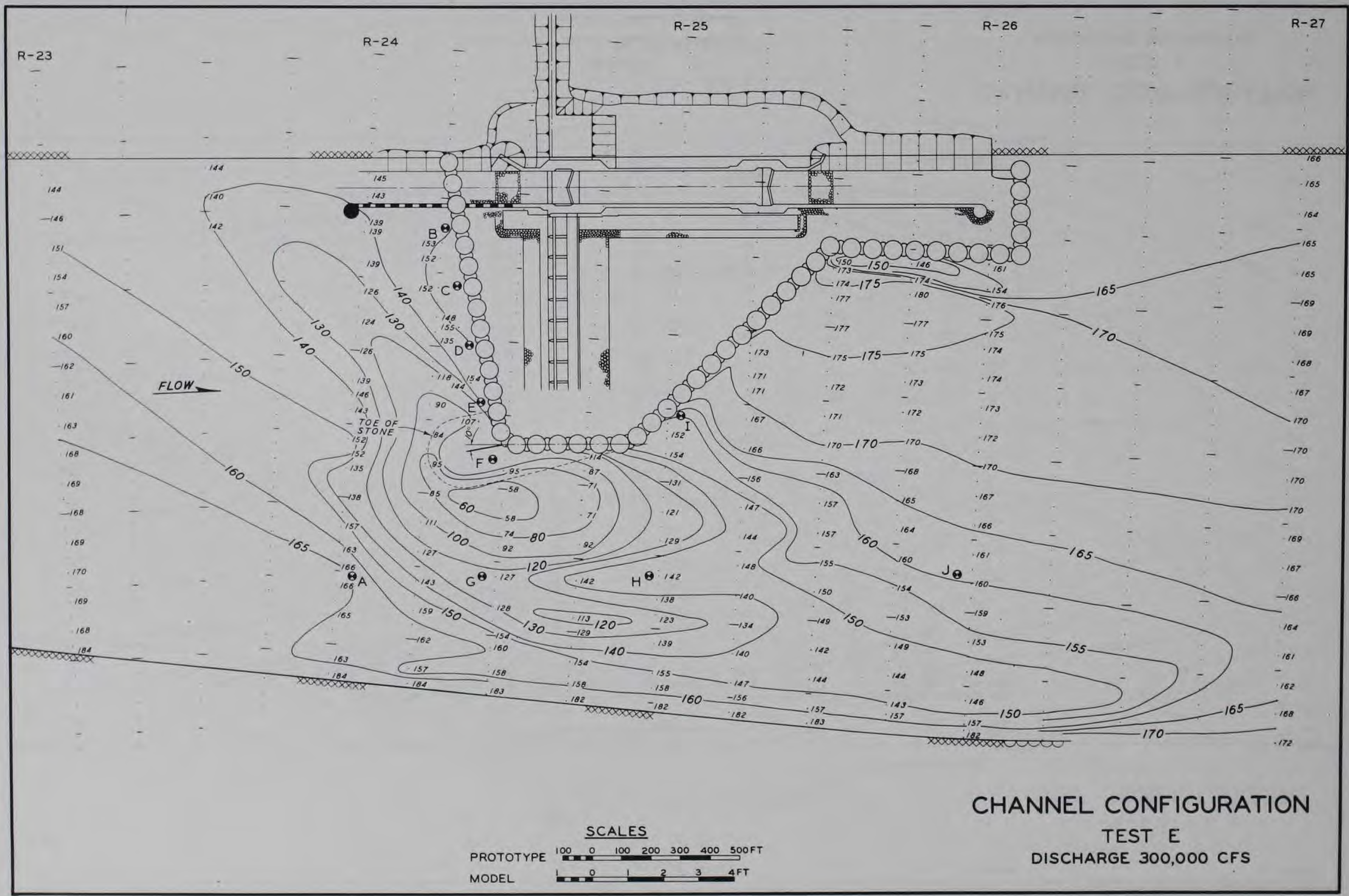


CHANNEL CONFIGURATION
TEST C
DISCHARGE 300,000 CFS

PLATE 51



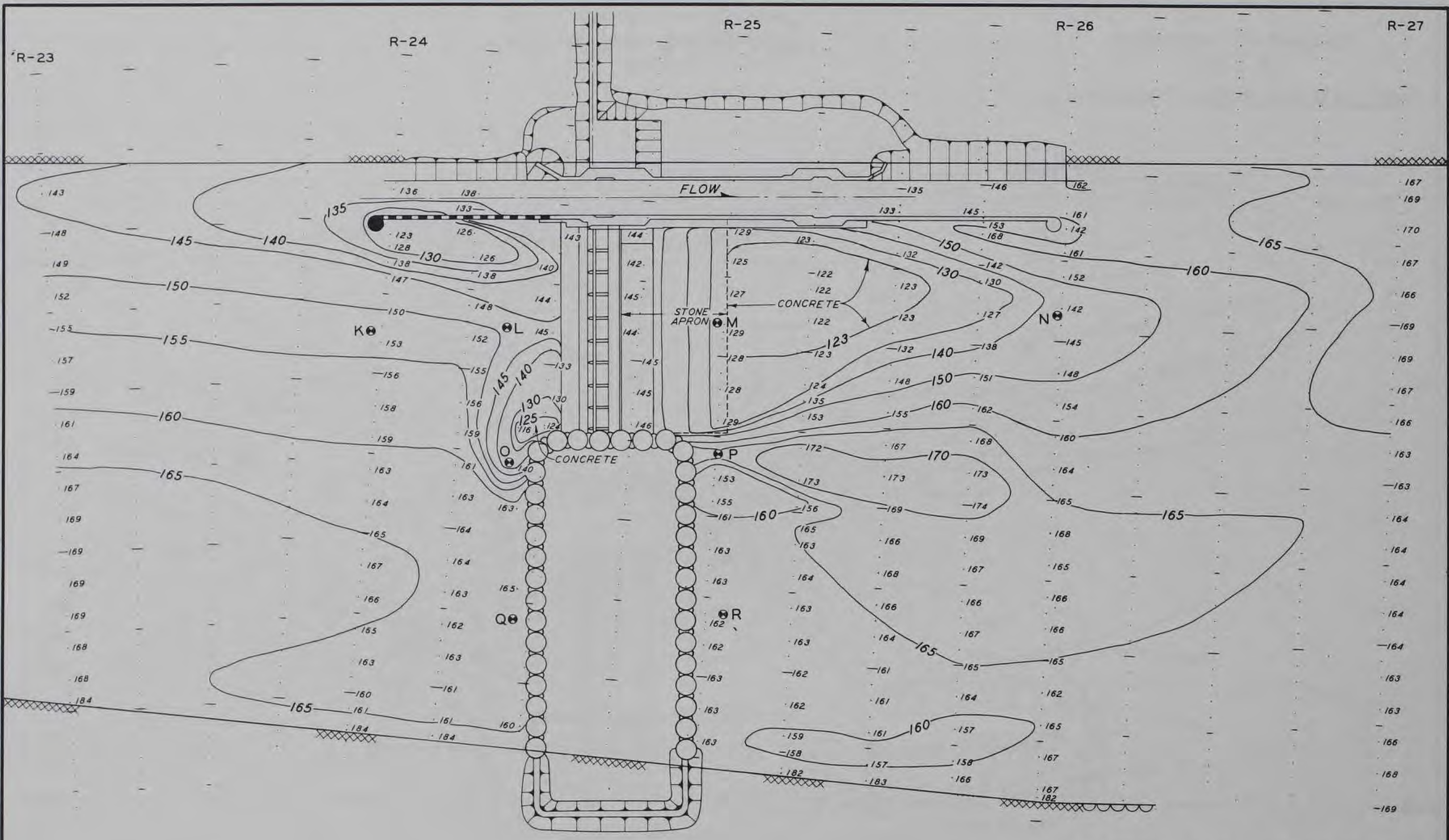
CHANNEL CONFIGURATION
TEST D
DISCHARGE 300,000 CFS



CHANNEL CONFIGURATION

TEST E

DISCHARGE 300,000 CFS



SCALES

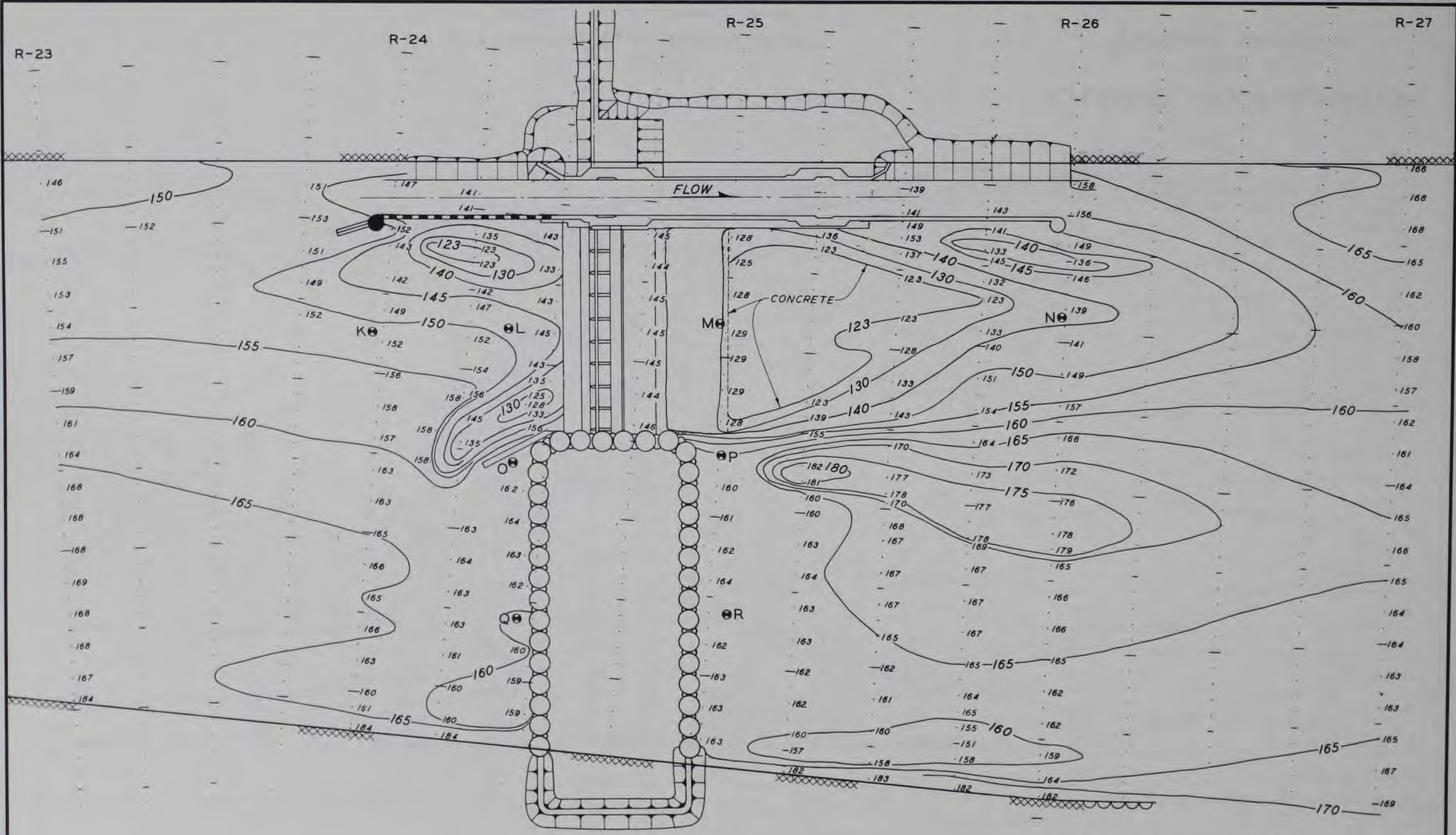
PROTOTYPE 100 0 100 200 300 400 500 FT

MODEL 1 0 1 2 3 4 FT

CHANNEL CONFIGURATION

TEST F

DISCHARGE 300,000 CFS



SCALES

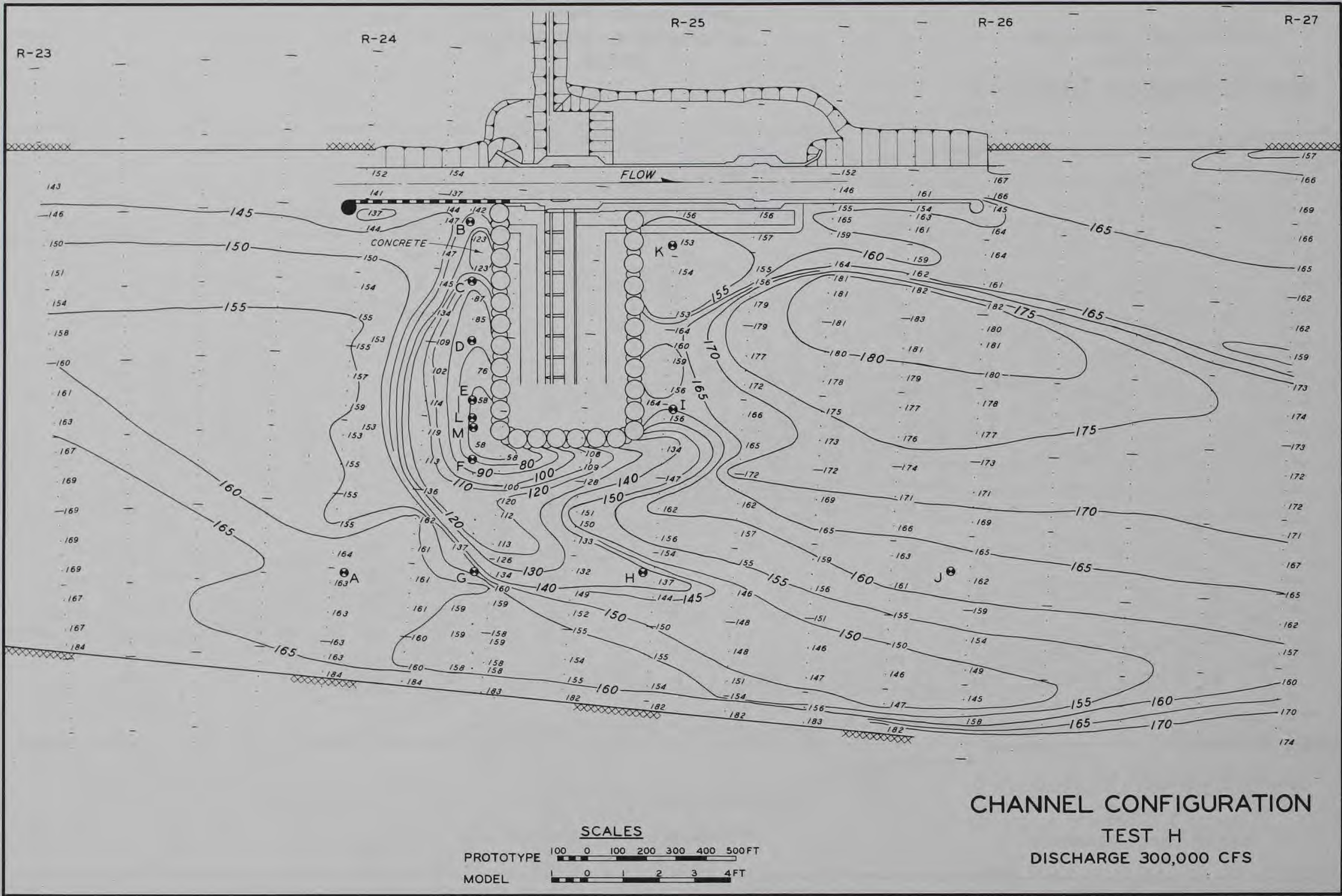
PROTOTYPE 100 0 100 200 300 400 500 FT

MODEL 1 0 1 2 3 4 FT

CHANNEL CONFIGURATION

TEST G

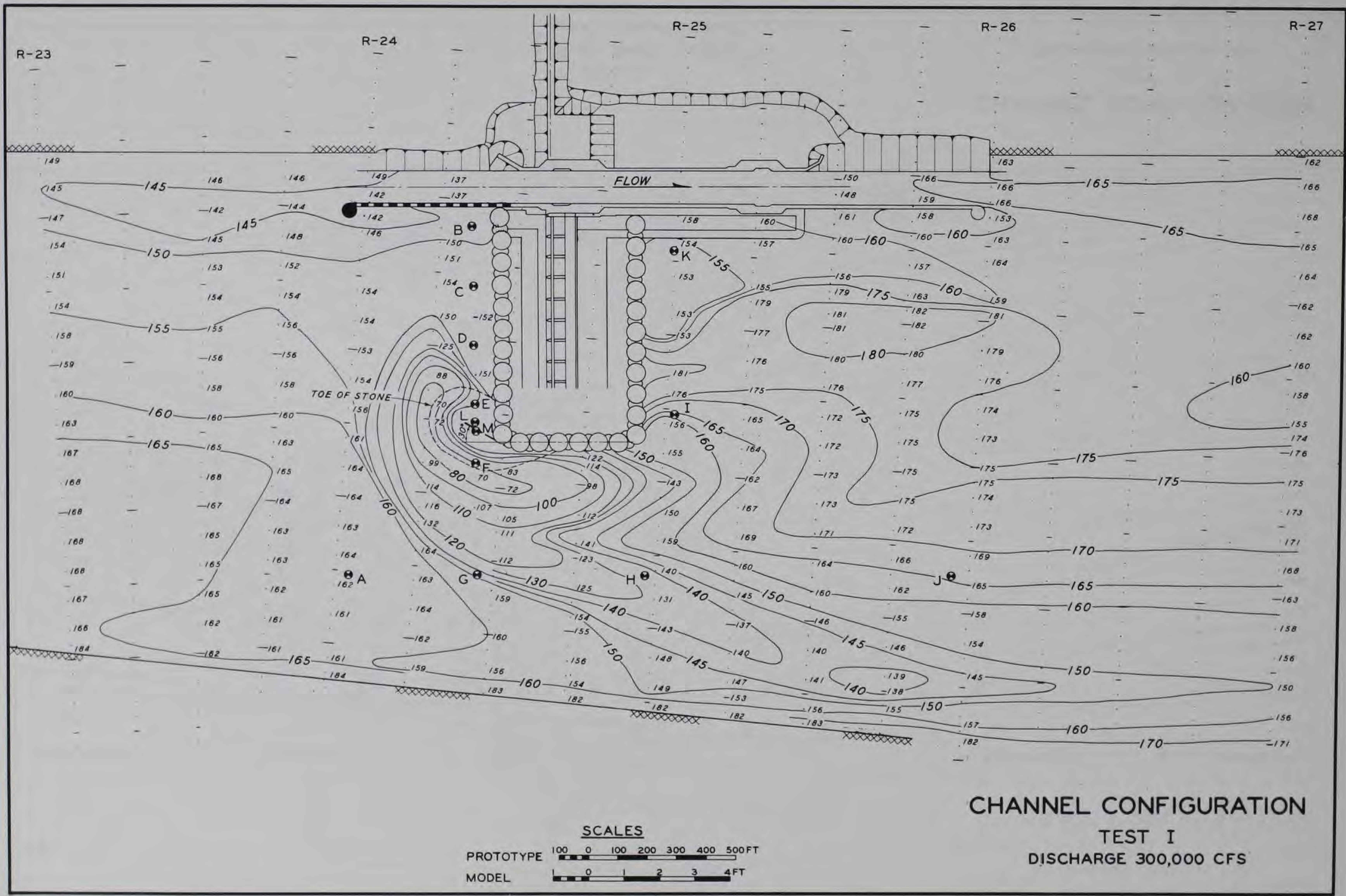
DISCHARGE 300,000 CFS



CHANNEL CONFIGURATION

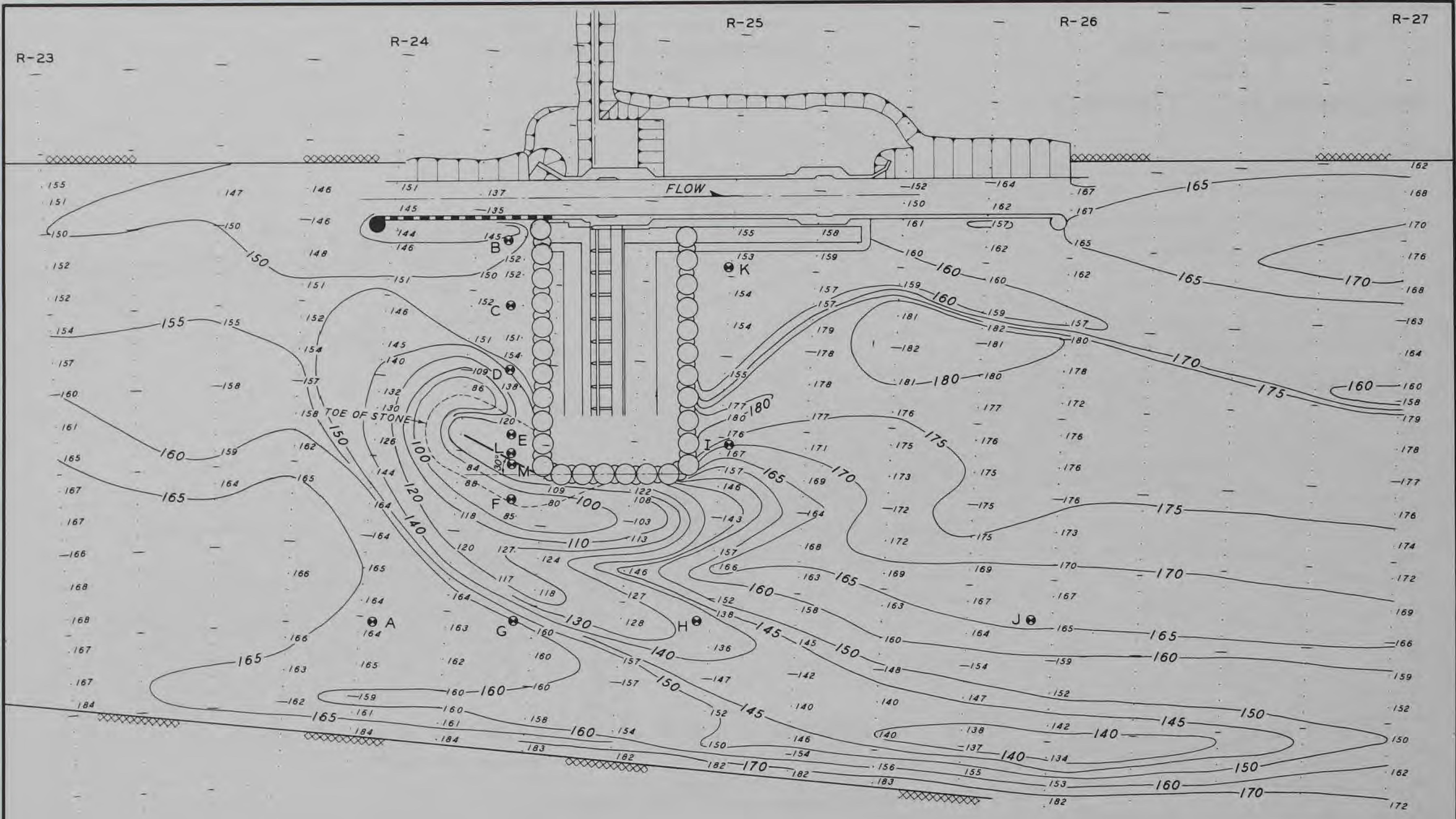
TEST H

DISCHARGE 300,000 CFS



CHANNEL CONFIGURATION
TEST I
DISCHARGE 300,000 CFS

PLATE 57



SCALES

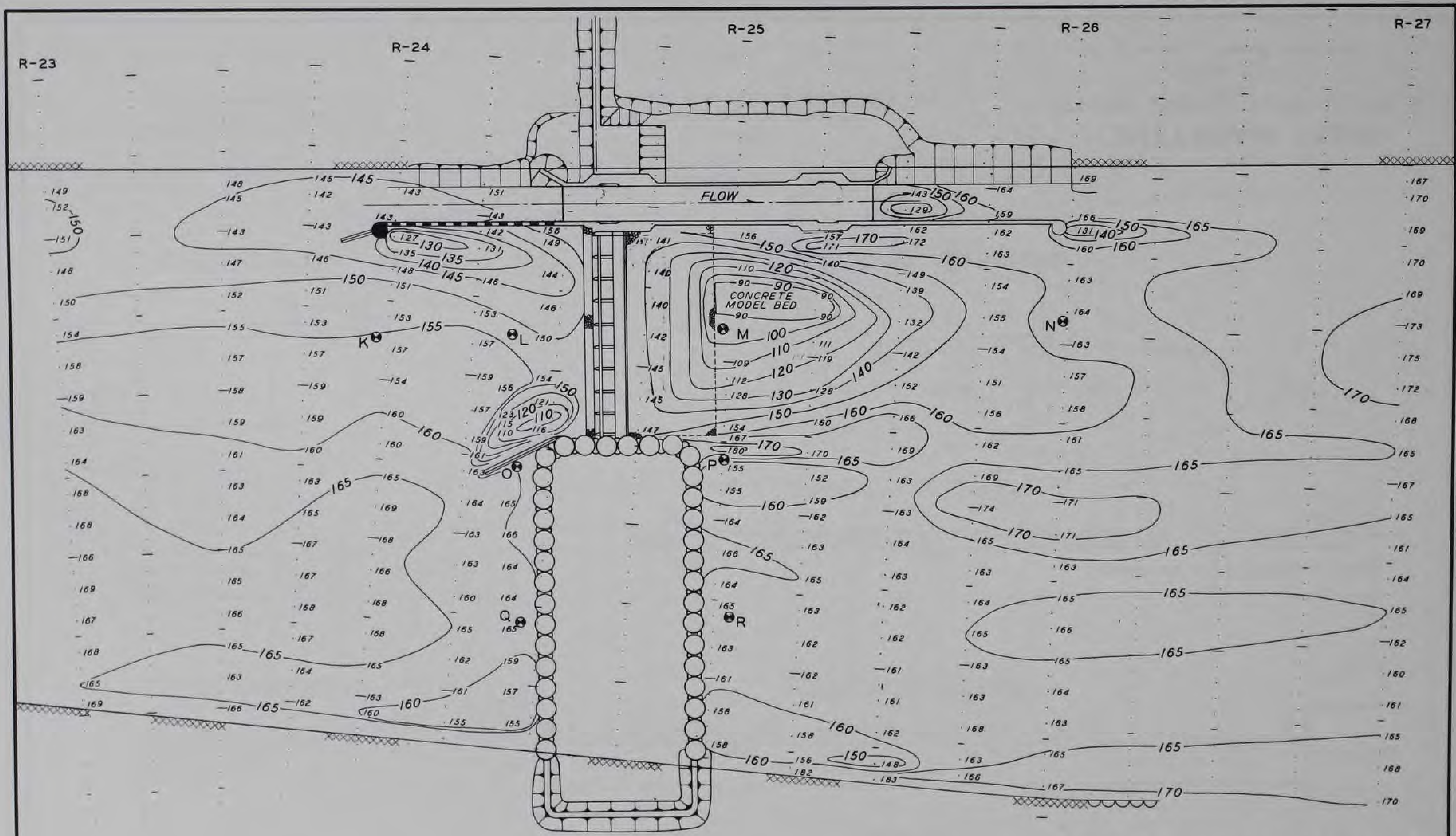
PROTOTYPE 100 0 100 200 300 400 500 FT

MODEL 1 0 1 2 3 4 FT

CHANNEL CONFIGURATION

TEST J

DISCHARGE 300,000 CFS

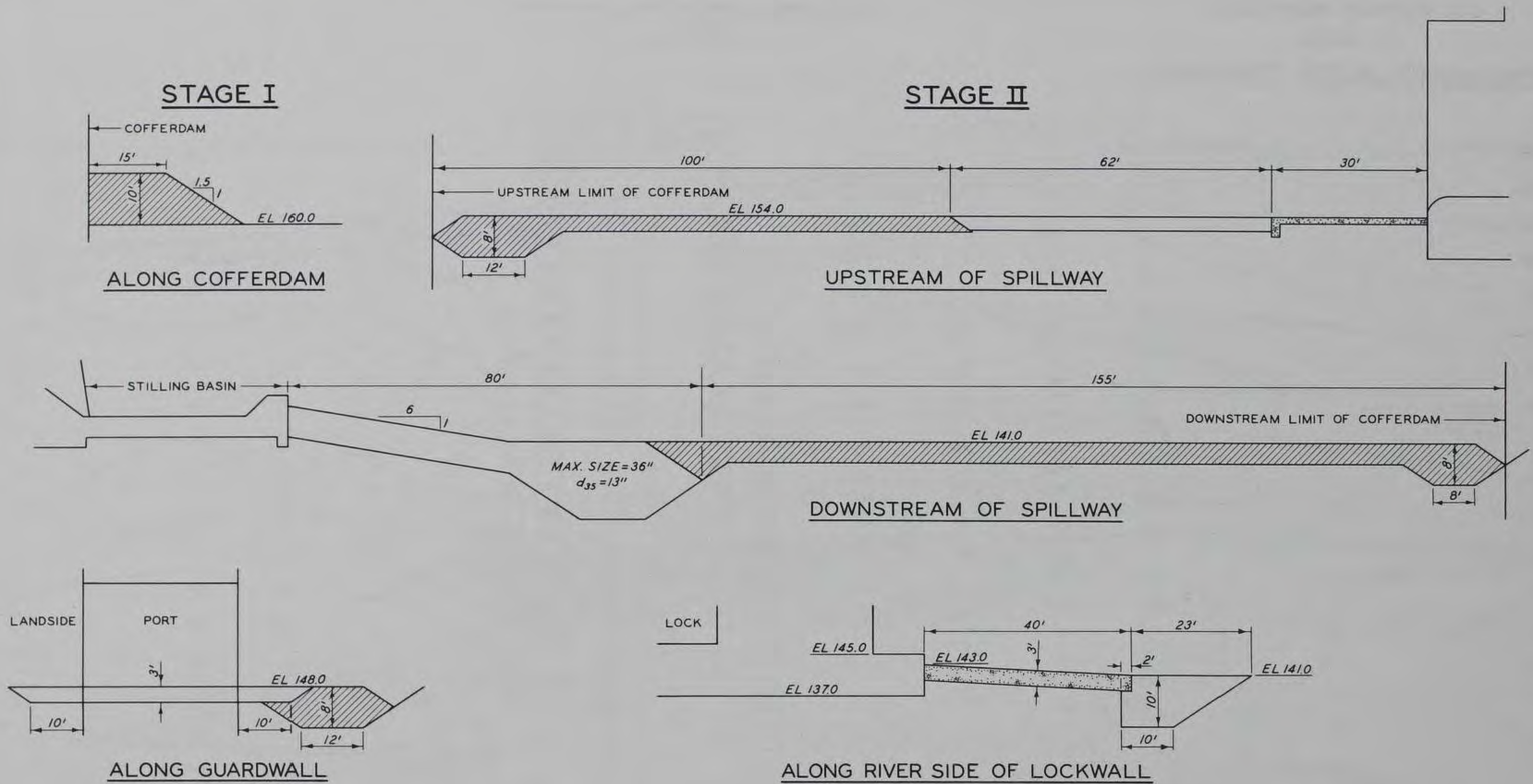


SCALES

PROTOTYPE 100 0 100 200 300 400 500 FT

MODEL 1 0 1 2 3 4 FT

CHANNEL CONFIGURATION
TEST 1
DISCHARGE 300,000 CFS



NOTE: ALL STONE, EXCEPT WHERE INDICATED OTHERWISE, TO BE A-1 RIPRAP. MAX. SIZE 24", d_{35}=12", MIN. 4"

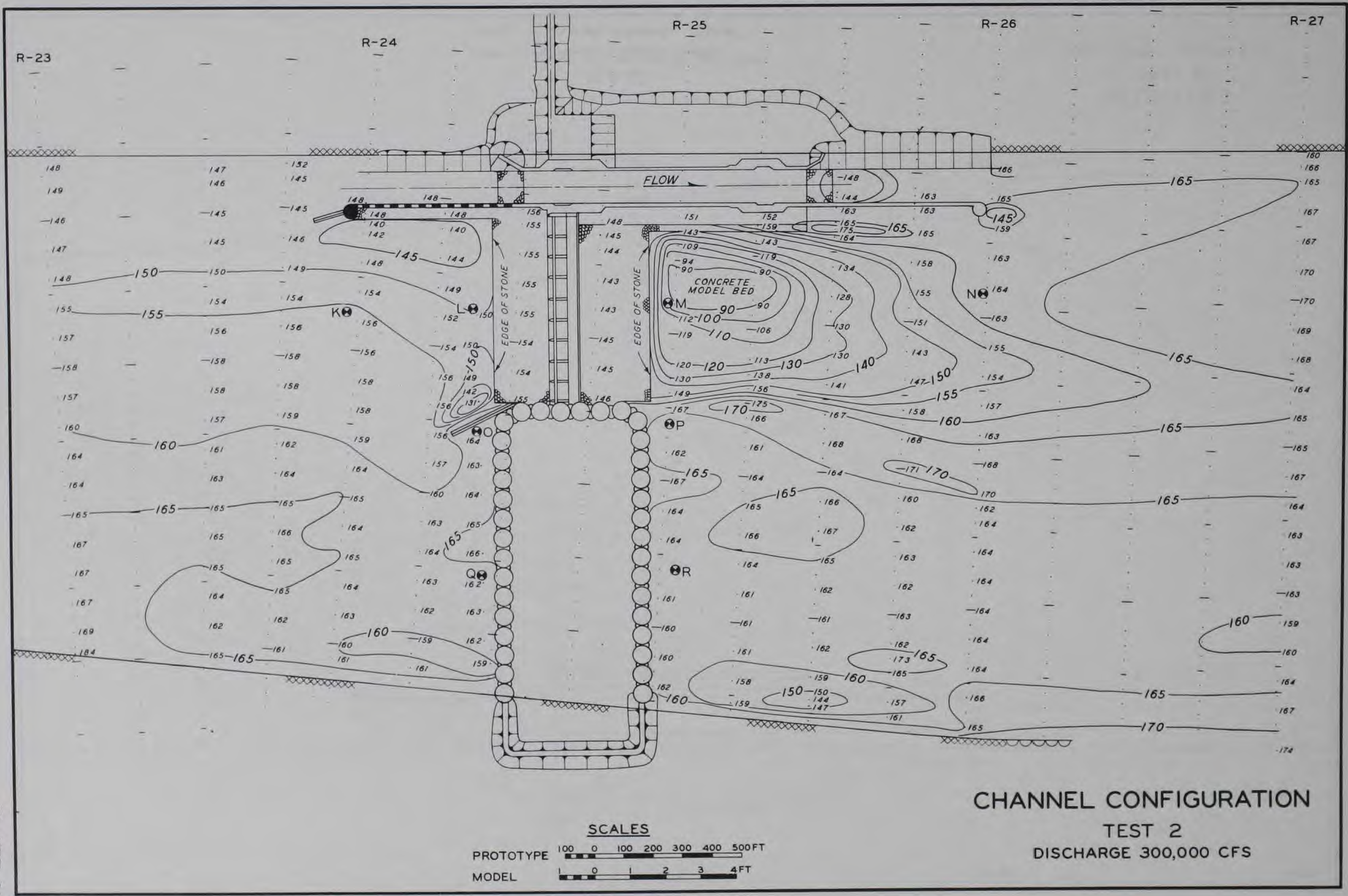
LEGEND

- STONE IN BASIC DESIGN FOR OPERATION
- ADDITIONAL STONE REQUIRED FOR DIVERSION, PLAN A

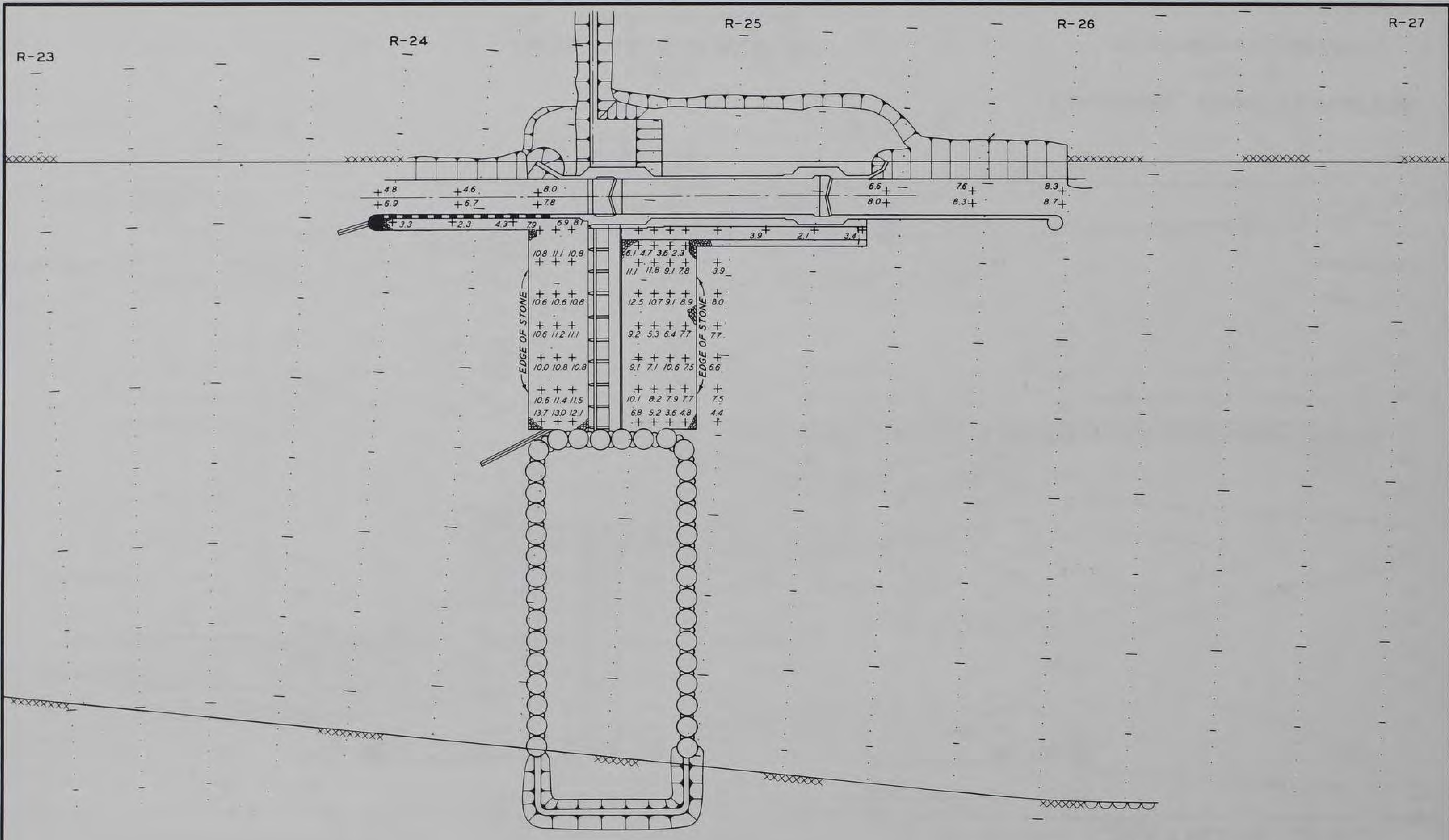
COFFERDAM TESTS
STONE PROTECTION, PLAN A

SCALE IN FEET

10 0 10 20 30 40



CHANNEL CONFIGURATION
TEST 2
DISCHARGE 300,000 CFS



SCALES

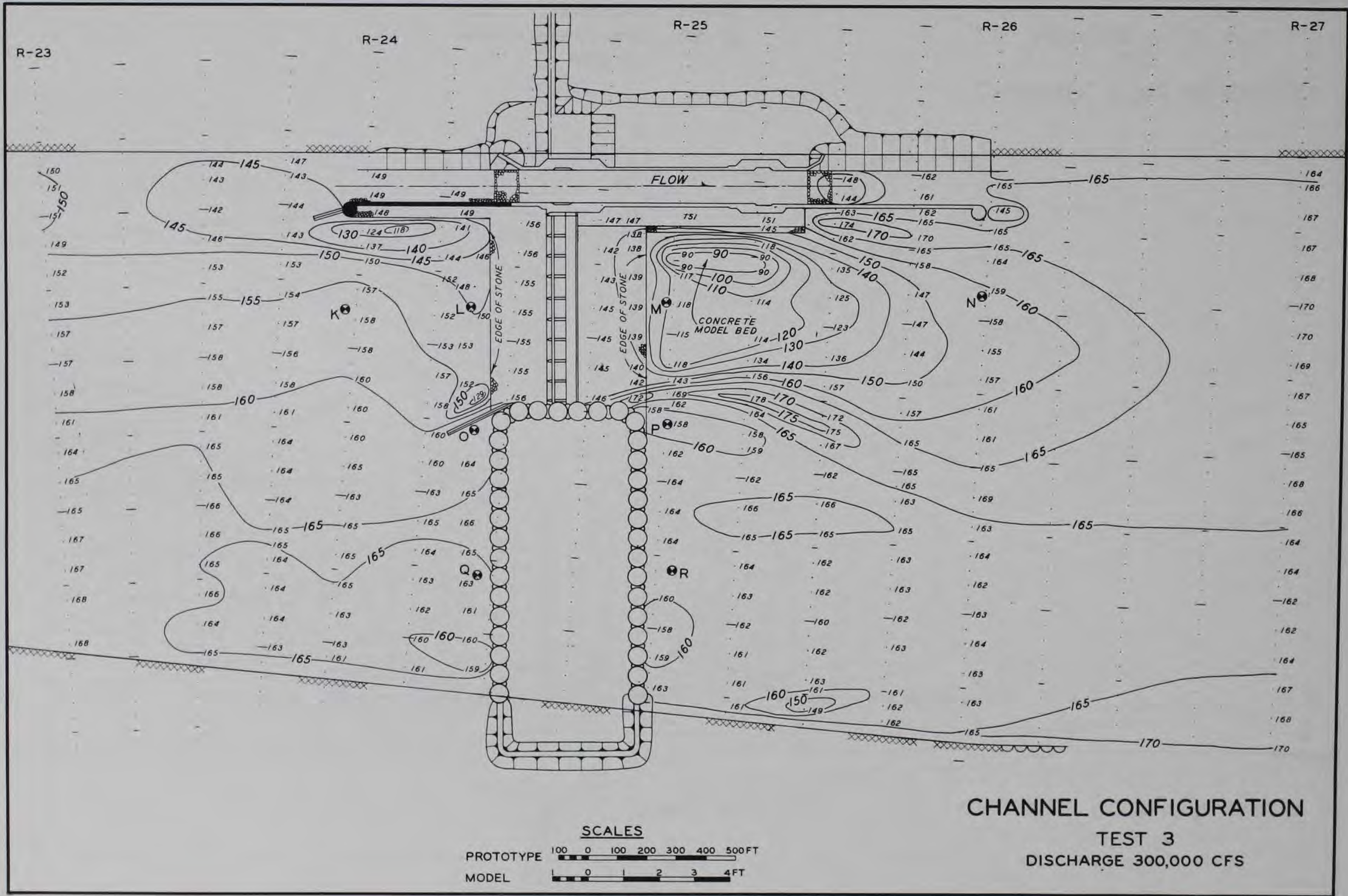
PROTOTYPE 100 0 100 200 300 400 500 FT

MODEL 1 0 1 2 3 4 FT

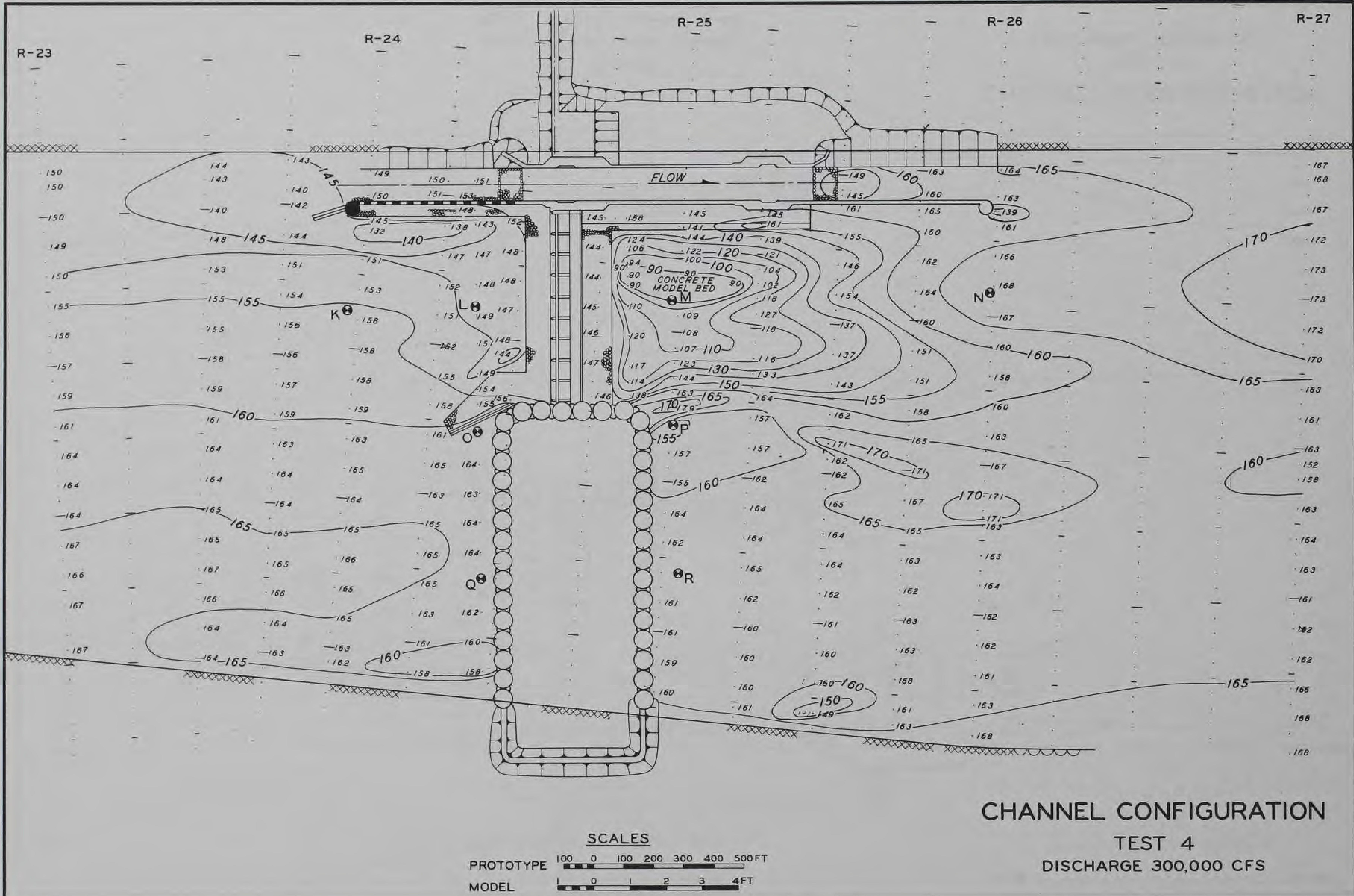
VELOCITIES

TEST 2

DISCHARGE 300,000 CFS



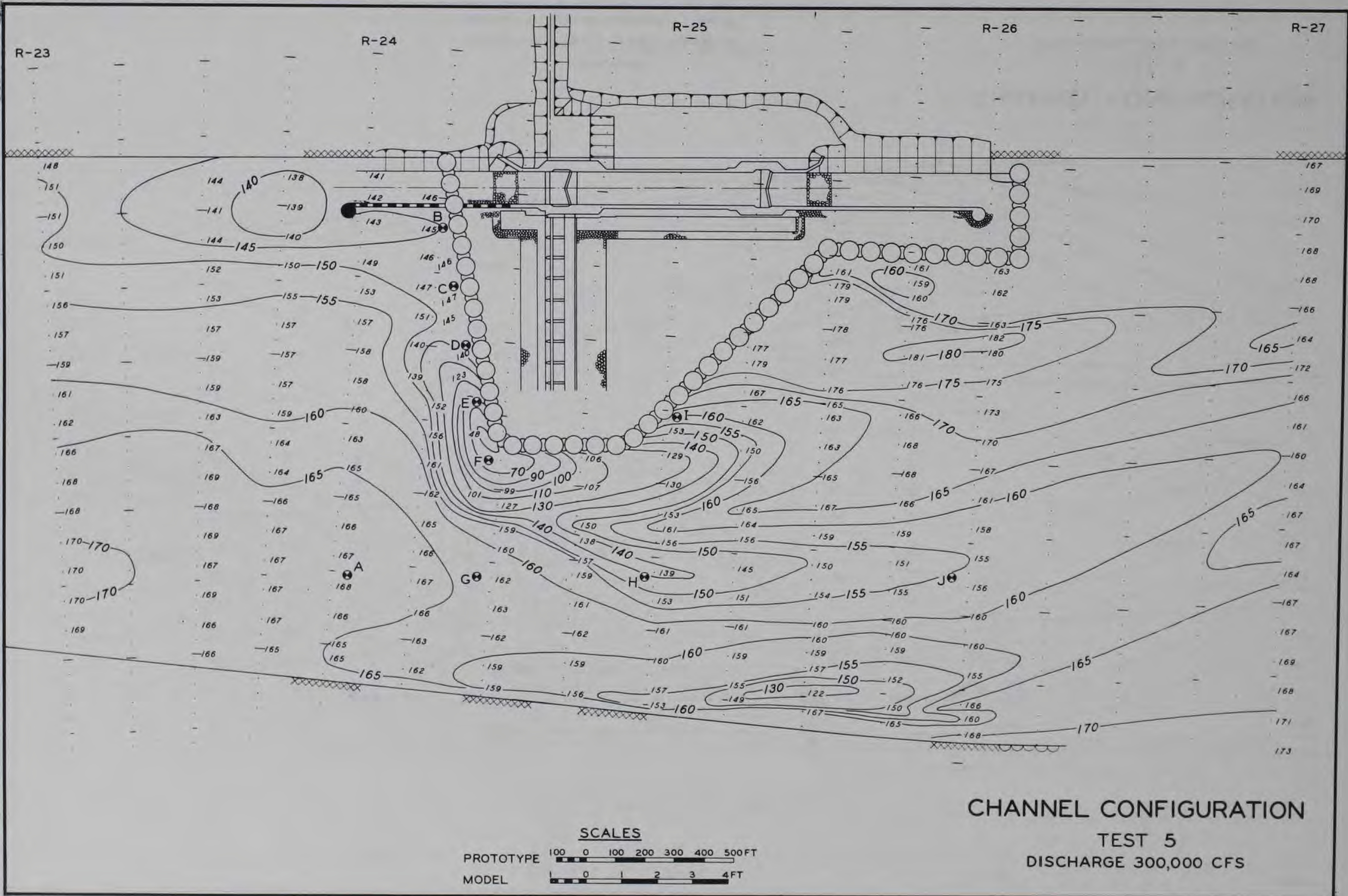
CHANNEL CONFIGURATION
TEST 3
DISCHARGE 300,000 CFS



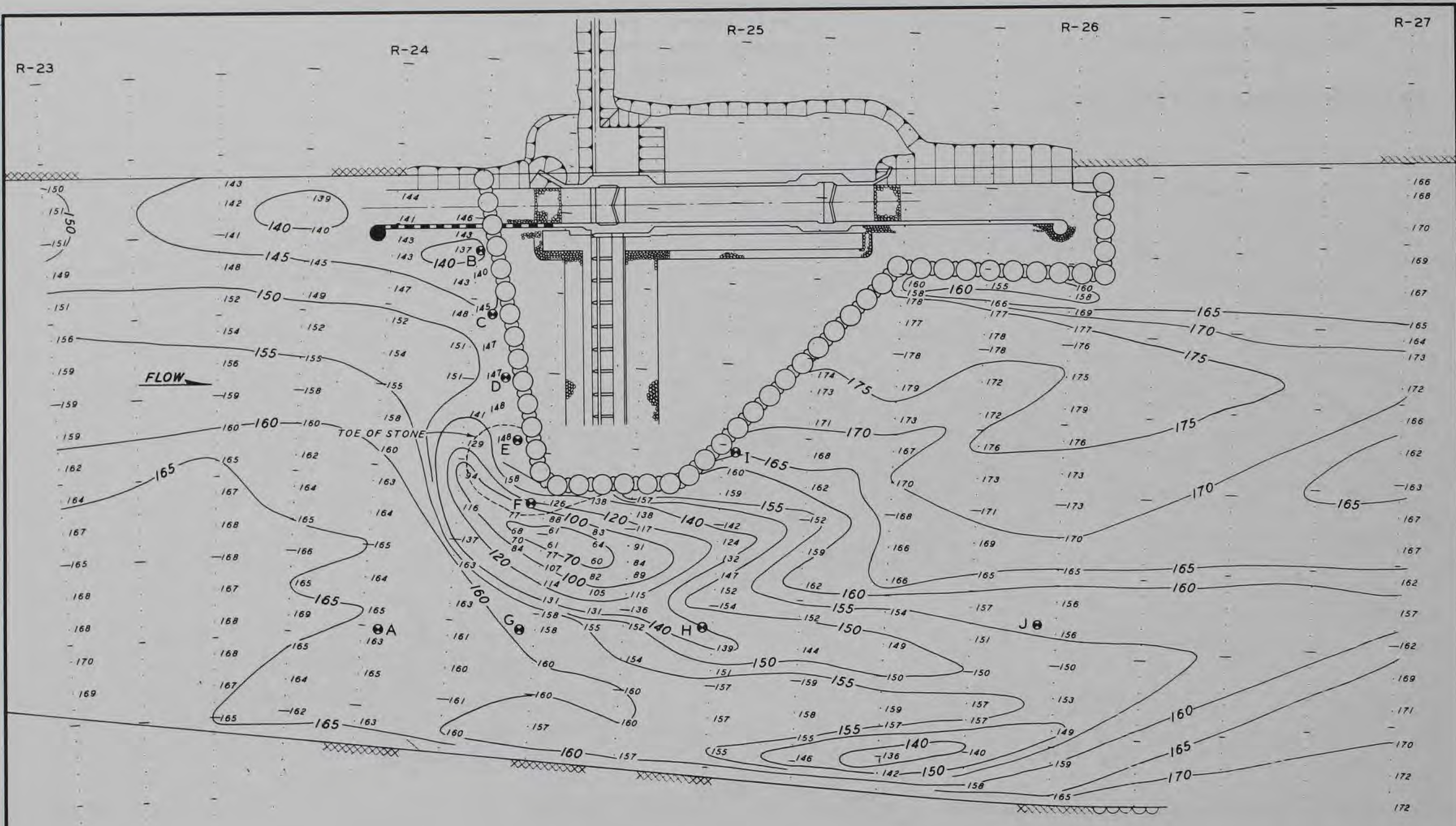
CHANNEL CONFIGURATION

TEST 4

DISCHARGE 300,000 CFS



CHANNEL CONFIGURATION
TEST 5
DISCHARGE 300,000 CFS



SCALES

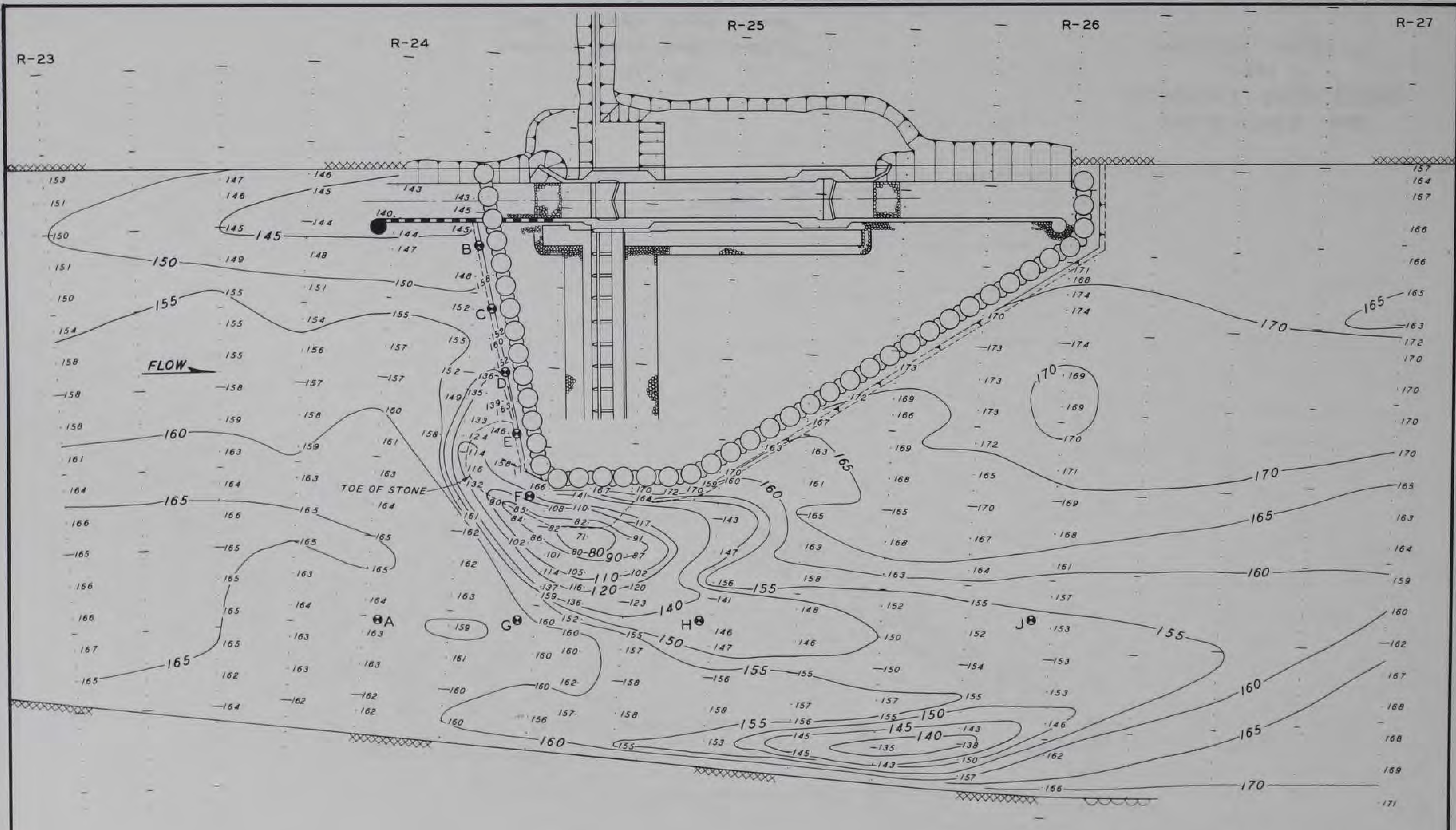
PROTOTYPE 100 0 100 200 300 400 500 FT

MODEL 1 0 1 2 3 4 FT

CHANNEL CONFIGURATION

TEST 6

DISCHARGE 300,000 CFS



SCALES

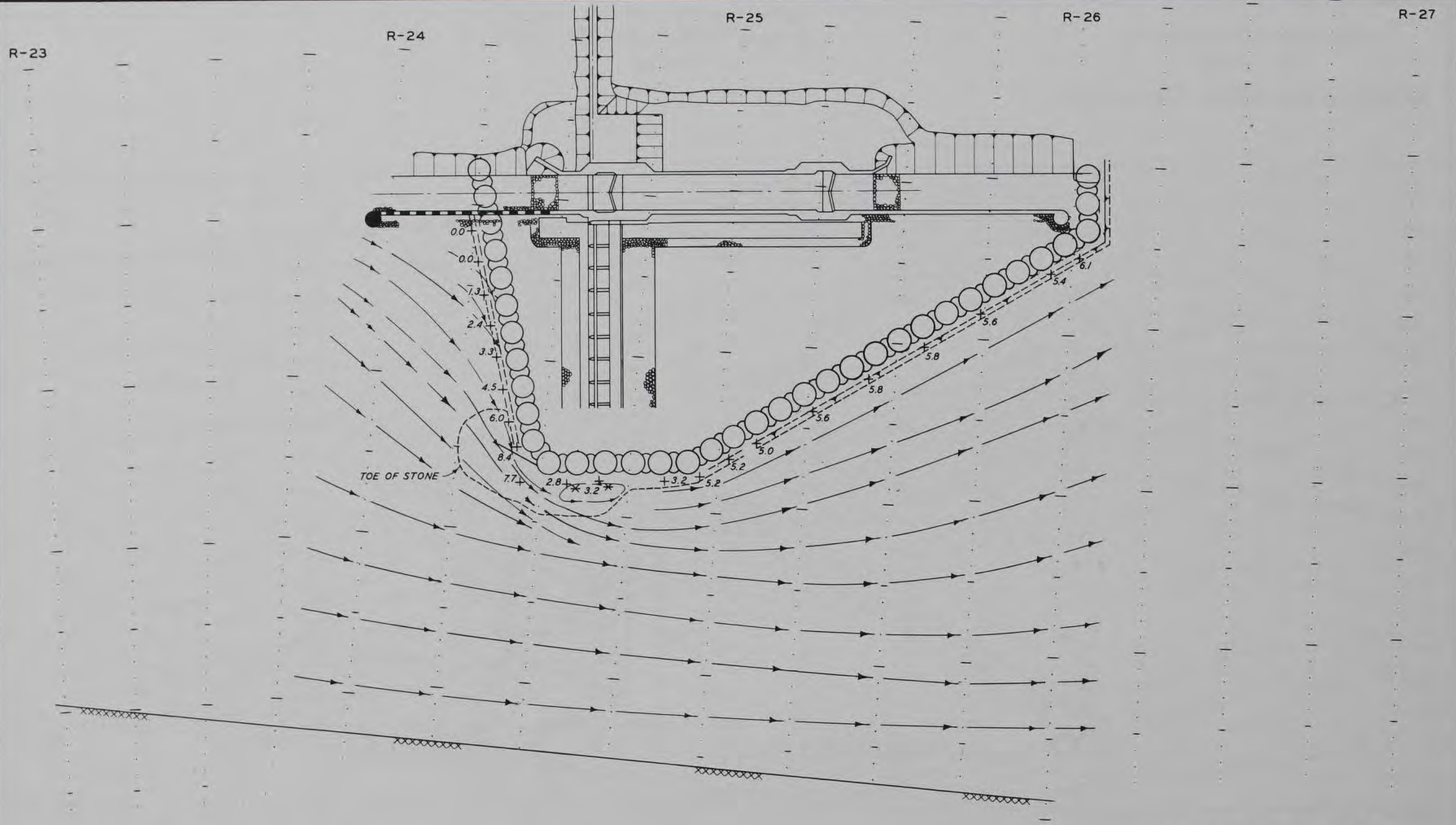
PROTOTYPE 100 0 100 200 300 400 500 FT

MODEL 1 0 1 2 3 4 FT

CHANNEL CONFIGURATION

TEST 7

DISCHARGE 300,000 CFS



1.3 + VELOCITY IN FT PER SEC
* UPSTREAM VELOCITY

SCALES

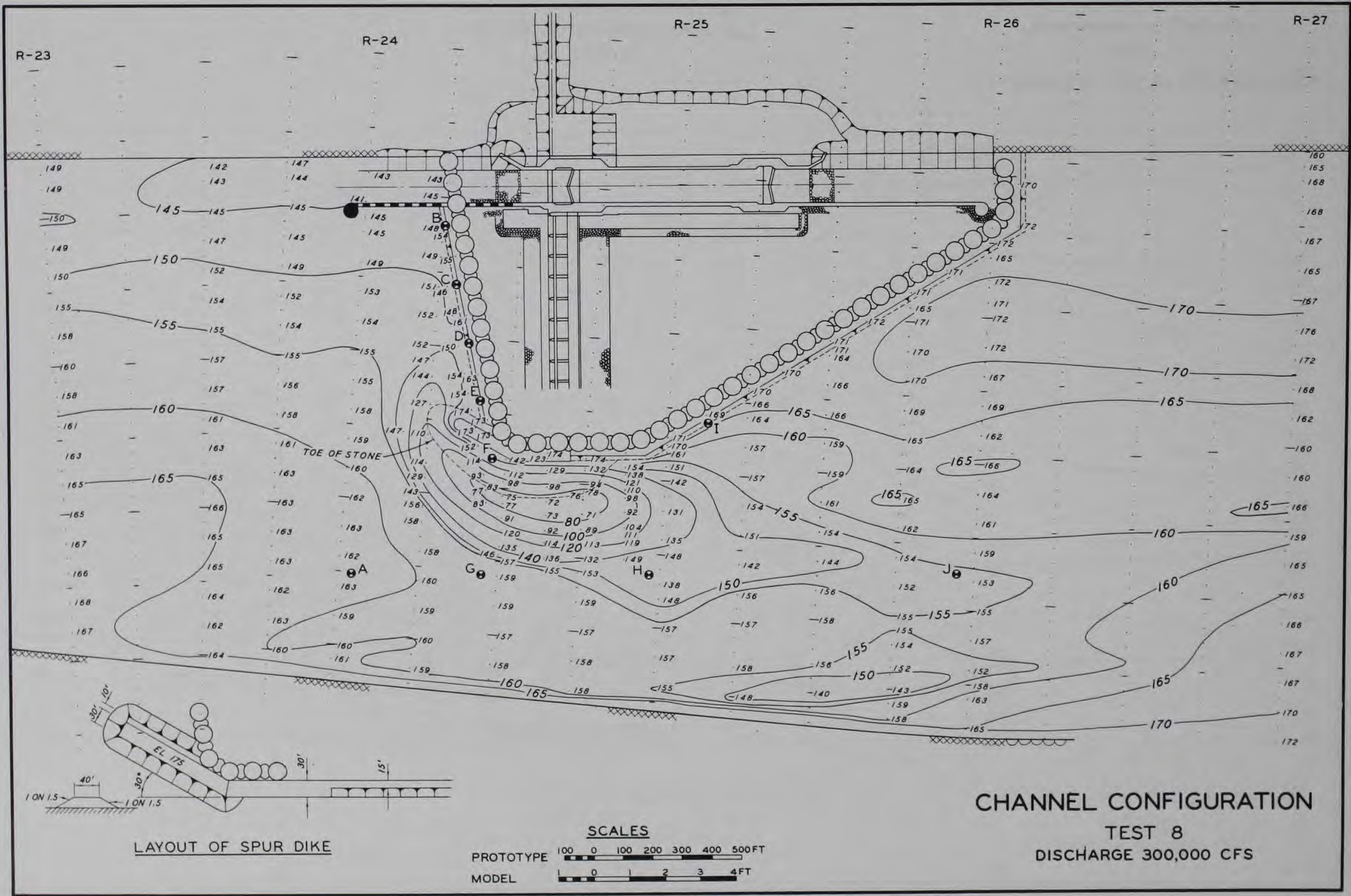
PROTOTYPE 100 0 100 200 300 400 500 FT

MODEL 1 0 1 2 3 4 FT

VELOCITIES AND CURRENT DIRECTIONS

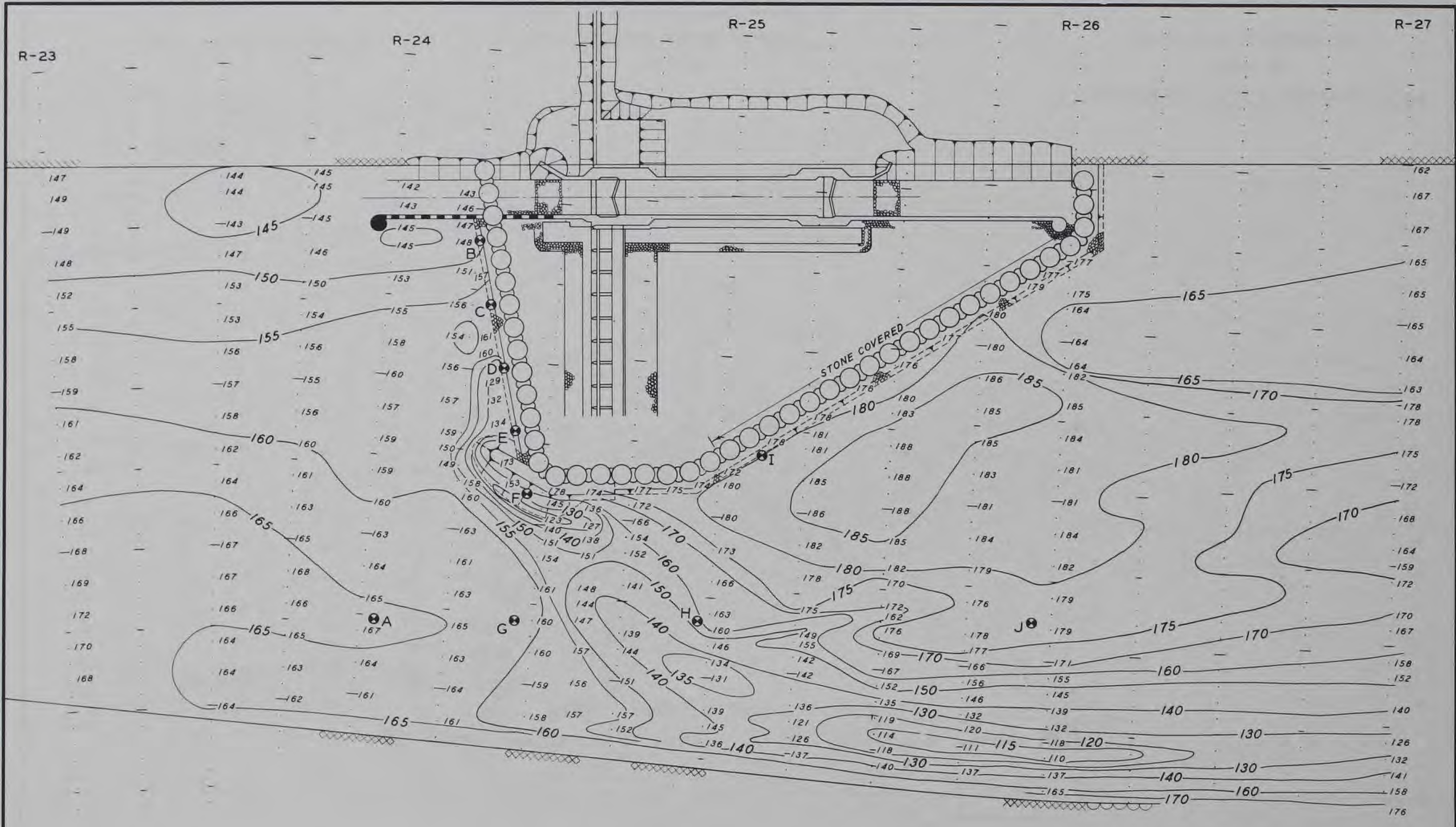
TEST 7

DISCHARGE 300,000 CFS



CHANNEL CONFIGURATION
TEST 8
DISCHARGE 300,000 CFS

PLATE 69



SCALES

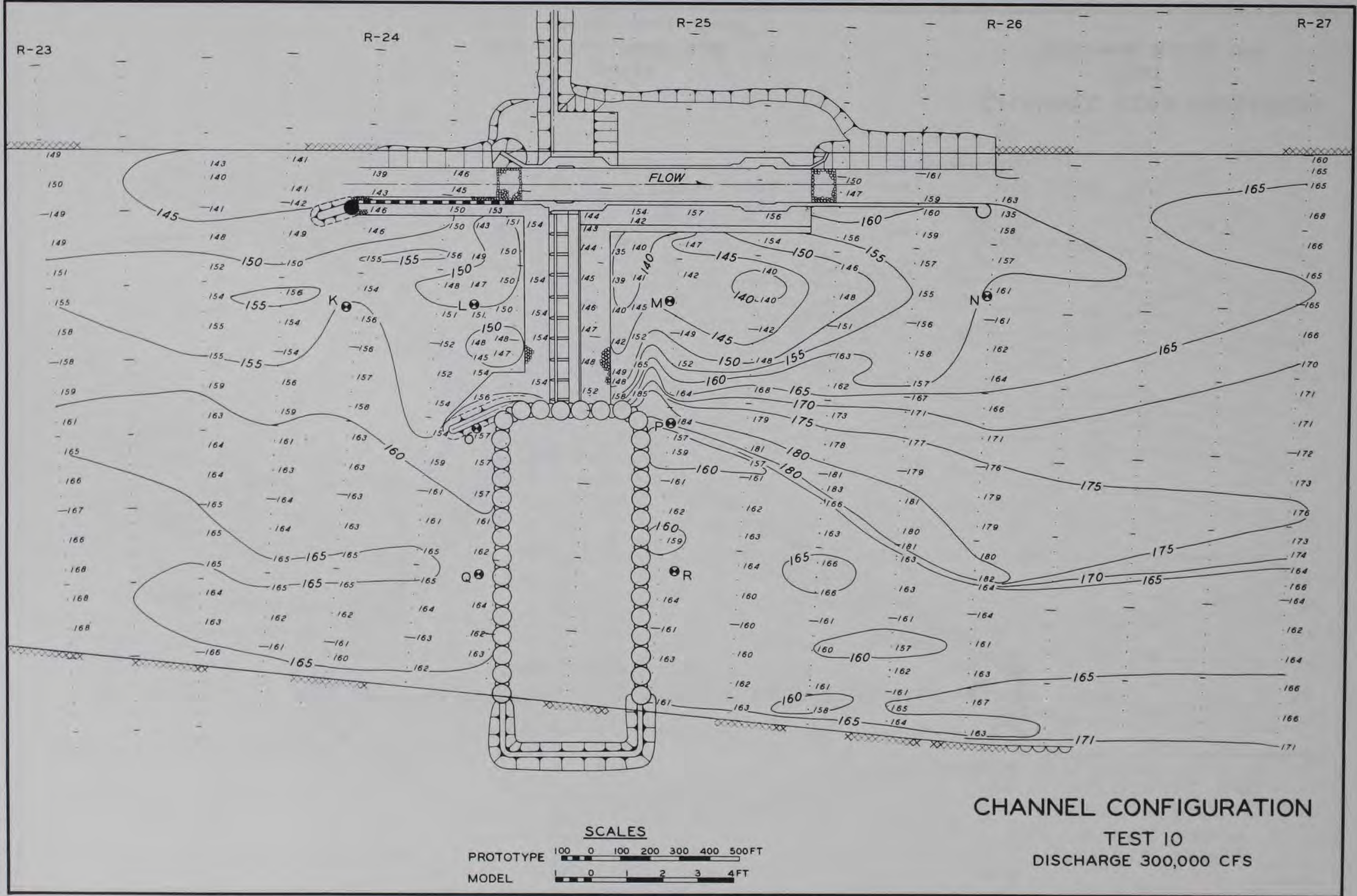
PROTOTYPE 100 0 100 200 300 400 500 FT

MODEL 1 0 1 2 3 4 FT

CHANNEL CONFIGURATION

TEST 9

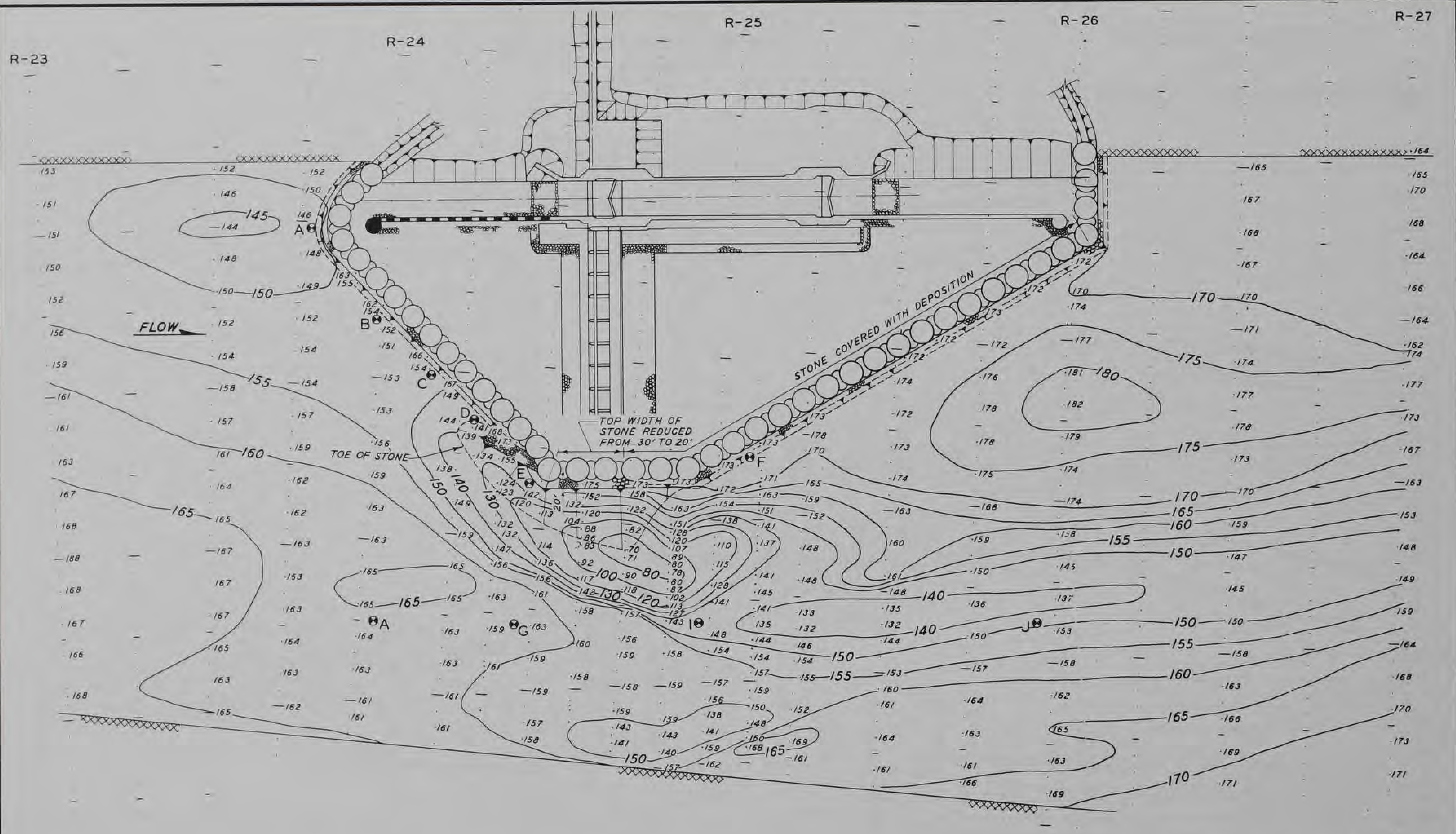
DISCHARGE 300,000 CFS



CHANNEL CONFIGURATION

TEST 10

DISCHARGE 300,000 CFS



SCALES

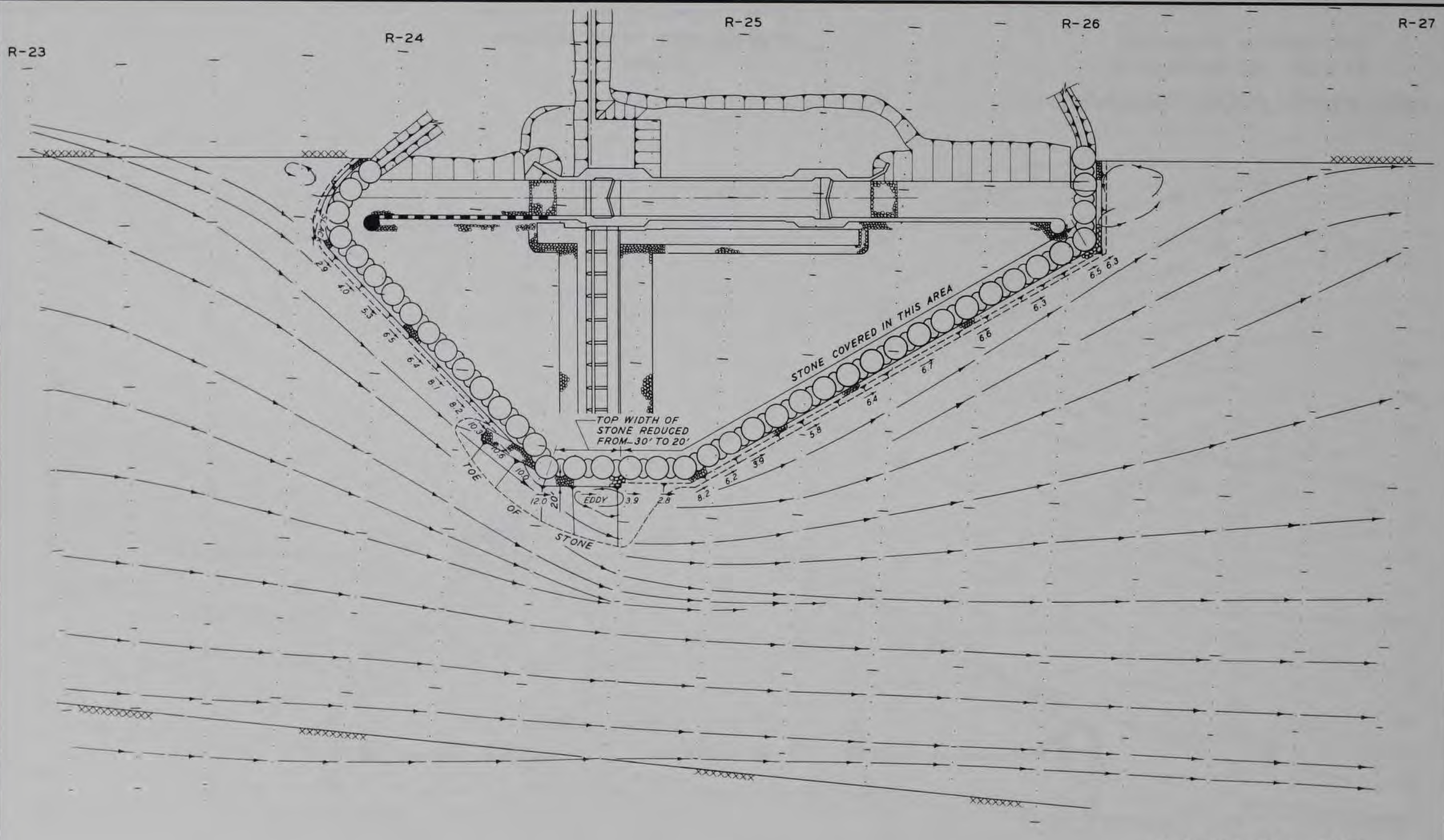
PROTOTYPE 100 0 100 200 300 400 500 FT

MODEL 1 0 1 2 3 4 FT

CHANNEL CONFIGURATION

TEST 11

DISCHARGE 300,000 CFS



SCALES

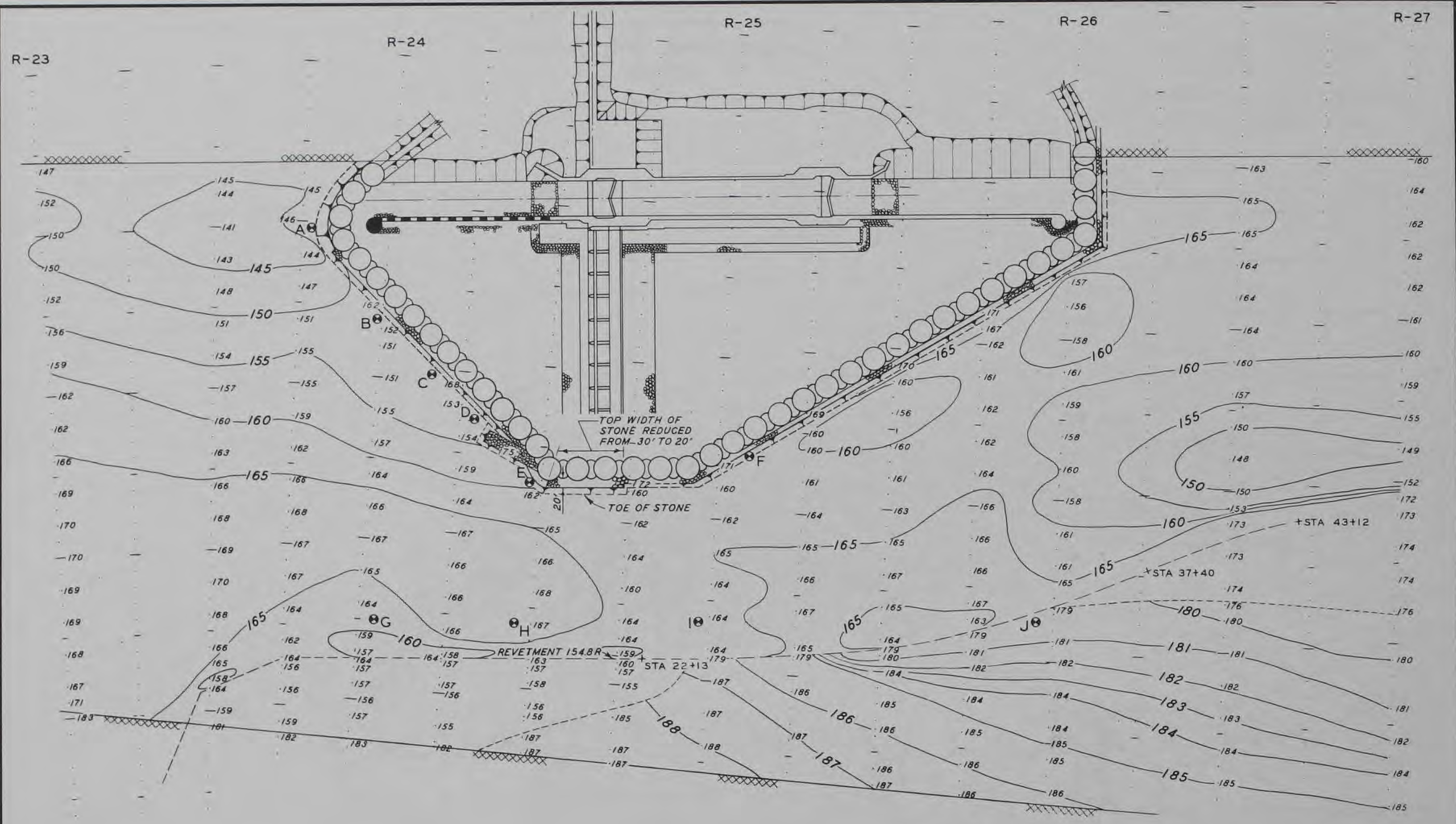
PROTOTYPE 100 0 100 200 300 400 500 FT

MODEL 1 0 1 2 3 4 FT

VELOCITIES AND CURRENT DIRECTIONS

STAGE I TEST II

DISCHARGE 300,000 CFS



SCALES

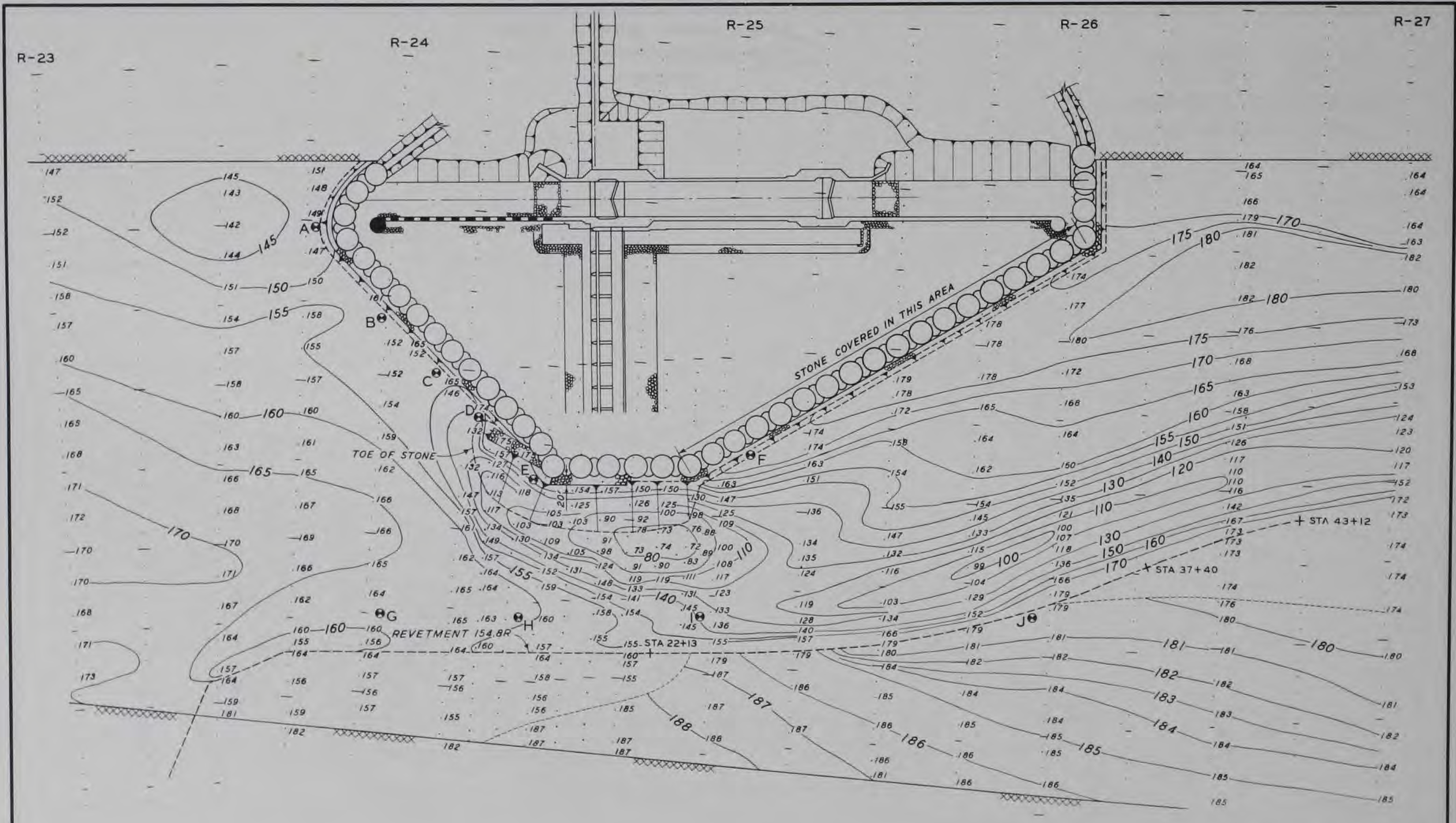
PROTOTYPE 100 0 100 200 300 400 500 FT

MODEL 1 0 1 2 3 4 FT

CHANNEL CONFIGURATION

BEGINNING OF TEST 12

DISCHARGE 300,000 CFS



CHANNEL CONFIGURATION

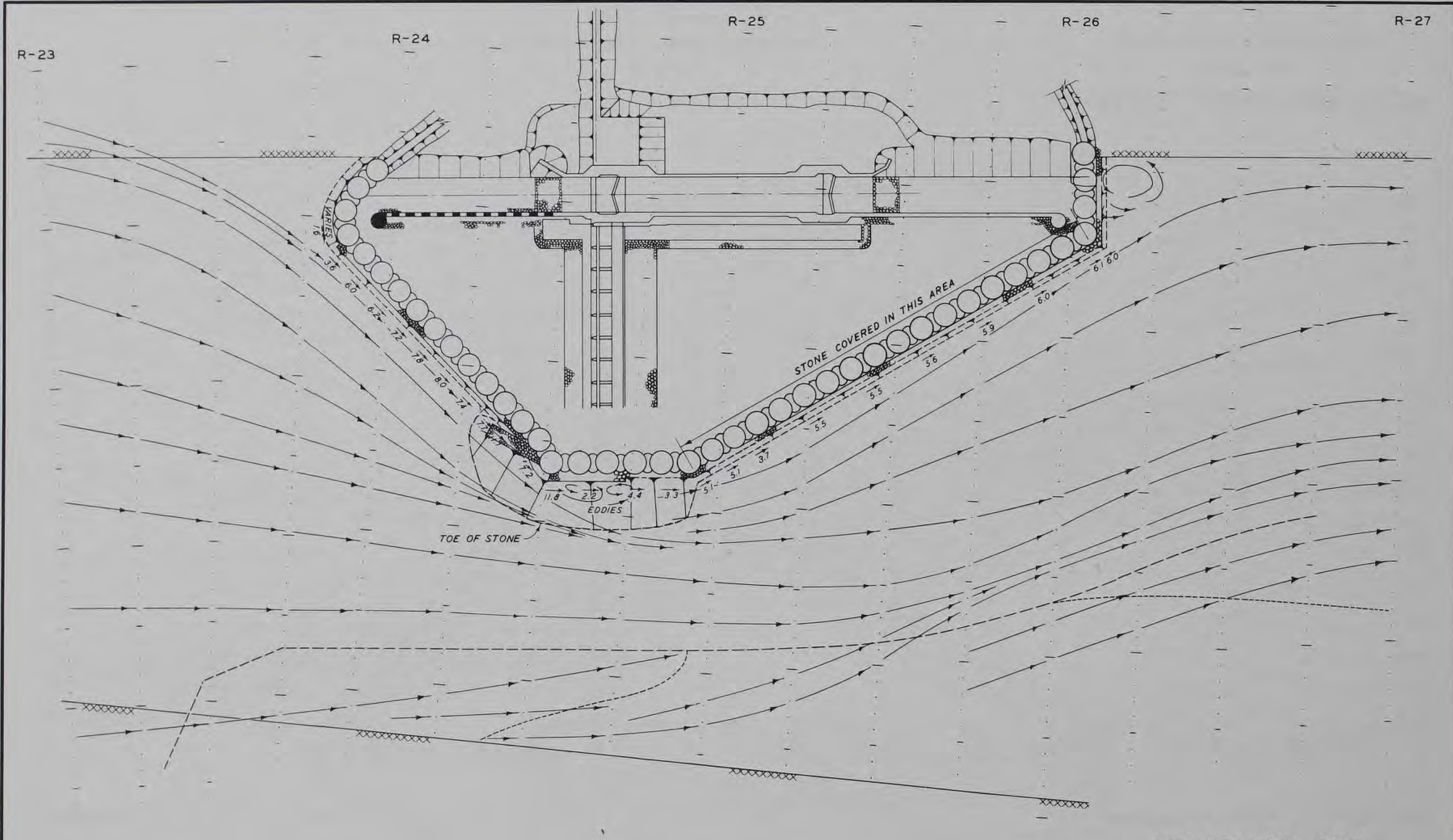
TEST 12

DISCHARGE 300,000 CFS

SCALES

PROTOTYPE 100 0 100 200 300 400 500 FT

MODEL 1 0 1 2 3 4 FT



SCALES

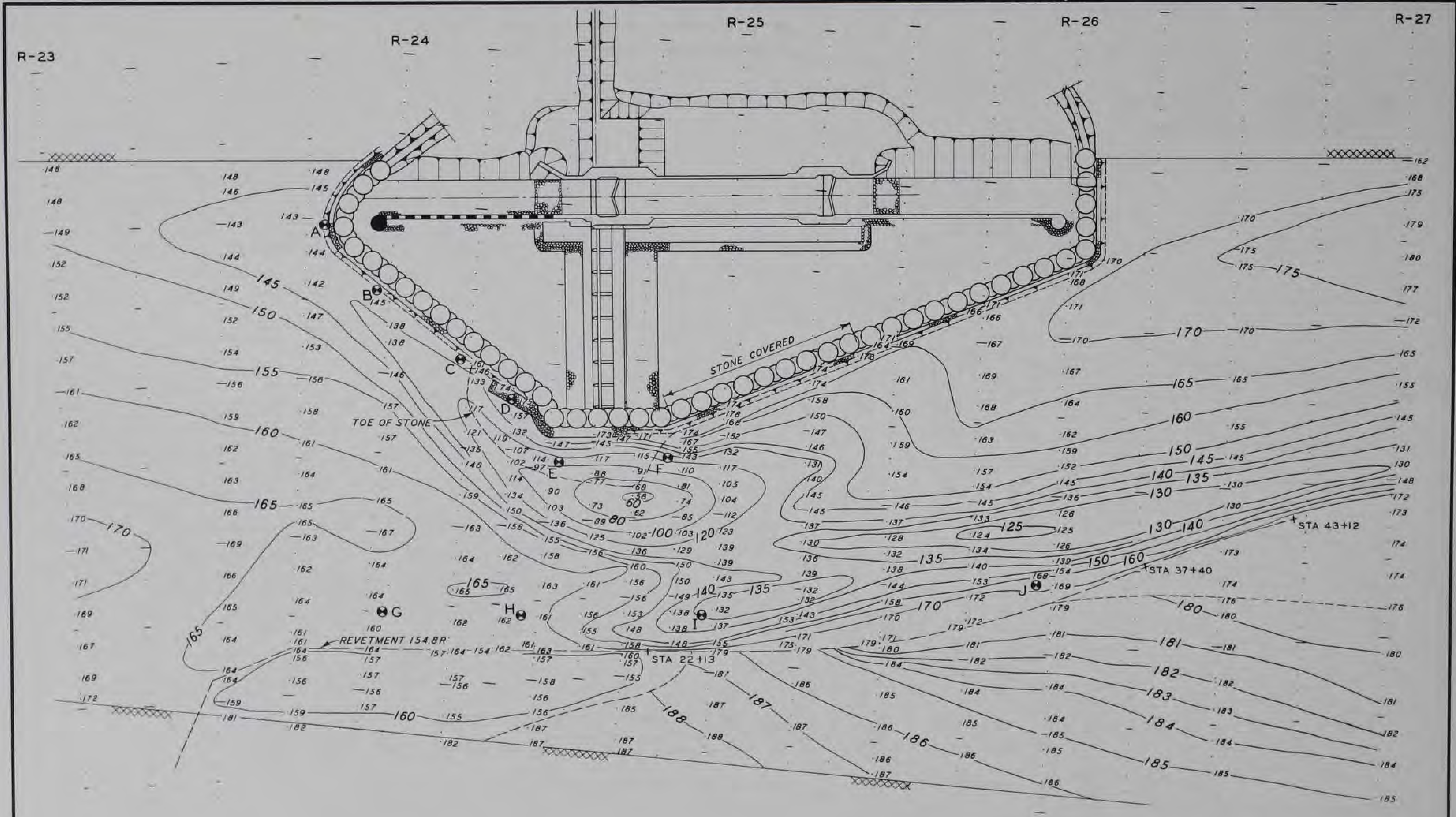
PROTOTYPE 100 0 100 200 300 400 500 FT

MODEL 1 0 1 2 3 4 FT

VELOCITIES AND
CURRENT DIRECTIONS

STAGE I TEST 12

DISCHARGE 300,000 CFS



SCALES

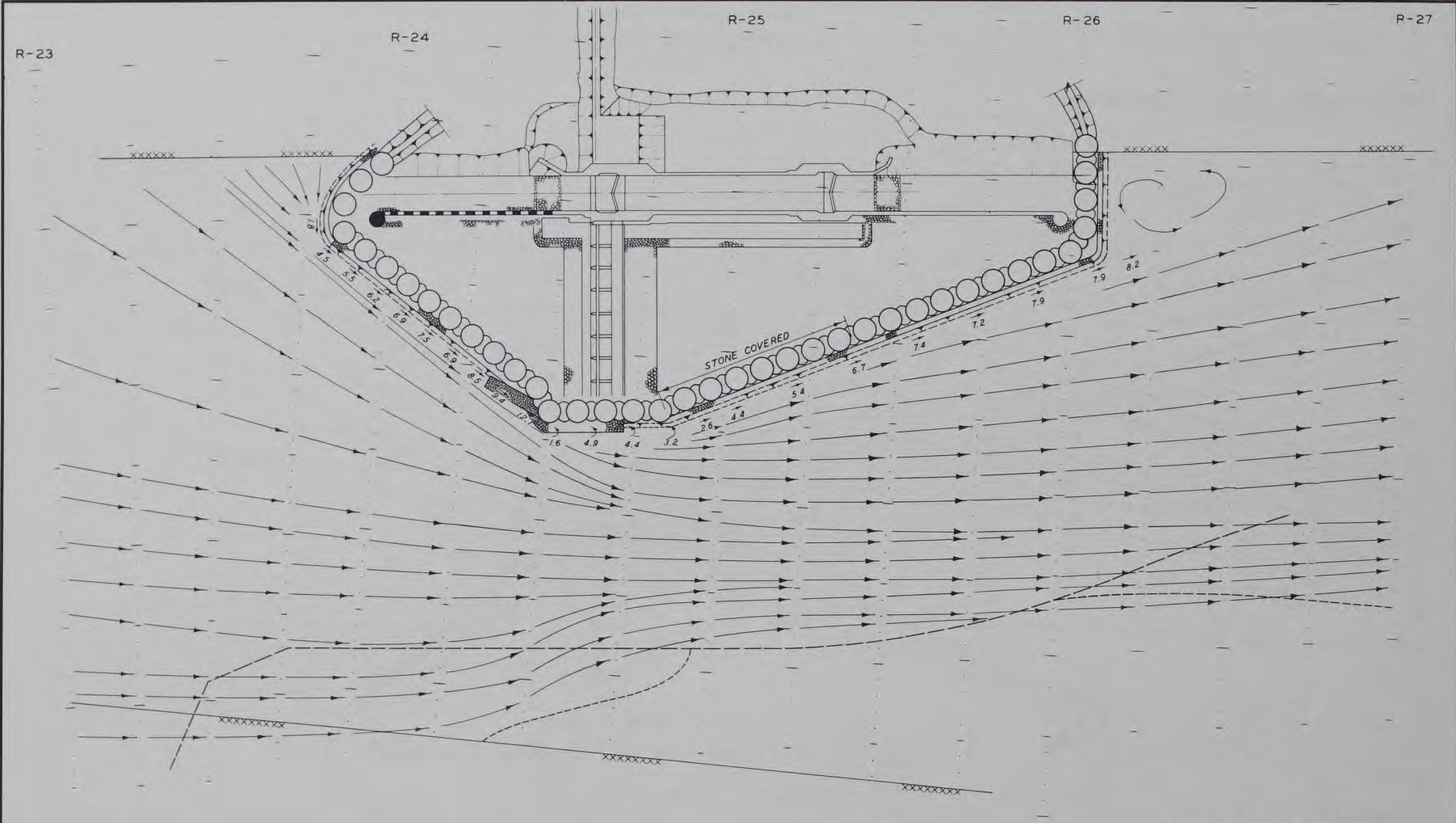
PROTOTYPE 100 0 100 200 300 400 500 FT

MODEL . 1 0 1 2 3 4 FT

CHANNEL CONFIGURATION

TEST 13

DISCHARGE 300,000 CFS



SCALES

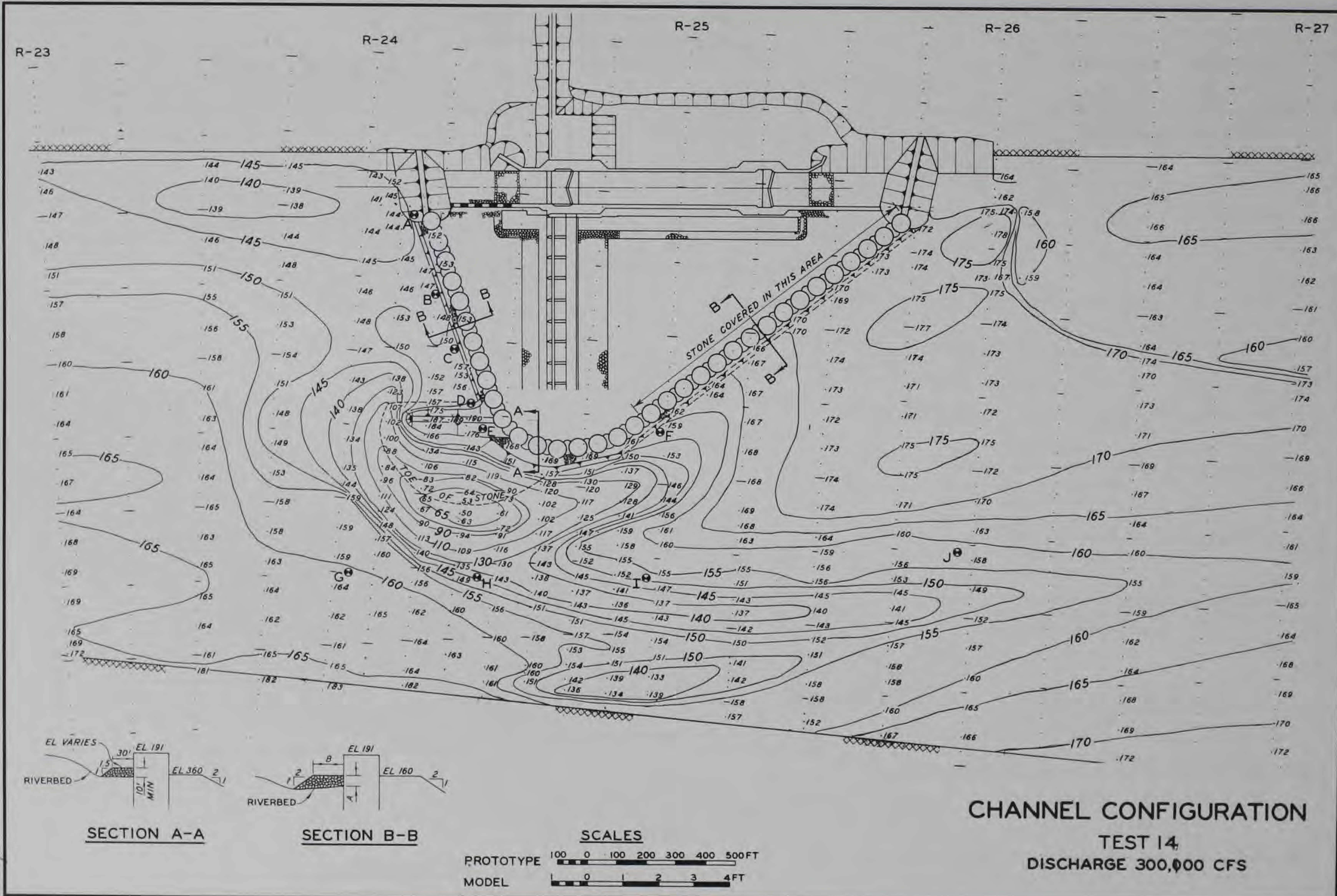
PROTOTYPE 100 0 100 200 300 400 500 FT

MODEL 1 0 1 2 3 4 FT

VELOCITIES AND CURRENT DIRECTIONS

STAGE I TEST 13

DISCHARGE 300,000 CFS

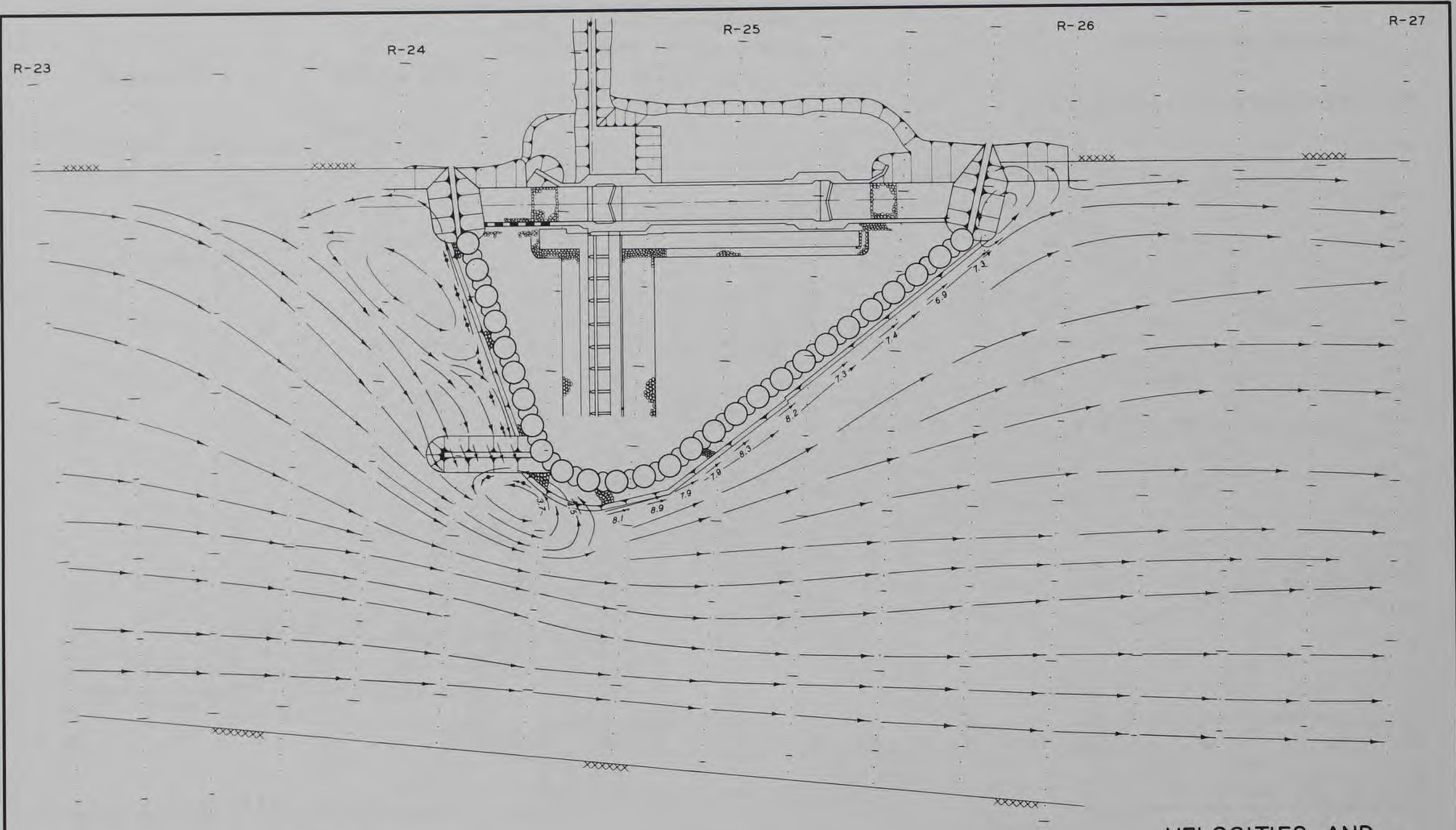


CHANNEL CONFIGURATION

TEST 14

DISCHARGE 300,000 CFS

PLATE 79



NOTE: 9.7 FPS MAXIMUM VELOCITY OBSERVED AT START OF TEST AT UPSTREAM END OF DIKE.

12.3 FPS MAXIMUM VELOCITY OBSERVED AT START OF TEST ALONG RIVER SIDE OF DIKE.

—•— VELOCITY LESS THAN 0.5 FPS

SCALES

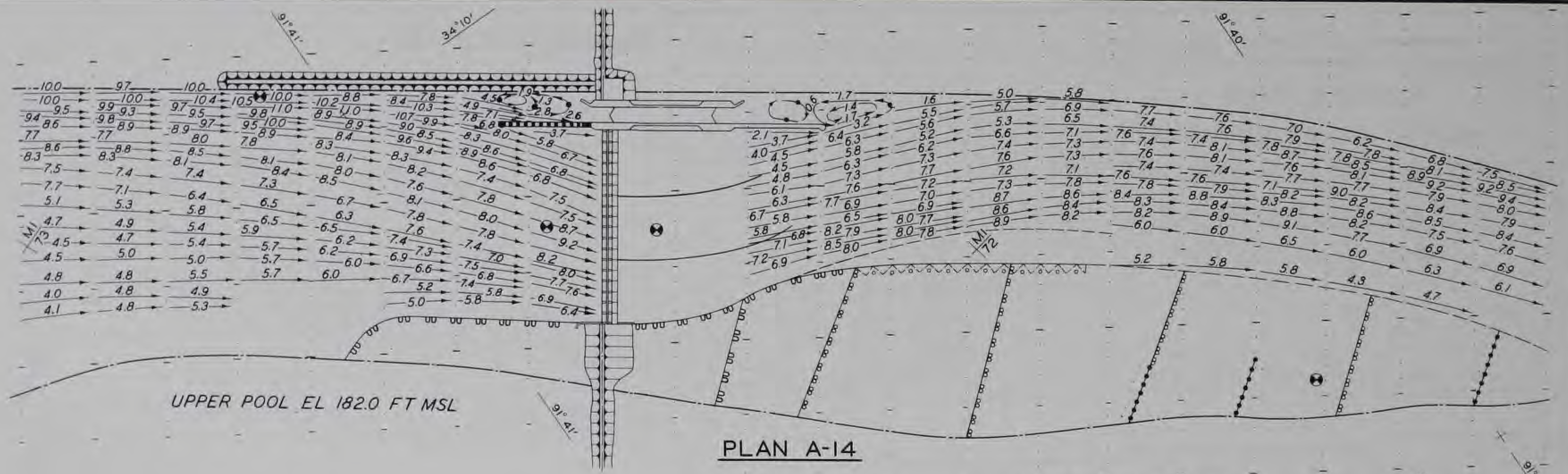
PROTOTYPE 100 0 100 200 300 400 500 FT

MODEL 1 0 1 2 3 4 FT

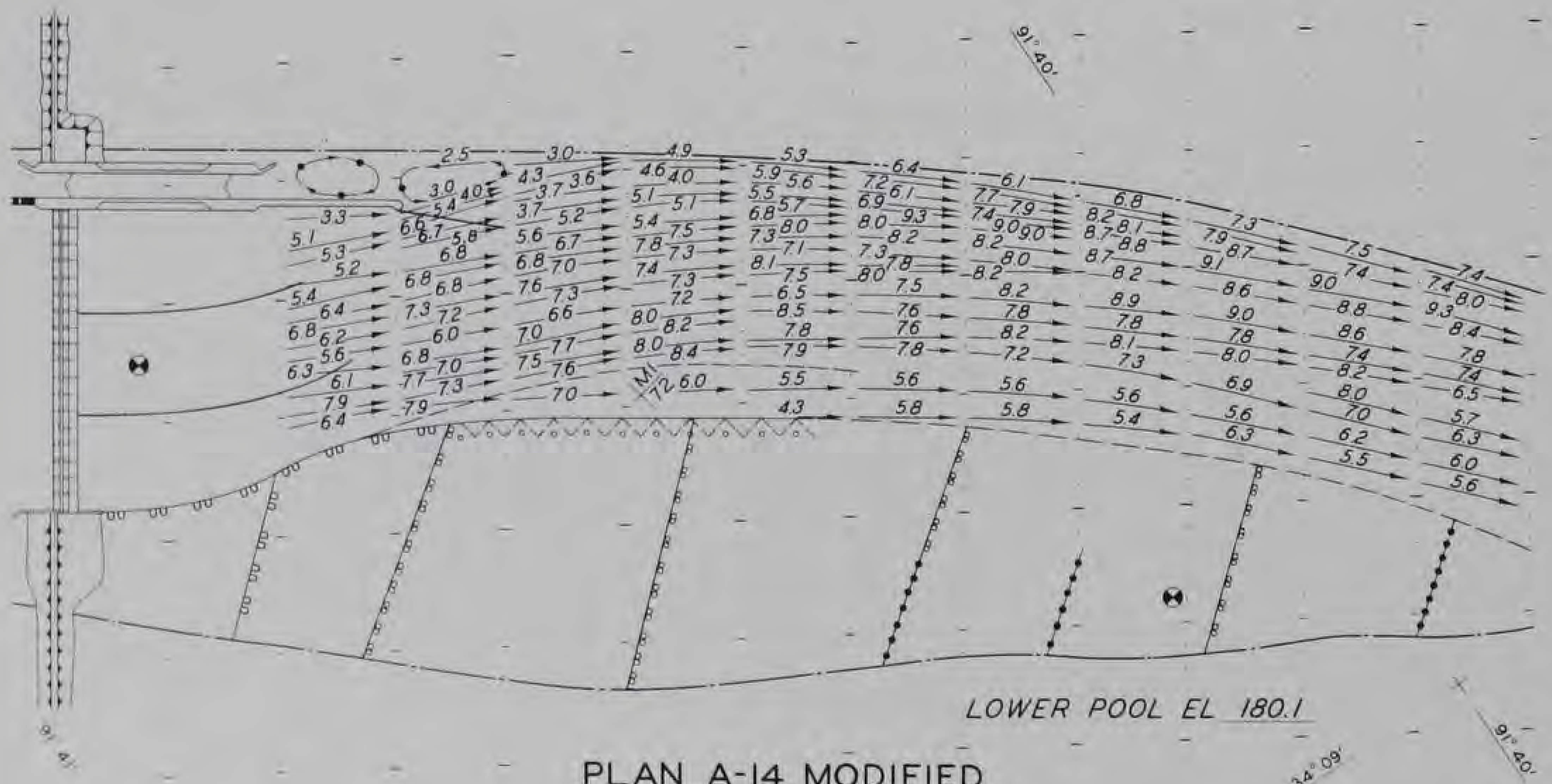
VELOCITIES AND CURRENT DIRECTIONS

STAGE I TEST 14

DISCHARGE 300,000 CFS



PLAN A-14



PLAN A-14 MODIFIED

LEGEND

- 2.5 VELOCITY IN FEET PER SECOND
- VELOCITY LESS THAN 0.5 FEET PER SECOND
- MI 72 1953 RIVER MILES
- APPROXIMATE TOP BANK
- PILE DIKE WITH STONE FILL
- PROPOSED STONE-FILL REVETMENT
- PROPOSED PILE REVETMENT
- PROPOSED PILE DIKE

NOTE: VELOCITIES AND CURRENT DIRECTIONS TAKEN WITH FLOATS SUBMERGED TO DRAFT OF LOADED BARGES (8 FT)

VELOCITIES AND CURRENT DIRECTIONS

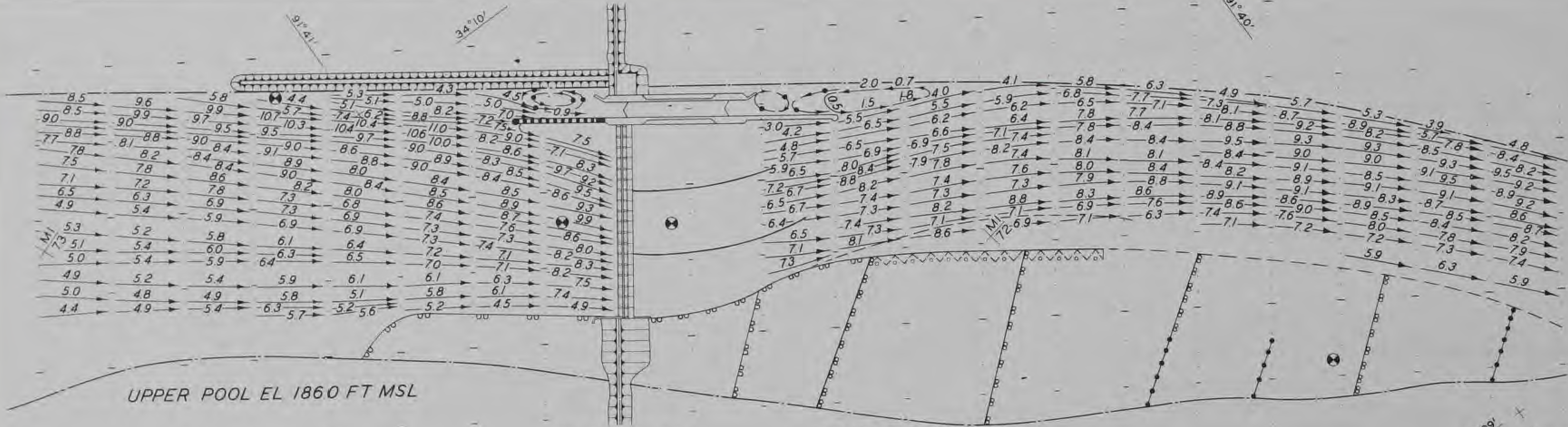
PLANS A-14 AND A-14 MODIFIED

DISCHARGE 150,000 CFS

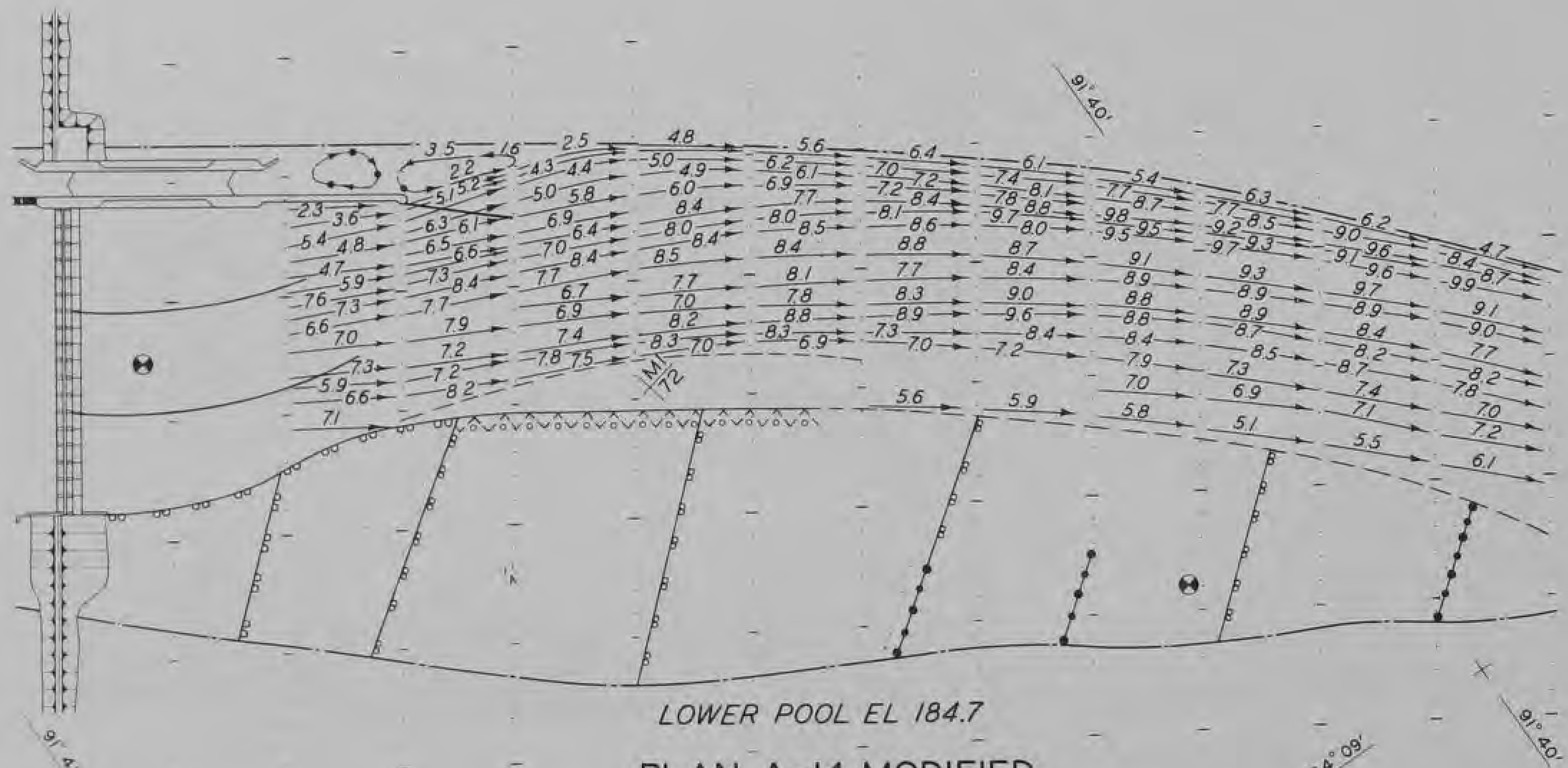
SCALES IN FEET

PROTOTYPE 500 0 500 1000 1500

MODEL 5 0 5 10



PLAN A-14



PLAN A-14 MODIFIED

LEGEND

- 2.5 — VELOCITY IN FEET PER SECOND
- VELOCITY LESS THAN 0.5 FEET PER SECOND
- MI 72 — 1953 RIVER MILES
- APPROXIMATE TOP BANK
- PILE DIKE WITH STONE FILL
- PROPOSED STONE-FILL REVETMENT
- PROPOSED PILE REVETMENT
- PROPOSED PILE DIKE

NOTE: VELOCITIES AND CURRENT DIRECTIONS TAKEN WITH FLOATS SUBMERGED TO DRAFT OF LOADED BARGES (8 FT)

VELOCITIES AND CURRENT DIRECTIONS

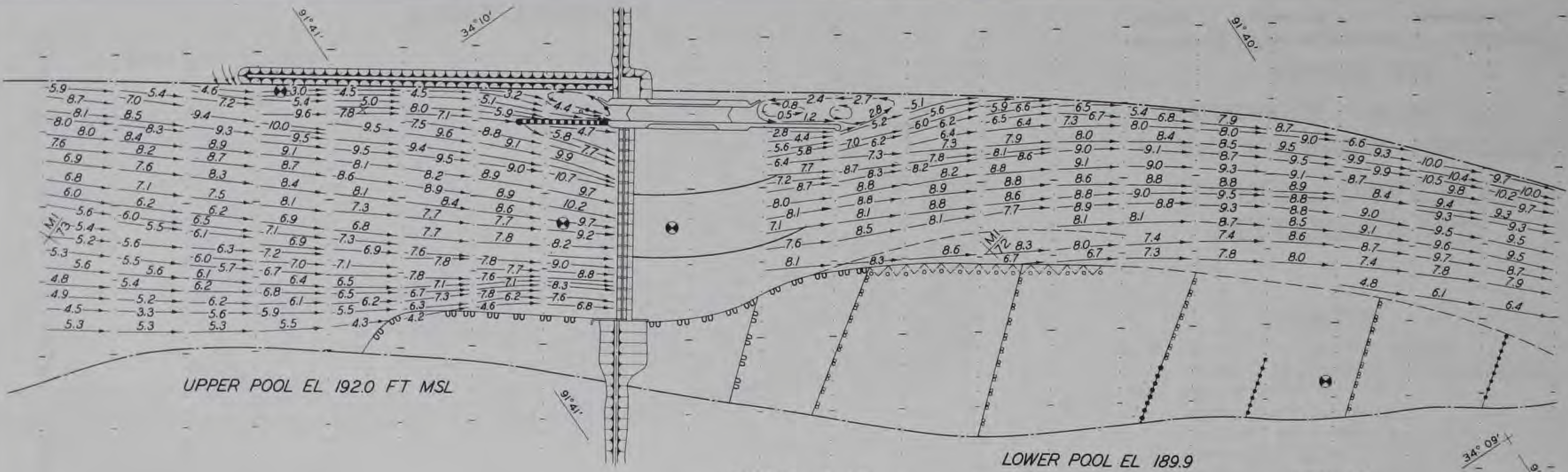
PLANS A-14 AND A-14 MODIFIED

DISCHARGE 230,000 CFS

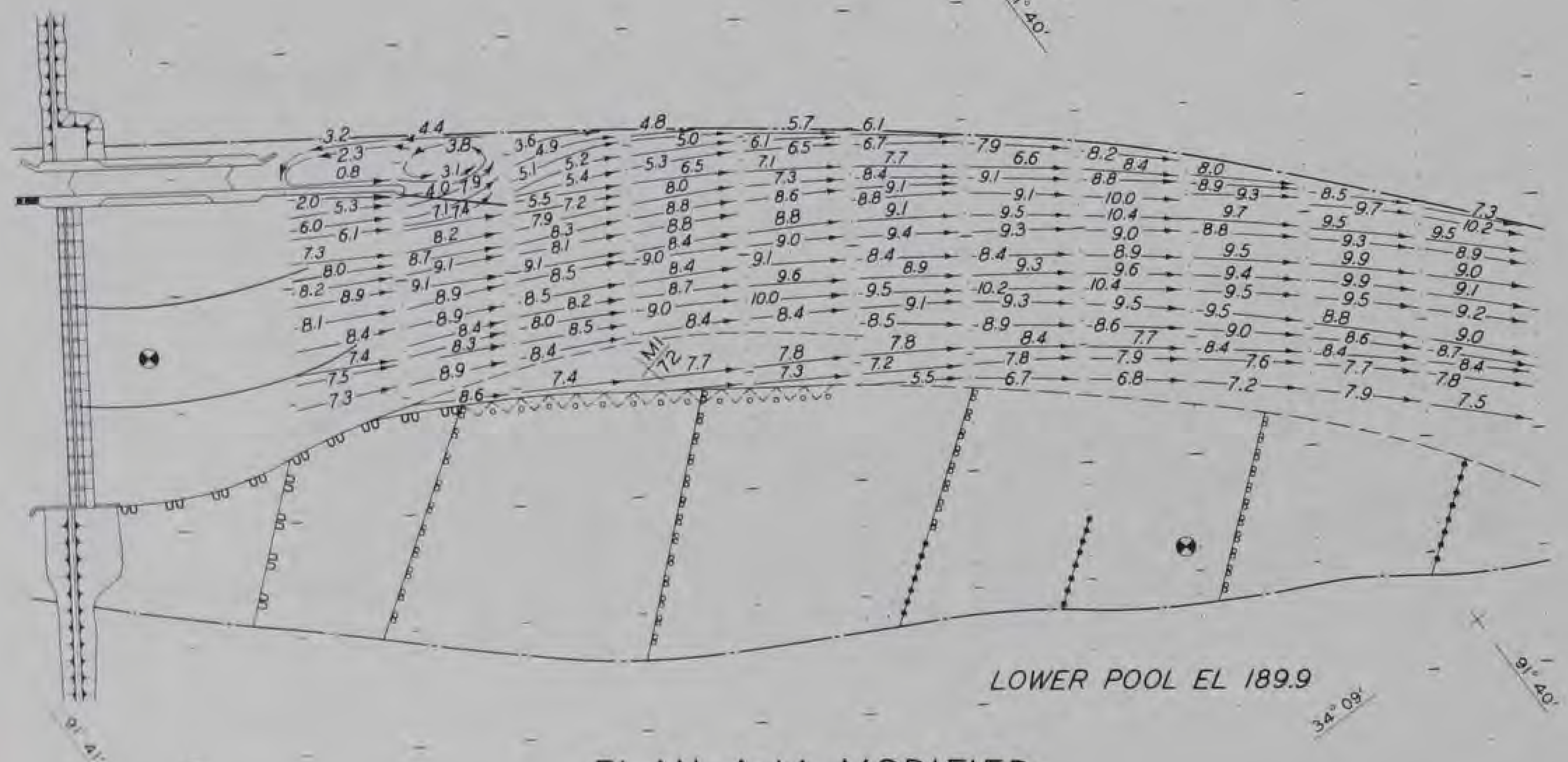
SCALES IN FEET

PROTOTYPE 500 0 500 1000 1500

MODEL 5 0 5 10



PLAN A-14



PLAN A-14 MODIFIED

LEGEND

2.5 — VELOCITY IN FEET PER SECOND
VELOCITY LESS THAN 0.5 FEET PER SECOND
MI 72 — 1953 RIVER MILES
APPROXIMATE TOP BANK
PILE DIKE WITH STONE FILL
PROPOSED STONE-FILL REVETMENT
PROPOSED PILE REVETMENT
PROPOSED PILE DIKE

NOTE: VELOCITIES AND CURRENT DIRECTIONS TAKEN WITH FLOATS SUBMERGED TO DRAFT OF LOADED BARGES (8 FT)

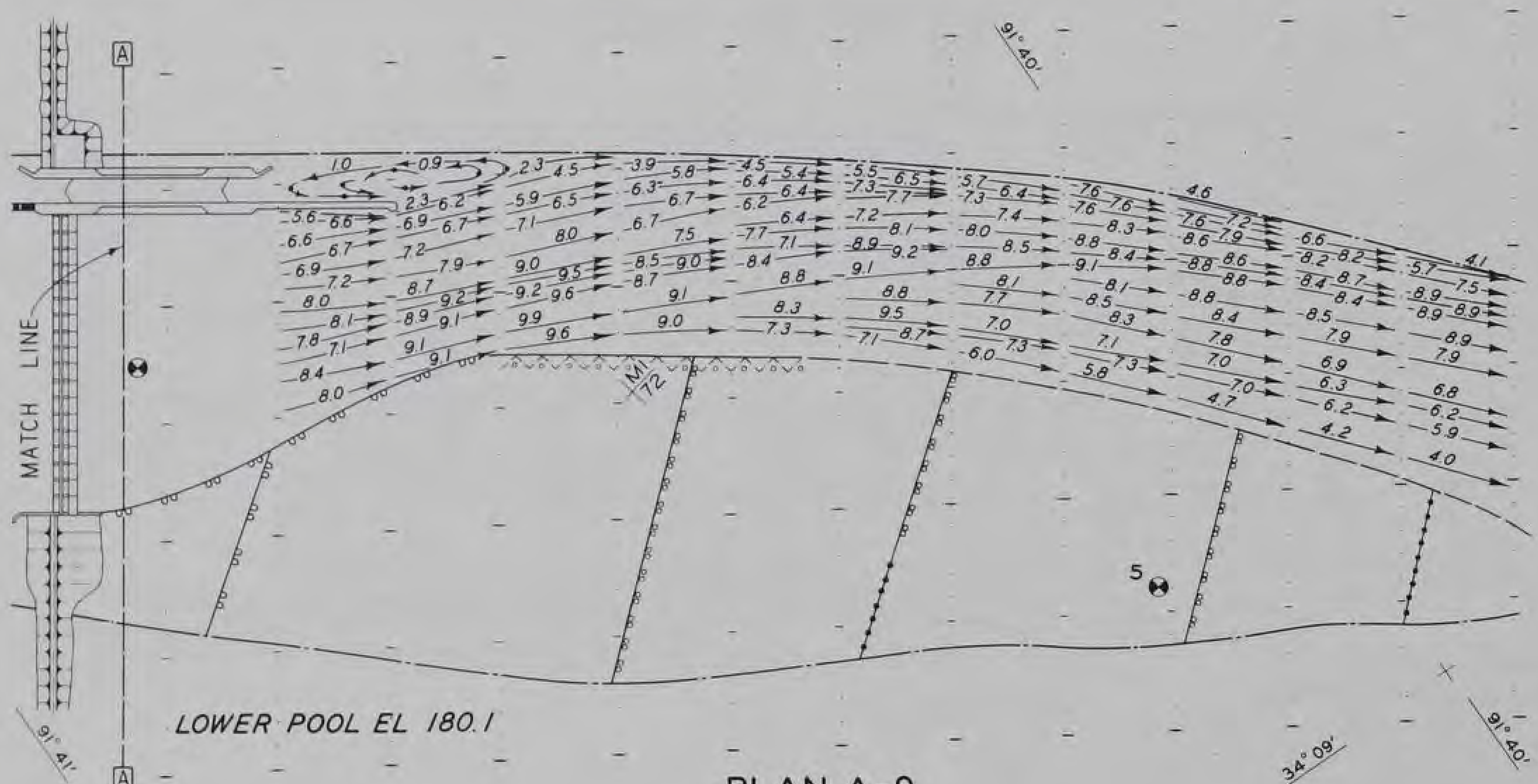
VELOCITIES AND CURRENT DIRECTIONS

PLANS A-14 AND A-14 MODIFIED

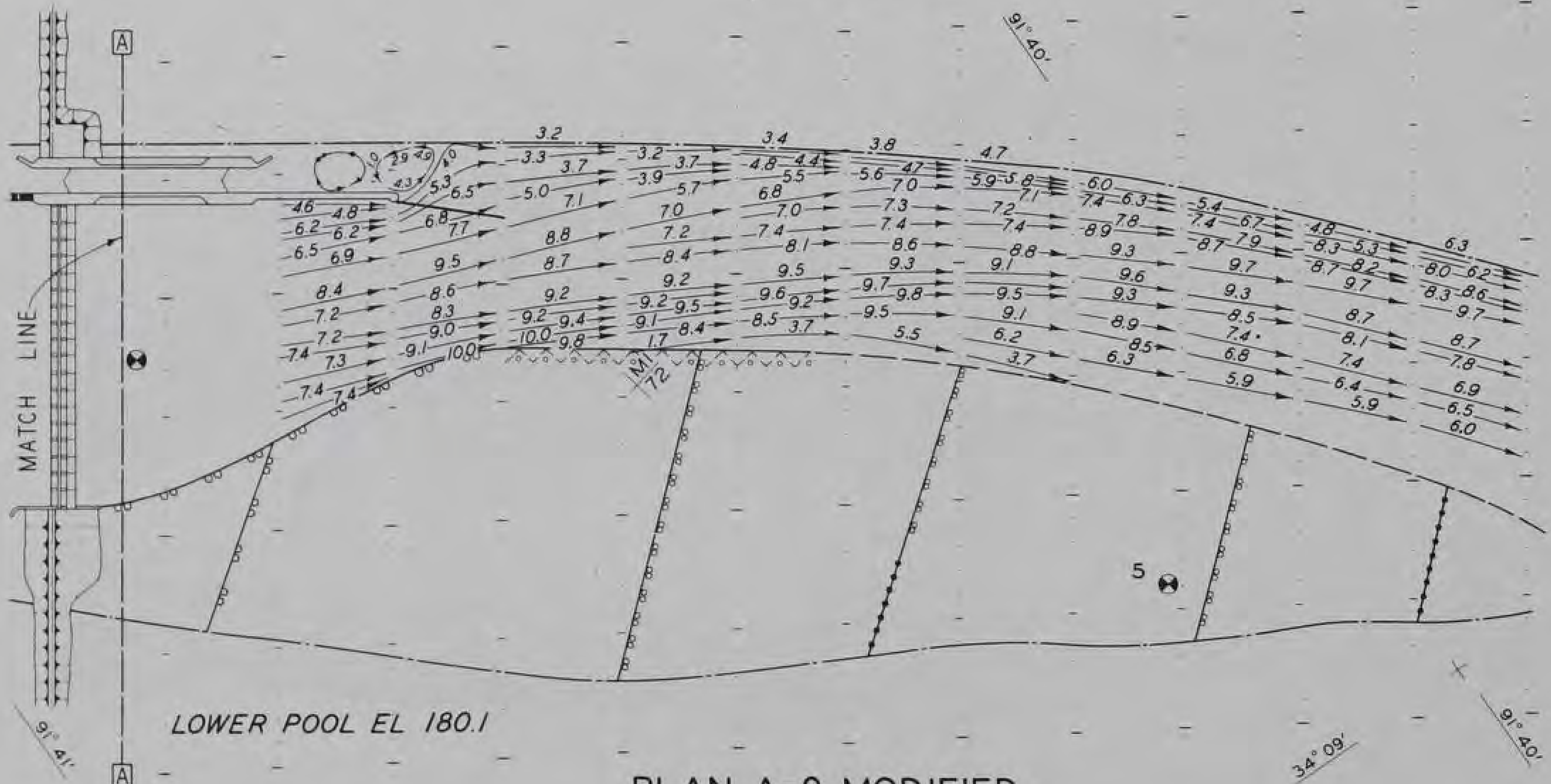
DISCHARGE 350,000 CFS

SCALES IN FEET

PROTOTYPE 500 0 500 1000 1500
MODEL 5 0 5 10



PLAN A-9



PLAN A-9 MODIFIED

LEGEND

- 2.5 VELOCITY IN FEET PER SECOND
- VELOCITY LESS THAN 0.5 FEET PER SECOND
- MI 72 1953 RIVER MILES
- APPROXIMATE TOP BANK
- PILE DIKE WITH STONE FILL
- PROPOSED STONE-FILL REVETMENT
- PROPOSED PILE REVETMENT
- PROPOSED PILE DIKE

NOTE: VELOCITIES AND CURRENT DIRECTIONS TAKEN WITH FLOATS SUBMERGED TO DRAFT OF LOADED BARGES (8 FT)

VELOCITIES AND CURRENT DIRECTIONS

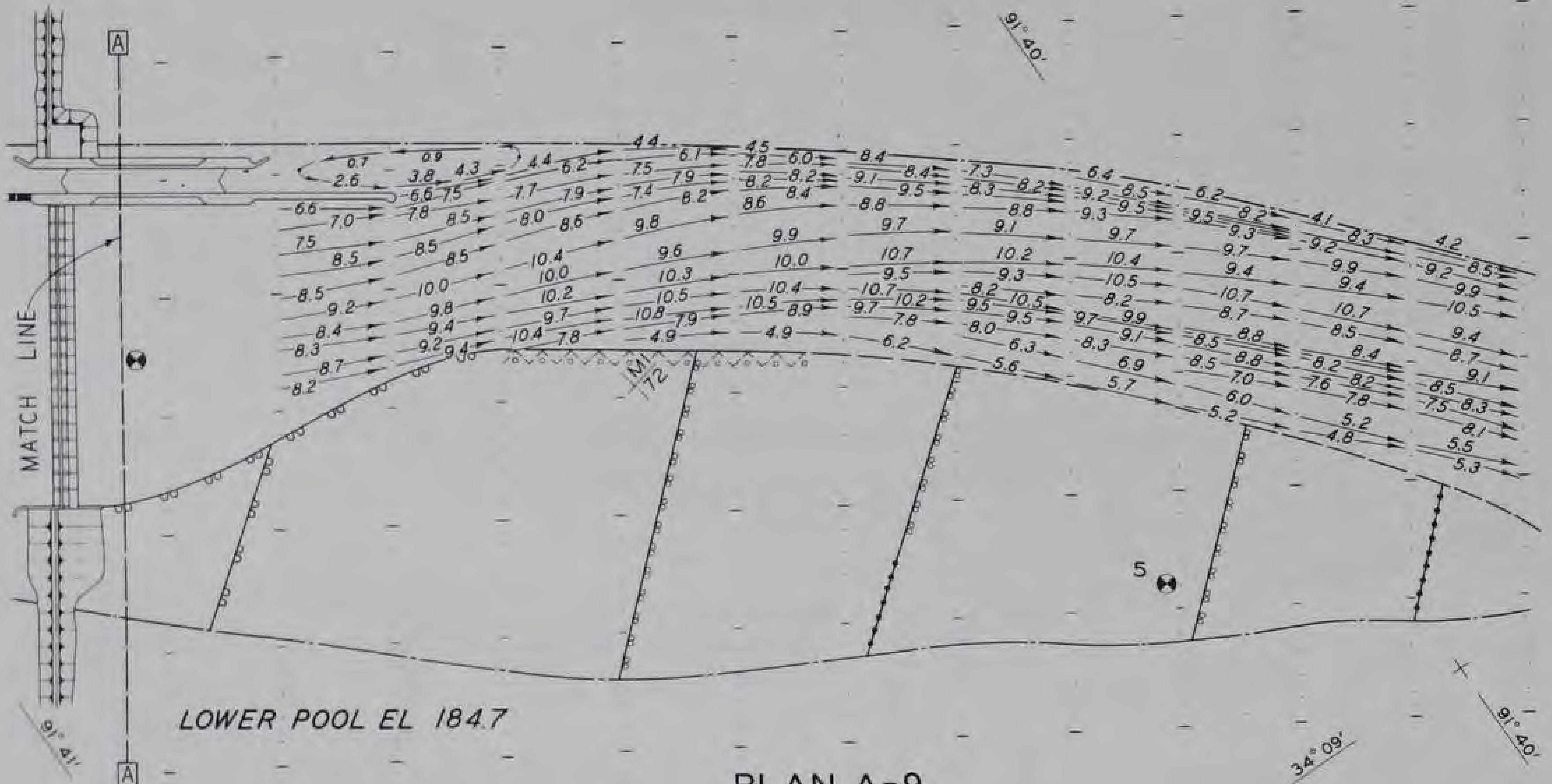
PLANS A-9 AND A-9 MODIFIED

DISCHARGE 150,000 CFS

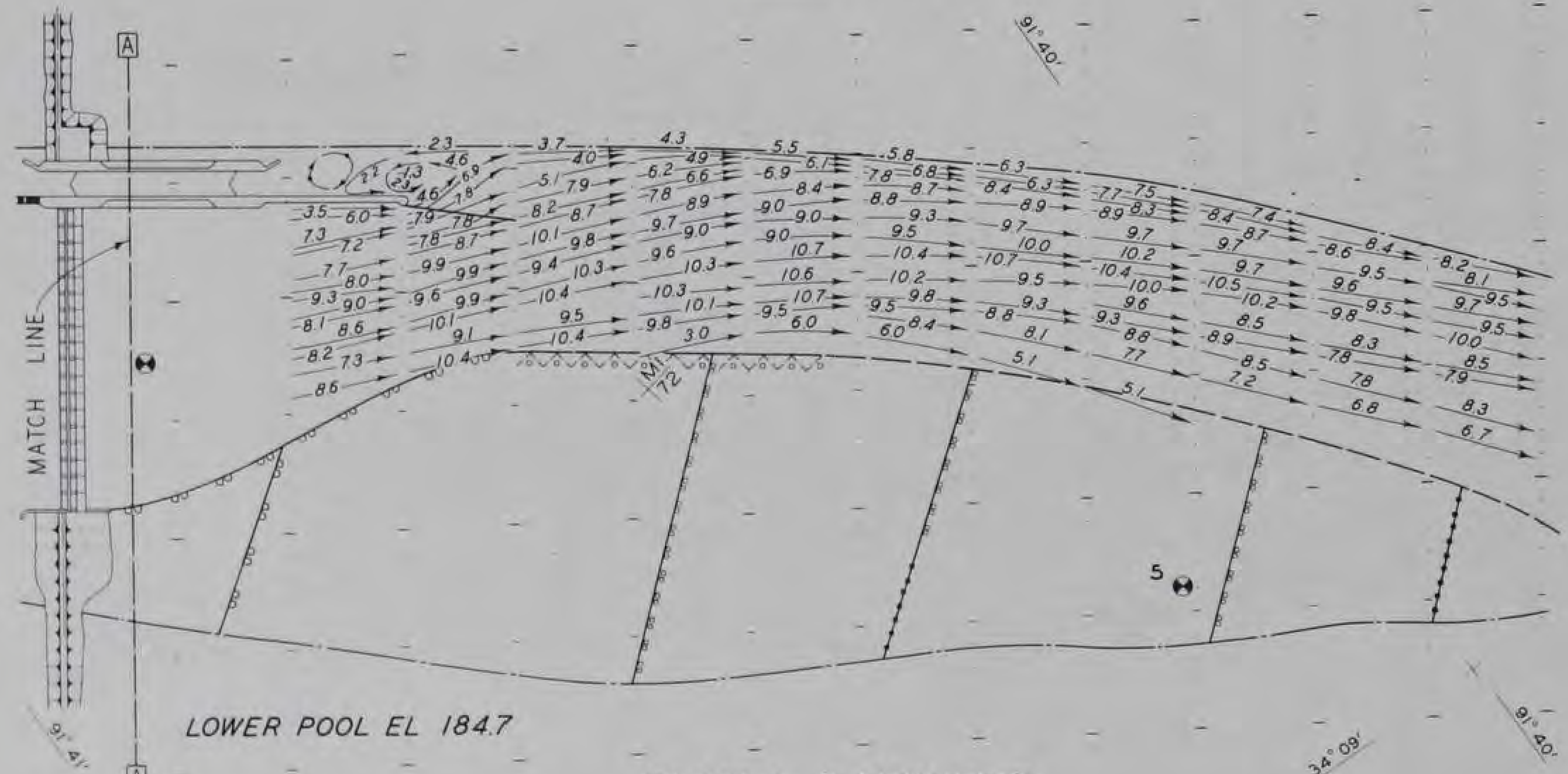
SCALES IN FEET

PROTOTYPE 500 0 500 1000 1500

MODEL 5 0 5 10



PLAN A-9



PLAN A-9 MODIFIED

LEGEND

- 2.5 VELOCITY IN FEET PER SECOND
- VELOCITY LESS THAN 0.5 FEET PER SECOND
- MI 72 1953 RIVER MILES
- APPROXIMATE TOP BANK
- PILE DIKE WITH STONE FILL
- PROPOSED STONE-FILL REVETMENT
- PROPOSED PILE REVETMENT
- PROPOSED PILE DIKE

NOTE: VELOCITIES AND CURRENT DIRECTIONS TAKEN WITH FLOATS SUBMERGED TO DRAFT OF LOADED BARGES (8 FT)

VELOCITIES AND CURRENT DIRECTIONS

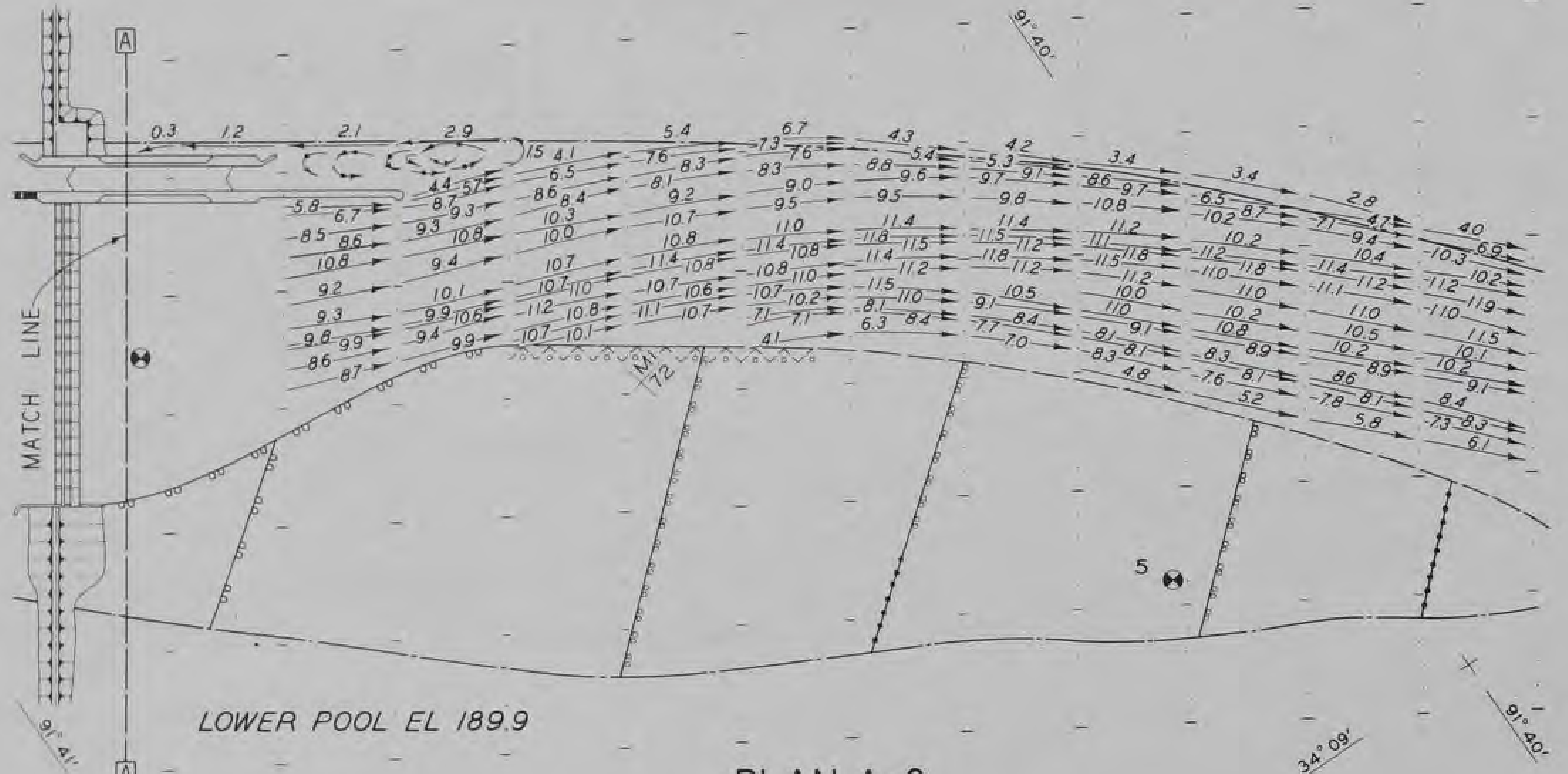
PLANS A-9 AND A-9 MODIFIED

DISCHARGE 230,000 CFS

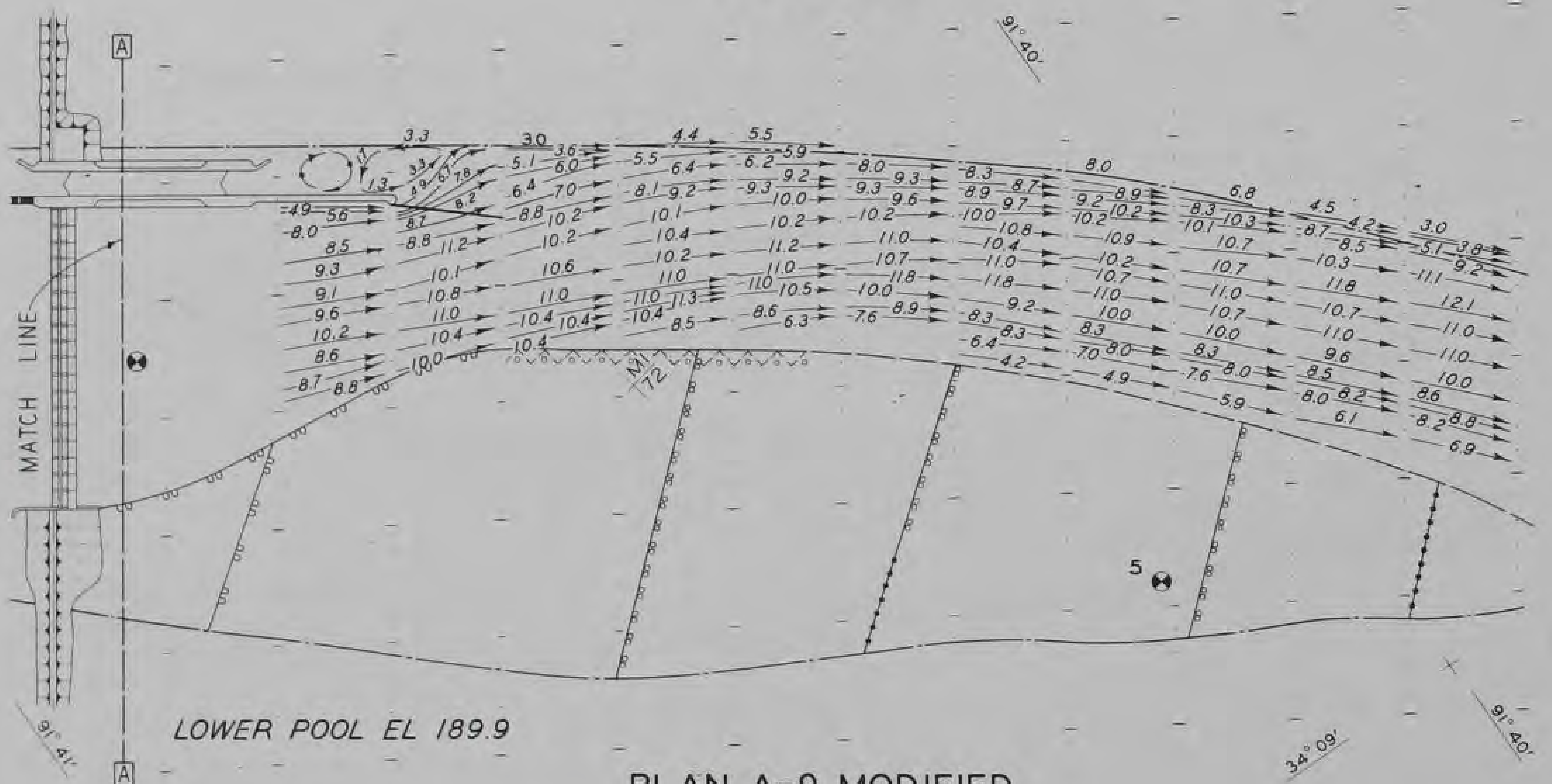
SCALES IN FEET

PROTOTYPE 500 0 500 1000 1500

MODEL 5 0 5 10



PLAN A-9



PLAN A-9 MODIFIED

LEGEND

- 2.5 VELOCITY IN FEET PER SECOND
- VELOCITY LESS THAN 0.5 FEET PER SECOND
- MI 72 1953 RIVER MILES
- APPROXIMATE TOP BANK
- PILE DIKE WITH STONE FILL
- PROPOSED STONE-FILL REVETMENT
- PROPOSED PILE REVETMENT
- PROPOSED PILE DIKE

NOTE: VELOCITIES AND CURRENT DIRECTIONS TAKEN WITH FLOATS SUBMERGED TO DRAFT OF LOADED BARGES (8 FT)

VELOCITIES AND CURRENT DIRECTIONS

PLANS A-9 AND A-9 MODIFIED

DISCHARGE 350,000 CFS

SCALES IN FEET

PROTOTYPE 500 0 500 1000 1500

MODEL 5 0 5 10

Unclassified

Security Classification

DOCUMENT CONTROL DATA - R & D

(Security classification of title, body of abstract and indexing annotation must be entered when the overall report is classified)

1. ORIGINATING ACTIVITY (Corporate author)	2a. REPORT SECURITY CLASSIFICATION
U. S. Army Engineer Waterways Experiment Station Vicksburg, Mississippi	Unclassified
	2b. GROUP

3. REPORT TITLE

NAVIGATION CONDITIONS AT LOCK AND DAM NO. 3, ARKANSAS RIVER, ARKANSAS AND OKLAHOMA; Hydraulic Model Investigation

4. DESCRIPTIVE NOTES (Type of report and inclusive dates)

Final report

5. AUTHOR(S) (First name, middle initial, last name)

John J. Franco
Cody D. McKellar

6. REPORT DATE	7a. TOTAL NO. OF PAGES	7b. NO. OF REFS
September 1968	177	0

8a. CONTRACT OR GRANT NO.	9a. ORIGINATOR'S REPORT NUMBER(S)
b. PROJECT NO.	Technical Report H-68-8
c.	9b. OTHER REPORT NO(S) (Any other numbers that may be assigned this report)
d.	

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11. SUPPLEMENTARY NOTES	12. SPONSORING MILITARY ACTIVITY
	U. S. Army Engineer District Little Rock, Arkansas

13. ABSTRACT

Lock and Dam No. 3 will consist of a 110- by 600-ft lock and a 1260-ft-long, gated, nonnavigable dam. A movable-bed model, reproducing 12.8 miles of the Arkansas River to a horizontal scale of 1:120 and vertical scale of 1:80, was used to determine: suitability of the proposed site for the structures; adequacy of proposed regulating works upstream and downstream including a proposed cutoff; modifications required to provide adequate channel dimensions and safe navigation conditions with minimum maintenance; and effects of various cofferdam and diversion plans. Special studies were also conducted with the model converted to a 1:120 scale (undistorted). Test results indicated that satisfactory navigation conditions can be developed with the lock and dam at the proposed site. Development of channel dimensions was affected considerably by the tendency of the channel to meander within the long, straight reach at the site and the effects of dam gate operation on sediment movement. Regulating structures that provided adequate channel dimensions under typical flow conditions were developed, except in the lower lock approach channel. A satisfactory plan for elimination of shoaling in the lower lock approach was not developed. Increasing the capacity of the ports in the upper guard wall would eliminate hazardous crosscurrents in the upper lock approach and reduce scour near the end of the upper guard wall. A fill along the left overbank would prevent overbank flow moving toward the spillway from seriously affecting downbound tows approaching the lock. In cofferdam tests, water-surface elevations along the upstream side, location of maximum scour, and scour pattern were determined together with the factors affecting these conditions.

DD FORM 1 NOV 65 1473 REPLACES DD FORM 1473, 1 JAN 64, WHICH IS OBSOLETE FOR ARMY USE.

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Security Classification

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14. KEY WORDS	LINK A		LINK B		LINK C	
	ROLE	WT	ROLE	WT	ROLE	WT
Arkansas River						
Dams, Navigation						
Hydraulic models						
Lock and Dam No. 3, Arkansas River						
Locks, Navigation						
Navigation						

Unclassified

Security Classification