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CONTROL OF ALLUVIAL RIVERS BY STEEL JETTIES

by

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SYNOPSIS

Both field and laboratory studies were conducted to refine the methods used in the design of steel jetty fields for river alinement. A set of dimensionless friction head-loss curves, verified by model studies are developed and described. Using the developed curves and reconnaissance field data, a method is given for predicting the changes in a riverbed after the designed jetty field is installed.

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INTRODUCTION

Steel jacks and jetties have been used successfully by the Corps of Engineers, highway departments, railway companies, and others to prevent damage to riverbanks, levees, bridge abutments, and other structures. The Bureau of Reclamation and the Corps of Engineers are using them to stabilize the channel of the Rio Grande within the floodway in the Middle Rio Grande Valley. (1)1/

The individual jack unit consists of three angle irons, 12 or 16 feet in length placed at 90° angles in three planes and joined at their centers, Figure 1. Wire is laced through the angle irons in a standard pattern to tie them together. The jacks are placed in rows along the proposed riverbank line and in tieback lines extending to the old riverbank. The jacks in each row are then fastened together on a common cable. The entire assembly is called a jetty field. Figure 2 shows a plan and cross section of a jetty field installation.

Jetty fields incorporate some of the good features of walls and groins and are also permeable, reducing the possibilities of overconfining the river and causing scour such as occurs at the ends of solid groins. Lines of jacks have the added desirable quality of being flexible and will settle as scour occurs conforming to the bed where they are most effective.

The ideal operation of a jetty field may be described as follows: Lines of jacks in the flow area provide additional resistance to the water passing through the field, which in turn reduces the flow velocity. This reduces the sediment carrying capacity of the water, and sediment is deposited in the field. Vegetal growth in the deposited sediment provides additional flow resistance. Sufficient sediment is accumulated to form a new riverbank and induce the river to flow in the designed channel. Channelization causes the riverbed to scour and this results in a lower water surface. This discourages water-loving plants growing along the floodway and banks, and transpiration losses are reduced.

The individual jack unit costs approximately $35 to $55 installed. To achieve the best economy, it was desirable to determine the hydraulic losses produced by a jack and jetty field, and to develop improved design methods.

1/Numerals in parentheses--thus (1)--refer to corresponding items in the Bibliography--See Appendix I.

PREVIOUS STUDIES

A report by H. A. Einstein, "Report on the Investigation of the Fundamentals of the Action of River Training Structures," (2) described a drag force study conducted with jacks and a movable bed model study of different jetty field installations. The study gives values of the coefficient of drag in dimensionless form for "loaded" and "unloaded" jacks--loaded meaning jacks that are entangled with river debris. The report states that jetty fields are only practical for use in straight reaches and that curves with radii greater than 14 channel widths act like straight reaches.

Another publication of interest is, "Use of Kellner Jetties on Alluvial Streams," (3) Corps of Engineers, June 1953. This paper is a compilation of experiences of the Albuquerque District Office with jetty field installations. The fabrication of steel jetties is described. The report gives a typical specification and compares costs of different installations. It states that 1 to 3 feet of deposition in a jetty field can be expected annually. The 1 foot of deposition is associated with average flow conditions, and the 3 feet is associated with an unusually heavy flow. The report states that a steel jetty field has a life expectancy of 50 years or longer.

Earlier USBR studies were described in the paper, "Use of Steel Jetties for Bank Protection and Channelization in Rivers," (4) by E. J. Carlson and P. F. Enger, presented at the Hydraulics Division Meeting, ASCE, Madison, Wisconsin, August 1956. In this paper the velocity change in the jetty field is expressed in terms of unit discharge. The number of tieback lines were varied from one to seven, the velocity of approach was equal to or less than 4.16 feet per second, the Froude number ranged from 0.066 to 0.30, and the model was always operated at depths greater than critical. Tieback jacks represented in the model study were made with 12-foot by 3- by 3- by 1/4-inch angle irons laced with No. 6 galvanized wire and the jetty field width equaled the channel width.

Further USBR work was described in a discussion prepared by E. J. Carlson for the Seminar, "Transportation of Material in Water," at the Eighth Congress of the International Association for Hydraulic Research, August 1959, in Montreal, Canada. (5) The discussion was based on the velocity recovery concept for jetty fields which can be described as follows: Consider a simple jetty field consisting of one tieback line and one continuous frontline. As flow passes through the tieback line, a velocity reduction occurs and some of the flow moves out into the channel due to the damming effect of the tieback. Downstream from the tieback, flow passes back into the jetty field from the channel. The velocity in the field continually increases in proportion to the distance downstream from the tieback line until it attains normal velocity for the slope and roughness of the river section. To maintain a velocity in the jetty field lower than the normal velocity for the river section, the tiebacks must be spaced so that complete velocity recovery does not occur between them.

Hydraulic model studies were conducted to relate tieback spacing with velocity recovery. A fixed bed model was arranged in an open channel flume 13 feet wide with a continuous frontline of 1:16 scale jacks dividing the channel along the centerline. One tieback line of model jacks was placed at an angle of 67.5° with the frontline at its upstream end. Flows of 8.33-, 16.67-, and 25.0-cubic-feet-per-second per foot of width, representing total discharges of 5,000, 10,000 and 15,000 cubic feet per second for the Middle Rio Grande in the Casa Colorada area, were used in the study. Prototype depths represented, ranged from 4 to 8 feet.

From dimensional analysis, the following relationship was adopted:

$$\frac{V_R}{V_o} = f\left(N_F, \frac{X}{Y}, \sin \phi\right)$$

where

- V_o = Velocity in the normal river channel upstream from a jetty field
- V_R = Velocity reduction in the jetty field. $V_R = V_o$ minus the velocity in the jetty field
- X = Tieback spacing or distance downstream from a tieback
- Y = Depth in normal river channel upstream from a jetty field installation
- $N_F = V_o / \sqrt{gY}$ = Froude number of normal flow in the river channel upstream

where g is the acceleration due to gravity

- ϕ = angle between a tieback line and a frontline

The basic equation for velocity reduction was determined from the model data to be:

$$\frac{V_R}{V_o} = (0.108 \ln N_F + 0.308)\left(1 - \frac{1}{160} \cdot \frac{X}{Y}\right)\left(\frac{\sin 67.5°}{\sin \phi}\right)$$

Details of the development of design curves and nomographs based on this equation appear in USBR Hydraulic Laboratory Report No. Hyd 477. (6) The equation and the curves give conservative values of initial velocity reduction to be expected in a jetty field.

PILOT STUDY REACH

In preparation for a general channelization program for the Middle Rio Grande Project, a 2-mile river reach near Belen, New Mexico, Figure 3, was selected for a pilot study. Jetty fields were designed for the prototype pilot reach and were installed both in the field and in a hydraulic laboratory model.

In designing the jetty fields for the Casa Colorada pilot reach, a study was made by USBR hydrologists to determine a design discharge and design channel width.

Difficulties were encountered due to the peculiar characteristics of the Rio Grande. These included days of zero discharge, large fluctuations in discharge during a particular runoff, and two distinct runoff seasons, each with its own characteristic sediment load. Using several methods of determining dominant or design discharge resulted in values ranging from 900 to 13,000 cubic feet per second. A value of 5,000 cubic feet per second was selected for design. This is the lowest discharge which is difficult to handle. Larger discharges would probably occur, but these would not present as great a problem.

Several methods of computing the design channel width were used, resulting in values ranging from 330 to 770 feet. A value of 600 feet was selected for the design channel width.

Movable Bed Model of Pilot Reach

The pilot river model was constructed with a horizontal scale of 1:140 and a vertical scale of 1:22, which gives a distortion of 1:6.36. The model represented the area in the prototype between the levees and between ranges 118.15 and 116.26. A plan of the model is shown in Figure 4.

To duplicate the jacks and jetty field in the movable bed model, 1/2-inch-mesh galvanized screen (hardware cloth) was used. The screen was bent in a zigzag shape to give the same head loss as a line of jacks. The bent wire mesh was first tested in a fixed bed 1:16 scale model to determine the density of screen required to duplicate the velocity reduction from a line of jacks. The wire mesh required for the movable bed model was then designed to have a similar projected area per unit length for the distorted scale. Figure 5 shows the three ways of representing a line of jacks in the fixed bed and movable bed model studies.

The model was operated to simulate an averaged hydrograph in steps of 5,000 cubic feet per second with a maximum discharge of 15,000 cubic feet per second, and a minimum discharge of 5,000 cubic feet per second. The total volume of water discharged during the hydrograph period in the model simulated the total volume of water discharged in the prototype in a similar period.

Two model sediments were used to represent the prototype material. A white uniform sand with mean diameter of 0.2 millimeter represented the bedload, and a lightweight plastic represented the suspended load. The black color of the plastic material made it easy to distinguish between suspended and bedload deposits. The size analyses of these two sediments are shown in Figure 6. The settling velocity characteristics of the plastic material are shown in Figure 7. The control weir at the downstream end of the model was shaped to represent the natural river cross section.

Six tests were made, and at the end of each test, cross-section elevations of the movable bed were measured at six ranges in the model. To determine the effectiveness of the jetty fields the average elevation of the bed in the jetty field and the average elevation of the bed in the channel were compared at the end of each test. The relative change in bed levels was used as a measure of the effectiveness of the jetty field. The first test, with a discharge of 5,000 cubic feet per second, was used as the base for comparing scour and deposition produced by the succeeding tests using the simulated hydrograph.

The successive changes in the bed elevations and the difference between jetty field bed and channel bed were plotted after each of the successive five tests in the order 10,000 cubic feet per second, 10,000 cubic feet per second, 15,000 cubic feet per second, 10,000 cubic feet per second and 5,000 cubic feet per second. During the first four tests, the bed in the channel scoured and deposition occurred in the jetty field. During the last two tests when the hydrograph was receding, deposition occurred in the channel. Deposition in the channel was at a greater rate than deposition in the jetty field. The change in relative elevations of the bed in the channel to the bed in the jetty field was negative for the last two tests when compared to the first test of 5,000 cubic feet per second. These results duplicated the sedimentation action in the prototype.

Prototype Data Obtained at Pilot Study Reach

Considerable field data from the Casa Colorada pilot reach and other jetty fields near Albuquerque were furnished by the Middle Rio Grande Project. Data were obtained at range lines corresponding to measuring stations on the laboratory pilot model. Most of the data were obtained during the months of April through June which is the runoff season with the lowest sediment load. The field data consisted of river cross sections, discharge measurements, suspended sediment load and size analyses, bed material size analyses, and slope measurements.

Before a generalized study of tieback spacing was conducted, a thorough analysis of the prototype data was made, the purpose being to obtain knowledge of the action of the river, to verify the model, and to modify the theoretical scale ratios when inexact scaling was detected.

Manning's "n" values were computed assuming that the slope associated with each point was equivalent to the average slope across the measuring section. A plot of point depth versus point "n" value was made. Another plot of point velocity versus point "n" value was made and a third plot was made of point depth versus point velocity. Three curves were first fitted to the data by eye, then adjusted to be consistent with each other, giving equal weight to the depth and velocity measurements. The adjusted curves are shown in Figures 8, 9, and 10. The original curves were fitted to 150 data sets taken from points which were not near sudden changes in bed profile or a line of jacks. All sets of data were in the design channel. Twenty one sets of these data, selected in a random manner by rolling a die, were replotted to show the range of data and its scatter. These points are the circles in Figures 8, 9, and 10.

A plot of the average width versus the average area computed from discharge measurements was made. A gap was found in the field data for areas smaller than 500 square feet and it was necessary to approximate the lower part of the curve. Consequently, an average cross section was determined using the lowest discharge measurement at three stations and widths and areas for lower water surfaces were obtained assuming that the cross section did not change its shape for lower discharges. The curve in Figure 11 for areas smaller than 500 square feet, is the result of these calculations. The average depth of flow versus area curve, Figure 12, was obtained by dividing flow areas from the curve in Figure 11 by corresponding widths to obtain depths.

Using the point data curves and the area curves, the average river characteristics were computed and are shown in Figure 13, plotted against total discharge. The curves of Figures 11 through 13 show that the maximum scatter (values of depth and width) occurs near a discharge of 5,200 cubic feet per second. The average depth for this discharge is 2.0 feet and the average width 500 feet. This is the lowest flow that is difficult to control, and corresponds closely with the value of dominant or design discharge determined by Bureau hydrologists.

Sediment Analyses

From the velocity-discharge measurements, average velocity was computed for both jetty fields and design channels. These values of velocity were plotted against corresponding values of the 90-percent finer sizes of the suspended sediment samples, Figure 14. Since the jetty field data and the channel data had the same range of point scatter, a single average curve was drawn.

To determine the prototype's capacity for carrying suspended sediment, velocity versus sediment concentration was plotted. There was considerable scatter of the points. The curve was fitted to a second-degree polynomial, Figure 15, using the method of least squares.

Relative Deposition in Jetty Field

The average relative rate of deposition in the jetty field was computed from five prototype cross sections. Relative jetty field deposition is defined as the increase in difference of bed elevations in the design channel and jetty field. Deposition is positive when the jetty field elevation is higher than the channel. The average rate of relative jetty field deposition was computed to be 3.36 feet per year, of which 18 percent was channel scour. The prototype hydrograph for the deposition period is shown in Figure 16; the average discharge was 2,570 cubic feet per second.

GENERAL MODEL TIEBACK SPACING STUDY

Control Run and Exact Scale Factors

To compare jetty fields having different tieback spacings, the Casa Colorada pilot model was modified and tested. The flow boundaries in the model were changed to provide nearly constant jetty field and channel widths. The total levee-to-levee width represented was 1,500 feet. The downstream control was changed from a natural river cross section to a level weir.

A control test was made and used as a basis for comparing the effectiveness of the different tieback spacings. For the control run, the movable bed was reshaped to a plane surface that sloped to the control weir; no jetties were installed. Velocity, depth, slope, and total discharge measurements were made. Using these model measurements and the prototype data, exact scale factors were determined for the model which were then plotted against the unit discharge used in the control run. The procedure is essentially equivalent to plotting scale factors versus Reynolds number since the unit discharge is equal to the velocity times depth, and the viscosity and density remain constant.

Tieback Spacing Tests

After the exact scale ratios had been determined from the control run and field data, the jetty field was installed in the model and tieback spacings of 250 and 500 feet were tested. For each test the model bed was remolded to the contours that existed prior to the control run. The design channel was 600 feet wide and the jetty fields extended an additional 450 feet on each side of the channel. The unit discharge was used to determine the proper scale ratios from the control test and prototype data.

Core samples 4 inches in diameter were taken from the jetty fields and the channel. Figure 17 shows three cores taken at an upstream range line after the test conducted with 250-foot tieback spacing. The core showing the greatest depth of black plastic sediment was on the inside of the curve. Deposits in the jetty field on the inside of the curve tended to crowd the flow into the channel and into the jetty field on the outside of the curve. The results of these tests indicated that jetty field installations should be constructed only on the outside of curves, initially, and should be constructed on the inside of curves only when the need develops.

FRICTION FACTOR ANALYSIS

A friction factor analysis was made of the tieback spacing data obtained in the model tests. Because the friction factor expressed by the Darcy-Weisbach equation is dimensionless it was used to provide data for both model and prototype uses. For open channel flow, assuming that the hydraulic radius is equal to depth, the friction factor may be expressed:

$$f = 8g\left(\frac{SY}{V^2}\right)$$

where

g = acceleration of gravity
S = slope
Y = depth
V = velocity

By substitution in this equation friction factors were computed and related to unit discharge with the relative tieback spacing $\frac{X}{Y}$ used as a parameter, Figure 18; X is the tieback spacing and Y is the depth in the jetty field. Figure 18 shows that the values of "f" become constant for discharges greater than about 10 cubic feet per second per foot of width corresponding to values of Reynolds numbers near 1×10^6 and greater. These curves are typical of friction head loss. For convenience of design computations described in Appendix II, the same data are presented in terms of percent increase of friction, Figure 19.

PREDICTING JETTY FIELD DEPOSITION

Using field data from a proposed jetty field site and the friction factor analysis of this study, predictions can be made of the rate of jetty field deposition. Details of an example computation are given in Appendix II. In this computation for a 2-mile long jetty field on each side of a 500-foot design channel, the tieback spacing is 250 feet and the levee-to-levee river width is 1,400 feet. Using the preinstallation river characteristics similar to those in the Casa Colorada reach, the computed average rate of relative jetty field deposition was found to be 2.16 feet per year.

SUMMARY AND CONCLUSIONS

Both model and prototype studies indicated that jetty fields are successful in alining rivers that carry appreciable amounts of suspended sediments. To compute the relative rate of sediment deposition, field data are required. The use of point data analysis, described in this report, aids greatly in extending field data and establishing river characteristic curves.

To determine the design discharge, it is recommended that width and depth be plotted versus flow area and that a value be selected which corresponds to the flow area having the maximum scatter or deviation of data points with respect to depth and width. This is the lowest discharge which will be difficult to control. The average river width corresponding to this discharge should be used for the design channel width.

The relative rate of deposition in a jetty field installation can be computed by the procedure demonstrated in this paper. However, future studies may indicate desirable modifications and refinements in the procedures.

APPENDIX I

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APPENDIX II

A METHOD OF COMPUTING DEPOSITION IN A JETTY FIELD

Jetty field installations can be designed using field data and the friction factor analysis presented earlier in this paper. To illustrate the method of predicting the changes in the riverbed when a jetty field is installed, a sample problem pertaining to channelizing a reach of the Middle Rio Grande is explained.

Tables 1, 2 and 3 at the end of Appendix II show the step by step procedures for making the calculations and tabulating the results in convenient form.

The basic data needed to make a prediction of the scour and deposition in the bed of a jetty field installation are similar to the prototype data analyzed earlier in this paper. The method of point data analysis previously described, makes it easier to obtain curves similar to Figure 13 which give the preinstallation river characteristics, and Figure 20 which gives friction factor, depth and velocity in terms of unit discharge. The point data method also helps to compensate for the usually small range of data values available during a field reconnaissance program. For the prediction computation, the sediment carrying capacity is most conveniently expressed in terms of velocity versus concentration, Figure 15. The geometry of the flow section, similar to the curves in Figures 11 and 12, aid in selecting the design discharge and its corresponding channel width. A flow-duration curve for the river at the proposed jetty field installation site is also needed.

For the example computations given in Tables 1, 2 and 3, it is assumed that the designer has the above basic data in his possession and that the proposed site has the same characteristics as the pilot study reach described earlier in the paper. Assume, then, that the designer desires to investigate a proposed installation with a length of 2 miles, a levee-to-levee width of 1,400 feet and a tieback spacing of 250 feet.

First select a design discharge and corresponding design channel width. The deviations in width and depth in Figures 11 and 12 show that the design width should be approximately 500 feet. The corresponding river discharge of approximately 5,000 cubic feet per second is determined from Figure 13. This width corresponds to the lowest discharge that is most difficult to control.

For convenience, and to simplify the computations, the process of scour and deposition is assumed to occur nonconcurrently with deposition occurring only in the jetty field and scour occurring only in the channel. Both deposition and scour are considered as relative deposition. In actual installations, deposition and scour may or may not occur at the same time. The information desired is the average rate of relative deposition in the jetty field for the selected tieback spacing, and the time for the river to become completely channelized.

Table 1 is a computation table for determining the average relative deposition for the jetty field reach for a typical year. This is done by determining the average relative deposition, Column 2, Table 1, for steps in discharge as related to a flow duration curve for the river. In the example, prototype data were available to determine the average rate of deposition for a discharge of 2,570 cubic feet per second. The yearly rate of deposition amounted to 3.36 feet per year assuming the discharge of 2,570 cubic feet per second was constant throughout the year. For other values of constant river discharge shown in Column 1, the average relative deposition, Column 2, can be computed by the method explained below and in Tables 2 and 3. For the example discussed here, average relative deposition was computed for 7,500 and 10,800 cubic feet per second. For the other values of discharge, deposition rates were extrapolated or interpolated from a plotted curve, Column 3, Table 1. The average rates of deposition for the intervals between discharges are listed in Column 4. The corresponding percent of time on the flow duration curve for the discharge interval is entered in Column 5. The flow duration curve for the Rio Grande gaging station near Bernardo, New Mexico, as compiled for the years 1936 through 1954 was used in this example. The weighted average deposition for each discharge interval, Column 6, is obtained by multiplying the values in Column 4 by the values in Column 5. A summation of Column 6 divided by 100 gives the average rate of relative deposition (deposition in the jetty field and/or scour in the channel) for a typical year. The rate for this example is 2.16 feet per year.

The method for computing the average rates of relative deposition, Column 2, Table 1, for the discharge steps in Column 1 will now be described. This is the basic process for computing deposition in a jetty field. The computations are shown in Tables 2 and 3. For the remainder of the example, only the computations for the constant river discharge of 10,800 cubic feet per second will be explained.

Table 2 shows an example computation for determining the resulting depth in the jetty field for an assumed unit discharge in the design channel. The example is for 10 cubic feet per second per foot in the channel; the resulting depth is plotted in Figure 20. Since the channel is 500 feet wide, 5,000 cubic feet per second is flowing in the channel leaving 5,800 cubic feet per second in the jetty fields. This quantity expressed in terms of unit discharge is 6.44 cubic feet per second for two jetty fields each 450 feet wide, Columns 1 and 2.

Various depths of flow are assumed for the jetty field, Column 3. From these depths, velocities are computed, Column 4. The friction factor corresponding to hydraulic characteristics given in Columns 1 through 4 and using an average slope of 0.000829 is computed and entered in Column 5. Ratios of assumed depths to the tieback spacing are computed and are listed in Column 6. The percentage increase in friction factor caused by the tiebacks at the assumed depths is determined from Figure 19 and appears in Column 7. The friction factor that would occur at a unit discharge of 6.44 cubic feet per second with respect to the bed roughness alone, without a jetty field, is 0.017. This value can be read from Figure 18 or 20, and when multiplied by one plus the decimal percentage of friction increase, results in the combined friction for the bed and tiebacks. The depth can be determined by interpolation or by cross plotting Column 5 and 9 versus Column 3 on the same coordinates. The jetty field depth for this example is 1.86 feet. This depth and the corresponding velocity were plotted on Figure 21 for the unit discharge of 10 cubic feet per second in the design channel. The values of channel depth and channel velocity for this same unit discharge are determined from Figure 20. To obtain curves similar to those shown in Figure 21, computations must be made for other values of unit discharges in the channel.

The computation of the time rate of relative deposition for the constant river discharge of 10,800 cubic feet per second appears in Table 3. Values of unit discharge (q_c) in Column 1, Table 3 for the design channel are assumed for the range indicated in Figure 21. The average channel unit discharge for each interval is entered in Column 2. By continuity and using the design widths, values of the unit discharge (q_f) for the jetty field are determined, Column 3. For each of the assumed values of q_c, the differences in depth between the channel and jetty field (Δy) are computed from values determined in Figure 21 and are listed in Column 4.

From Column 4, the second differences of depth ($\Delta^2 y$) for the unit discharge intervals are computed and appear in Column 5. The values of channel and jetty field discharges and velocities were determined from the continuity equation, the selected design width, and Figure 21. The values for these variables appear in Columns 6 through 9. From Figure 15, the corresponding values of suspended sediment concentration in parts per million by weight in the channel and jetty field are entered in Columns 10 and 11, respectively. The QC terms (discharge x concentration) in Columns 12, 13, and 14 are proportional to the sediment-carrying capacities for the design channel, jetty field and the whole channelized river reach. The QC term in Column 15 is proportional to the river sediment-carrying capacity prior to installation of the jetty field. This term is constant since the computation concerns one total discharge. Whether there is deposit in the jetty field or scour in the channel is noted in Column 16.

The rate of scour and/or deposition, whichever occurs is proportional to the values of the difference between the river sediment-carrying capacity before and after channelization. The sediment-carrying values times a constant (1.2) times the unit weight of water divided by the unit weight of the sediment gives the inplace volume of the solids scoured or deposited per second, Column 20. The factor (1.2) is an approximate value and is used to convert suspended load to total load. In Column 18 are the areas associated with the scour and deposition. These areas times the change in depth difference, Column 5, result in the volumes expected to be scoured or deposited at the rate given in Column 20. These volumes are entered in Column 19. Dividing the values in Column 19 by those in Column 20 gives the incremental times (Δ T) to deposit or scour the incremental volumes, Column 21. The sum of the values in Column 21 is the total time for the flow to be completely channelized for the discharge that had originally covered the total jetty field installation width. This total time is 2.77×10^6 seconds. The average relative rate of jetty field deposition is 37.2 feet per year for the constant river discharge of 10,800 cubic feet per second. This rate was entered in Table 1. In the example, the relative deposition (difference in channel and jetty field bed elevations) amounted to 3.27 feet in 32 days to reach complete channelization at a constant discharge of 10,800 cubic feet per second.

FIGURES

APPENDIX II

Table 1

COMPUTATION TABLE FOR AVERAGE RATE OF RELATIVE JETTY FIELD DEPOSITION FOR HYDROGRAPH YEAR

1	2	3	4	5	6
Discharge cfs Q	Average relative deposition ft/yr R	Method of obtaining relative deposition	Average deposition for time interval ft/yr	Percent of time from flow duration curve	Weighted average deposition times 100 4 X 5
0	0				
			0.00	2.60	0.00
10	0				
			0.00	83.40	0.00
2,000	0	Extrapolated			
			1.68	3.20	5.38
2,570	3.36	Field Data			
			9.68	5.20	50.34
5,000	16.0	Interpolated			
			21.40	2.50	53.50
7,500	26.8	Computed			
			32.00	2.15	68.80
10,800	37.2	Computed*			
			41.40	0.60	28.84
15,000	45.6	Extrapolated			
			47.00	0.18	8.46
17,000	48.4	Extrapolated			
			49.90	0.10	4.99
20,000	51.4	Extrapolated			
			--	0.07	--
				100.00	216.31

*See sample computations in Table 3.

Average rate of deposition = $\frac{216.31}{100}$ = 2.16 feet per year.

APPENDIX II

Table 2

COMPUTATION TABLE FOR ONE DESIGN
CHANNEL UNIT DISCHARGE IN FIGURE 21

Design channel	Jetty field							
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Unit discharge cfs/ft	Unit discharge cfs/ft	Y_f depth ft	V_f velocity ft/sec	Friction factor f_f	Y_f/X	$\%(f_f)$	f_{fb}	$f_{y/x}$
10	6.44	1.00	6.44	0.0052	0.004	29	0.017	0.0219
10	6.44	1.50	4.29	0.0174	0.006	53	0.017	0.0260
10	6.44	2.00	3.22	0.0411	0.008	91	0.017	0.0325
10	6.44	2.50	2.58	0.0802	0.010	121	0.017	0.0376

Tieback spacing (X) = 250 feet
Channel width = 500 feet
Levee-to-levee width = 1,400 feet
Total river discharge = 10,800 cfs

(1) q_c assumed = 10 cfs/ft

(2) q_f is then = 6.44 cfs/ft

(3) Y_f = assumed jetty field depth.

(4) $V_f = q_f/Y_f$

(5) $f_f = 8gS \frac{Y}{V^2}$; where slope = 0.000829

(6) $Y_f/X = Y_f/250$

(7) $\%(f_f)$ = percent increase in friction caused by tiebacks--from Figure 19

(8) f_{fb} = friction with respect to bed alone at q_f = 6.44 cfs/ft--from Figure 18 or 20

(9) $f_{fb}\,(1 + \%f_f/100)$ = friction for bed and tiebacks combined

Depth in jetty field for q_c = 10 is 1.86 feet determined by interpolating or by cross plotting (5) and (9) with respect to (3) .

APPENDIX II

Table 3

COMPUTATION TABLE FOR AVERAGE RATE OF RELATIVE JETTY FIELD DEPOSITION FOR TOTAL RIVER DISCHARGE OF 10,800 CFS

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
q_c cfs/ft	q_c av cfs/ft	q_f av cfs/ft	ΔY ft	$\Delta^2 Y$ ft	Q_c cfs	Q_f cfs	V_c ft/sec	V_f ft/sec	C_c ppmw	C_f ppmw	$\frac{Q_cC_c}{10^6}$	$\frac{Q_fC_f}{10^6}$	12 + 13	$\frac{Q_rC_r}{10^6}$	Remarks	1.2(15 - 14)	Area 10^6 Ft2	Vol 10^6 Ft3	V_{ss} Ft3/sec	$\frac{\Delta T}{10^6}$ seconds
9.5			0.00																	
	9.75	6.58		0.11	4,875	5,925	4.97	3.40	7,700	4,350	37.5	25.8	63.3	74	Deposit	12.8	9.5	1.04	8.87	0.12
*10.0			0.11																	
	11.0	5.88		0.45	5,500	5,300	5.16	3.29	8,200	4,150	45.1	22.0	67.1	74	Deposit	8.28	9.5	4.28	5.74	0.75
12.0			0.56																	
	13.0	4.77		0.46	6,500	4,300	5.50	3.07	9,150	3,850	59.5	16.6	76.1	74	Scour	-2.52	5.28	2.38	1.74	1.37
14.0			1.02																	
	15.0	3.66		0.48	7,500	3,300	5.80	2.75	10,050	3,400	75.4	11.2	86.6	74	Scour	-15.1	5.28	2.53	10.5	0.24
16.0			1.50																	
	17.0	2.55		0.46	8,500	2,300	6.05	2.35	10,900	2,950	92.6	6.78	99.4	74	Scour	-30.5	5.28	2.43	21.1	0.12
18.0			1.96																	
	19.0	1.44		0.50	9,500	1,300	6.25	1.70	11,650	2,300	111.0	2.99	114.0	74	Scour	-48.0	5.28	2.64	33.3	0.08
20.0			2.46																	
	20.8	0.44		0.81	10,400	400	6.46	0.65	12,400	1,000	129.0	0.40	129.0	74	Scour	-66.0	5.28	4.28	45.7	0.09
21.6			3.27																	
				$\Sigma = 3.27$																$\Sigma = 2.77 \times 10^6$

* See example computation Table 2

c) (design channel
f)Subscripts denoting (jetty field
r) (unchannelized river

q = Unit discharge, 1 2 3

Y = Depth, Figure 21

$\Delta Y = Y_c - Y_f$, $\Delta^2 Y$ is second diff. 4 5

Q = Discharge, 6 7

$Q_r = Q_c + Q_f$ = 10,800 cfs

River width = 1,400 feet

Channel width = 500 feet

Channelization length = 2 miles

Tieback spacing = 250 feet

V = Velocity 8 9 Figure 21

C = Concentration 10 11 , Figure 15

QxC terms are proportional to suspended sediment carrying capacities 12 13 14 15

17 proportional to sediment that will be scoured or deposited

1.2 is factor that converts suspended load to total load

18 is the area that is aggraded or degraded

19 = Volume scoured or deposited

V_{ss} = Volume of sediment scouring or settling per second.

ΔT = Time used in obtaining 19

APPENDIX III

NOTATION

b = Subscript denoting bed

c = Subscript denoting channel

C = Sediment concentration parts per million by weight

$\Delta y = y_c - y_f$ = Difference in depth between channel and jetty field

$\Delta^2 y$ = Second difference of depth

f = Darcy-Weisbach friction factor

= Subscript denoting jetty field

= A function

$\emptyset$ = Angle between a tieback line and a front line

g = Acceleration of gravity

N_F = Froude Number = $\frac{Vo}{\sqrt{gy}}$

n = Manning's friction factor

o = Subscript denoting upstream from jetty field

q = Unit discharge, cubic-feet-per-second per foot of width

Q = Total discharge, cubic feet per second

r = Subscript denoting river

R = Subscript denoting reduction

= Relative rate of jetty field deposition

S = Slope

T = Time

V = Velocity

V = Volume

X = Tieback spacing or distance downstream from a tieback

Y = Depth of flow

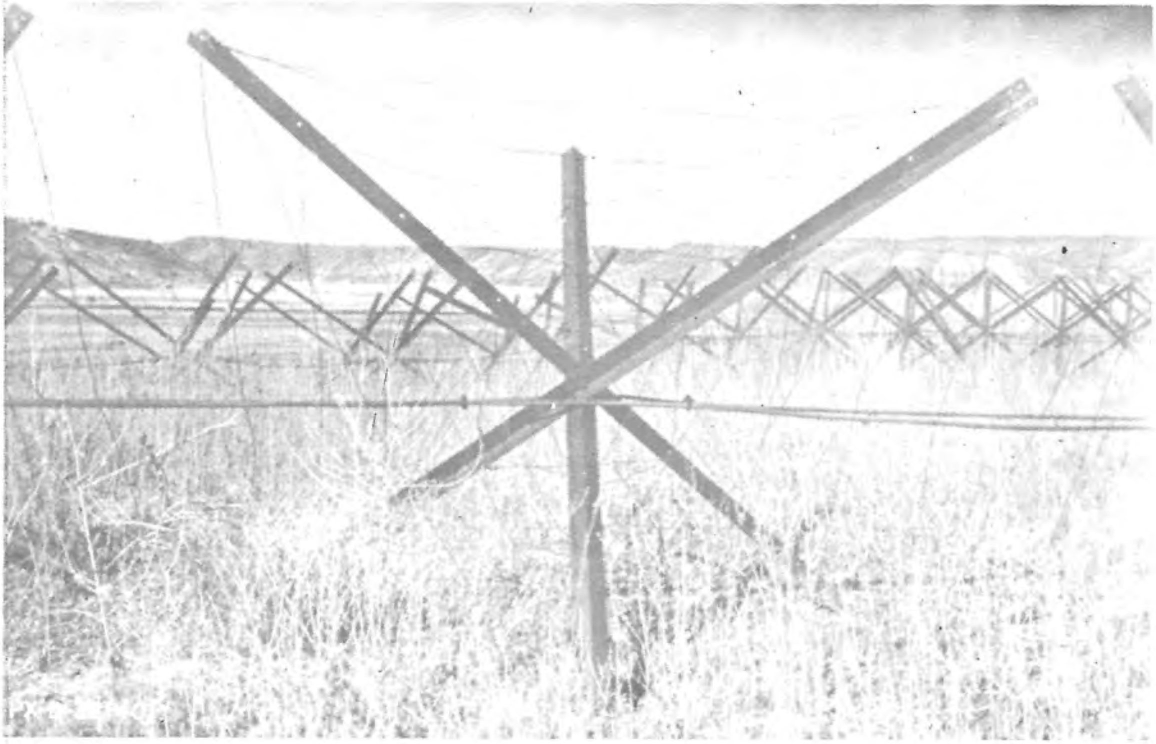
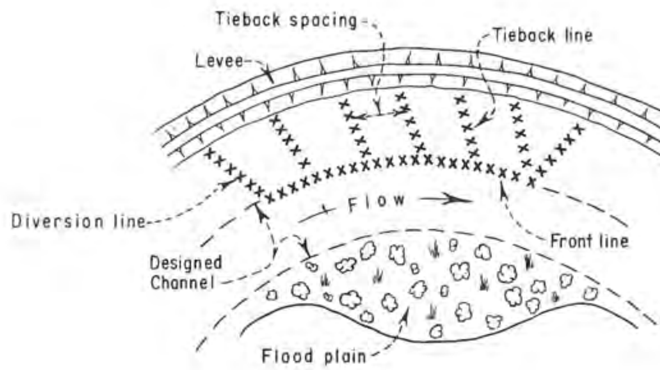
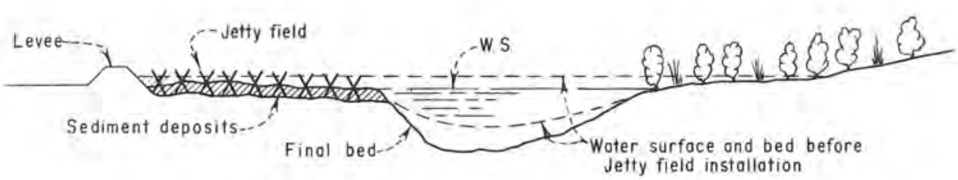


FIG. 1. --A SINGLE JACK UNIT



PLAN AT JETTY FIELD



CROSS SECTION THROUGH JETTY FIELD

FIG. 2.— PLAN AND CROSS SECTION OF JETTY FIELD

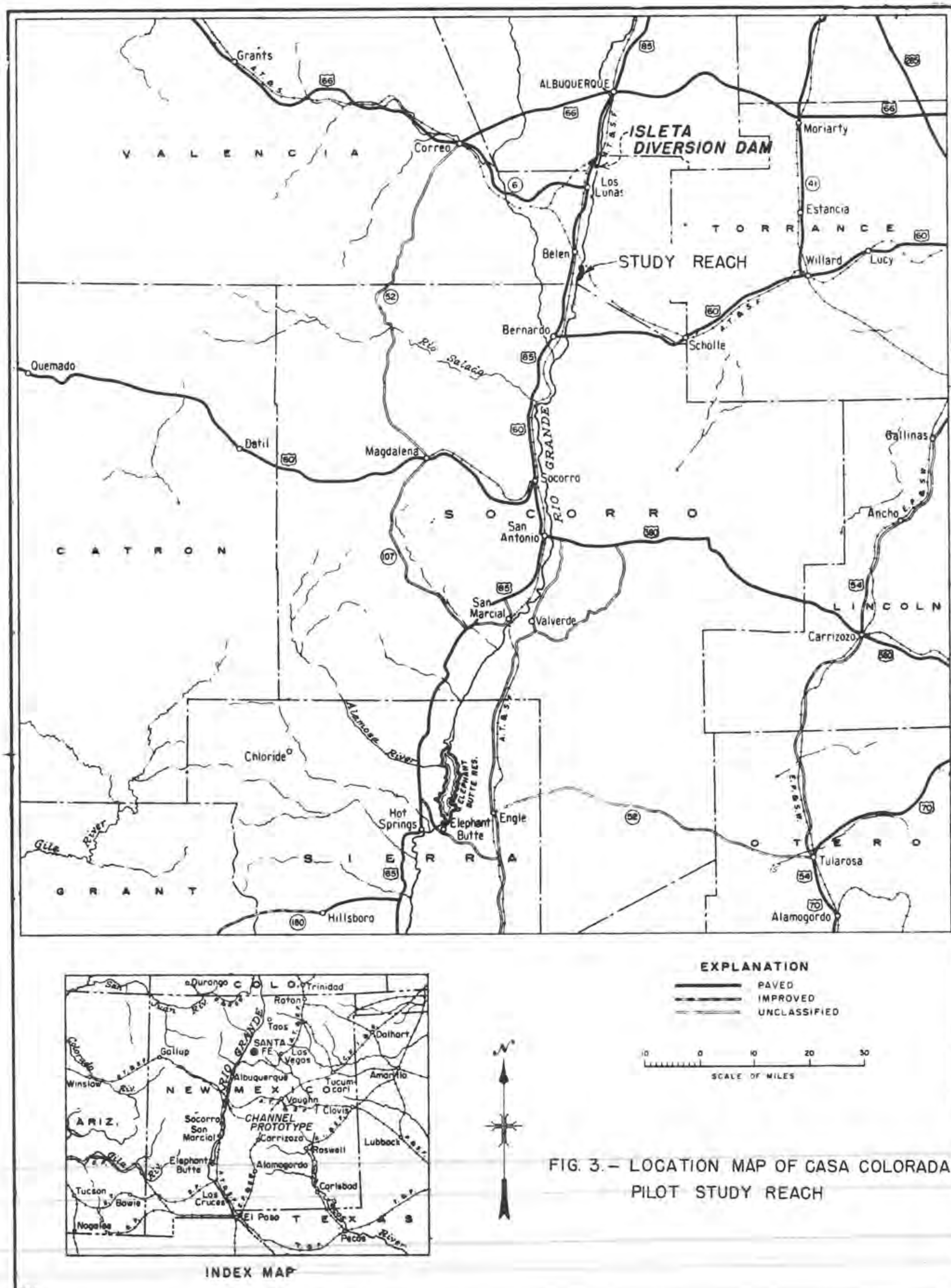


FIG. 3.— LOCATION MAP OF CASA COLORADA PILOT STUDY REACH

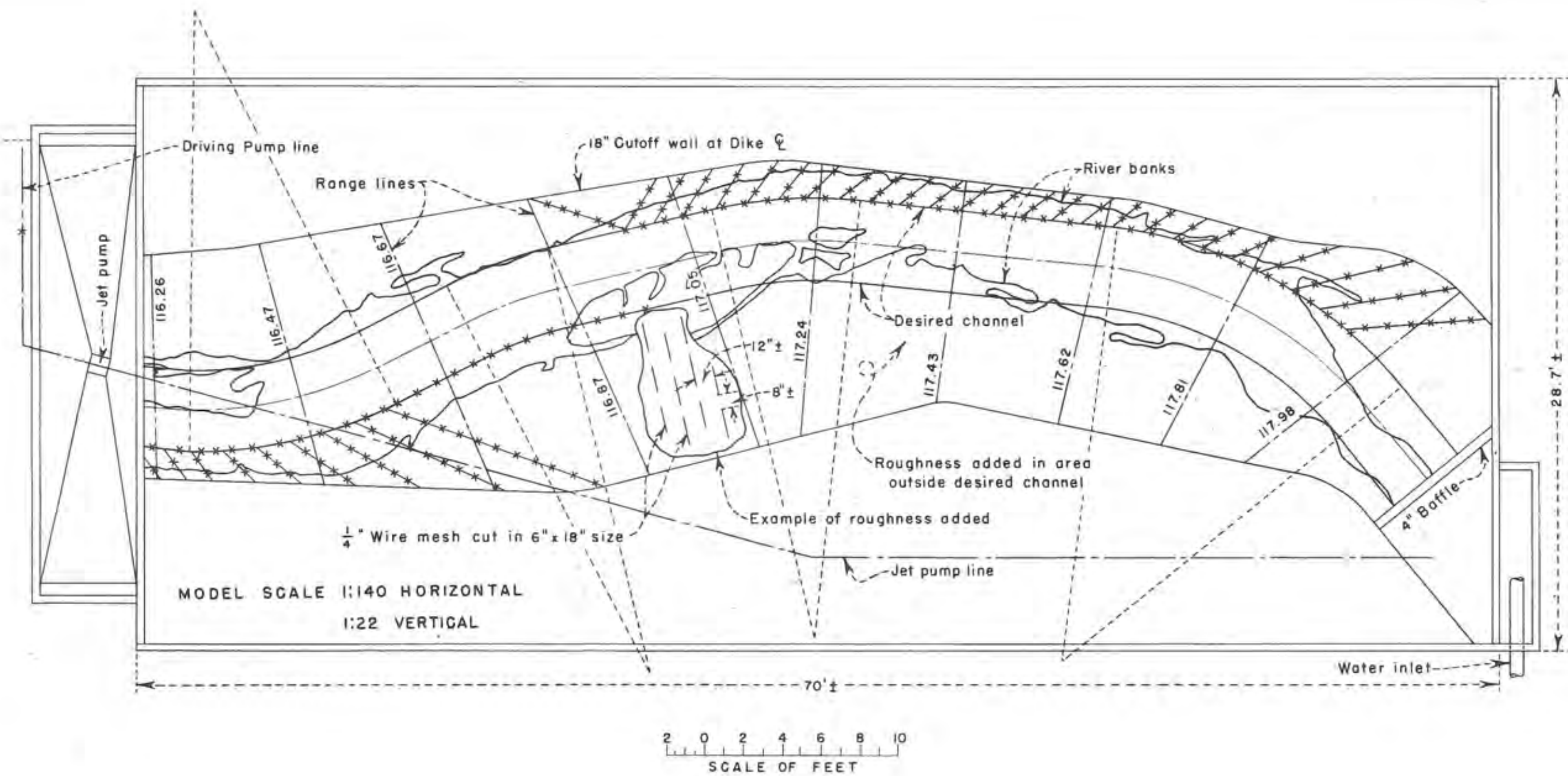
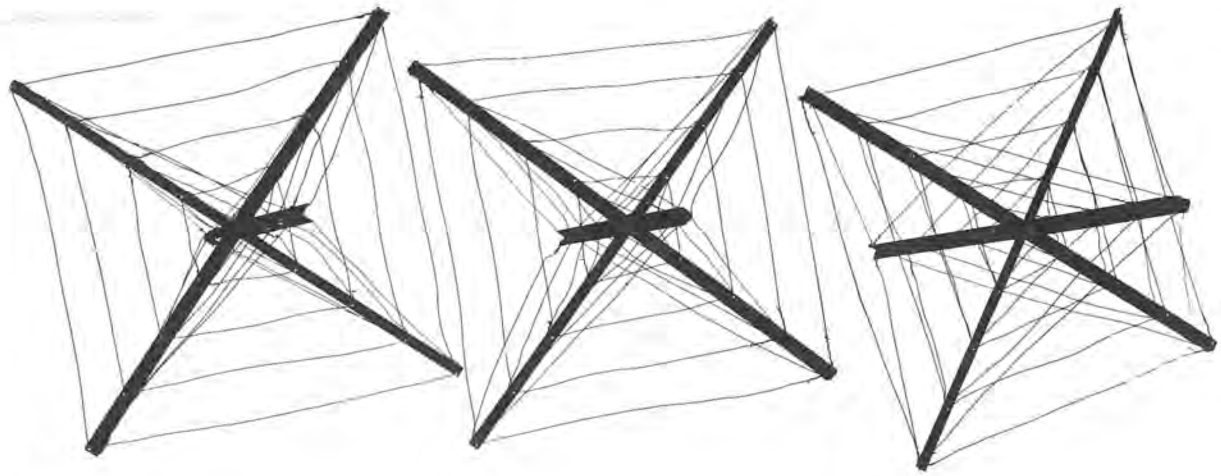
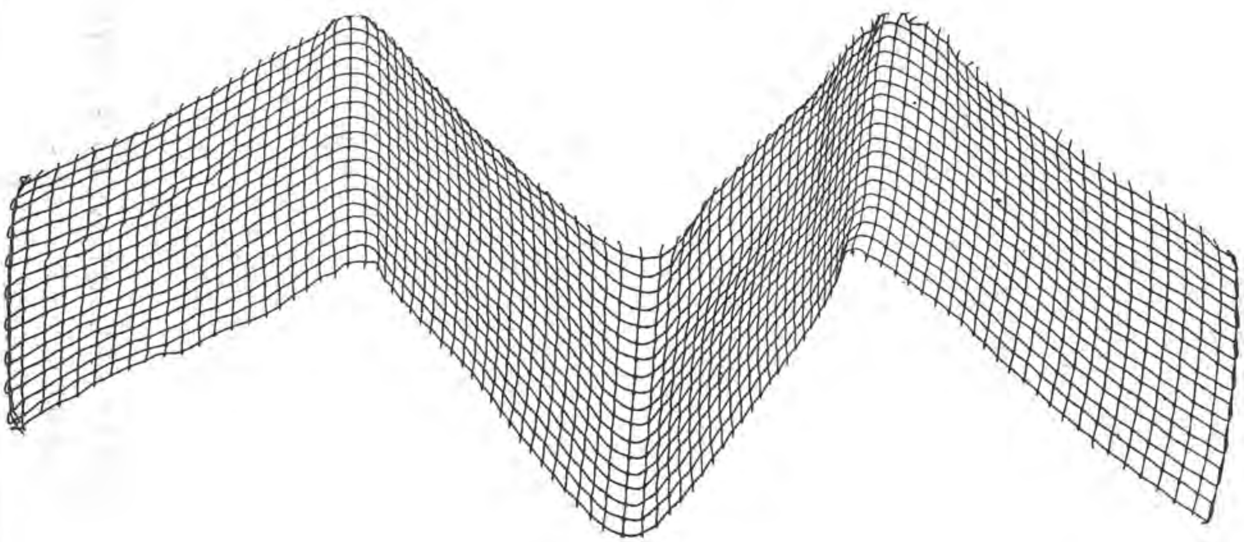


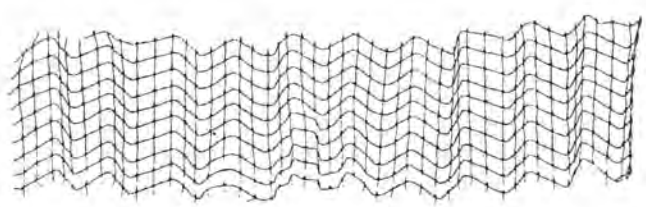
FIG. 4.– LAYOUT OF CASA COLORADA RIVER REACH MOVABLE BED MODEL



(a) Line of 1:16 Scale Jacks

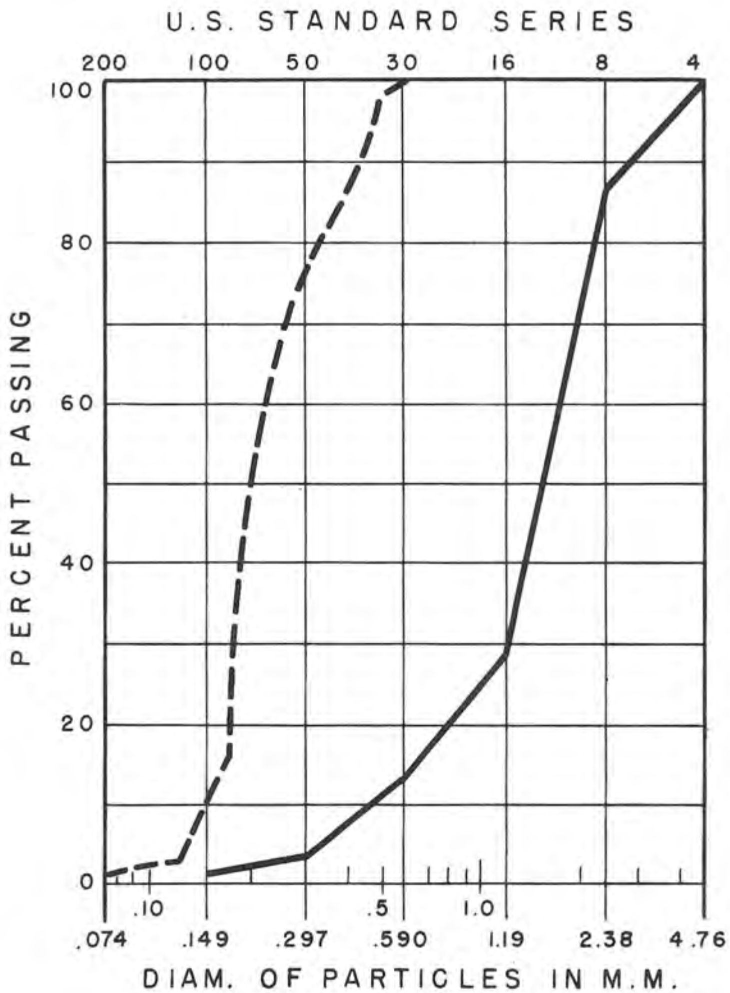


(b) Line of Jacks Represented by 1/2"
Wire Mesh, for 1:16 Scale Model



(c) Line of Jacks Represented by 1/2"
Wire Mesh, for Movable Bed Model; Scale
1:140 Horizontal, 1:22 Vertical

FIG. 5. --THREE WAYS OF REPRESENTING A LINE
OF JACKS IN THE MODEL STUDIES



NOTES

- - - - Sand used in river model.

——— Plastic used in river model.

FIG. 6.— SIZE ANALYSES OF SEDIMENTS
MOVABLE BED MODEL

564

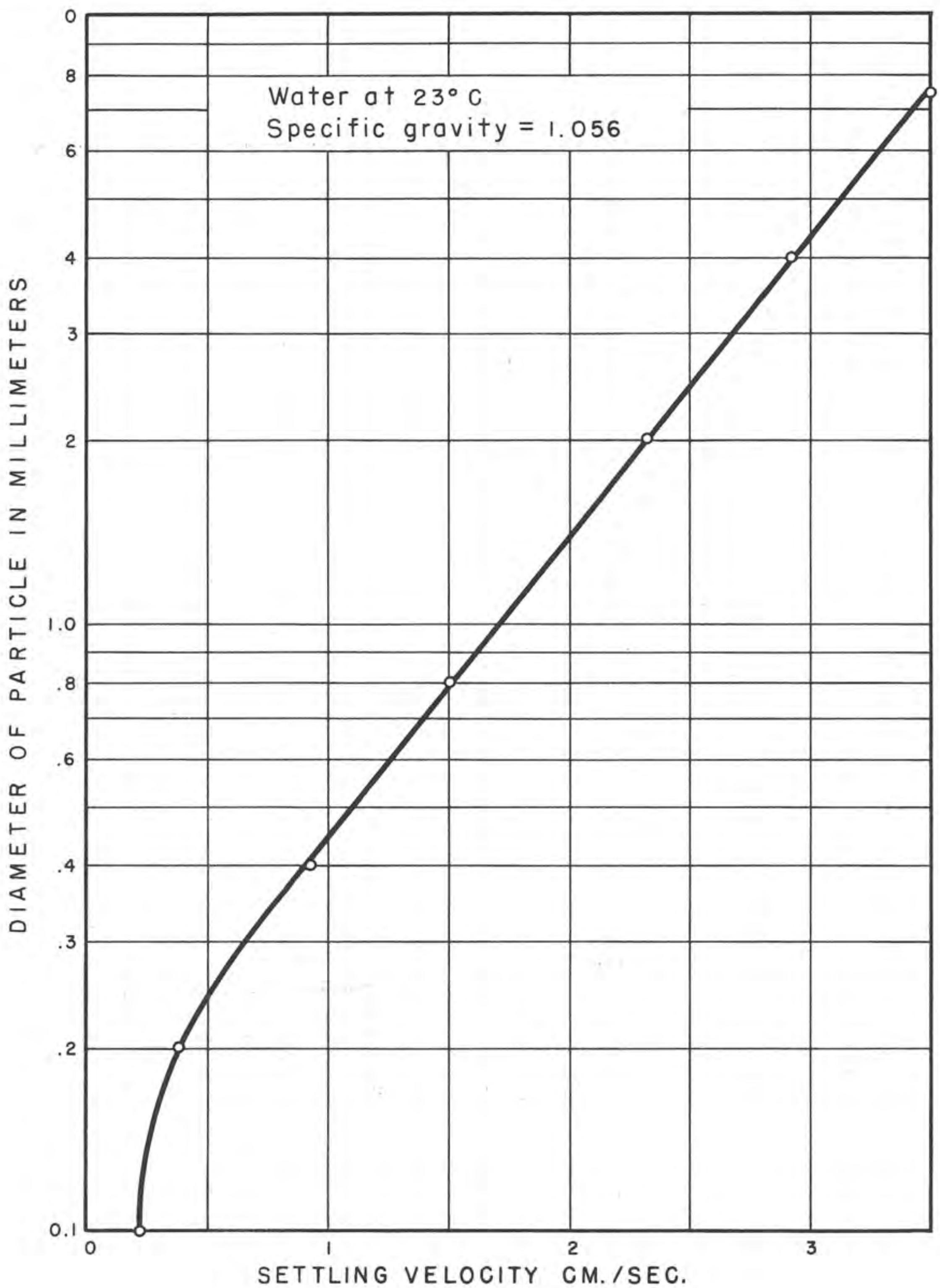


FIG. 7. —

SETTLING VELOCITY CHARACTERISTICS OF PLASTIC MATERIAL

MOVABLE BED MODEL

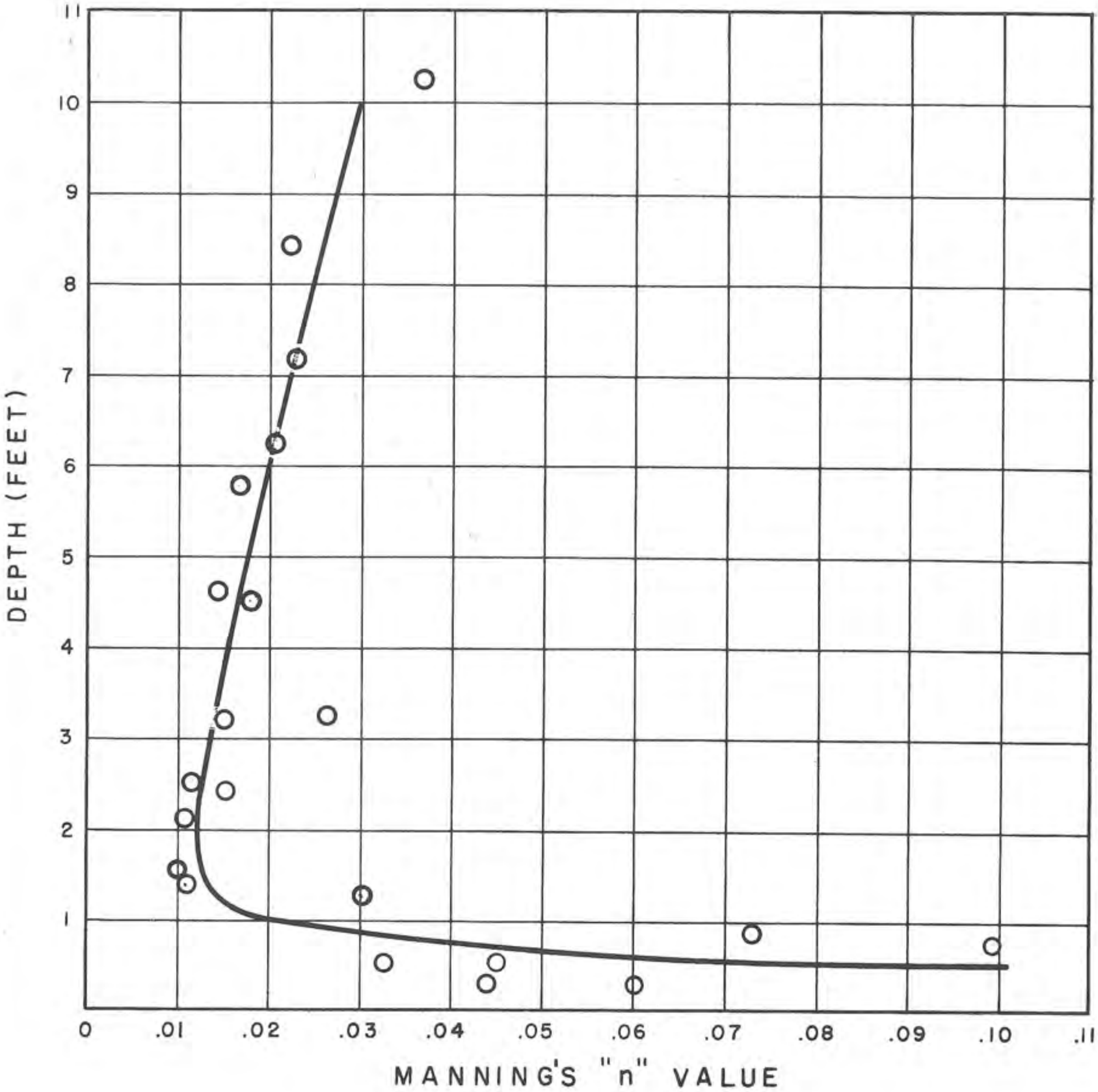


FIG. 8. MANNING'S "n" VALUE VS. DEPTH
PROTOTYPE DATA

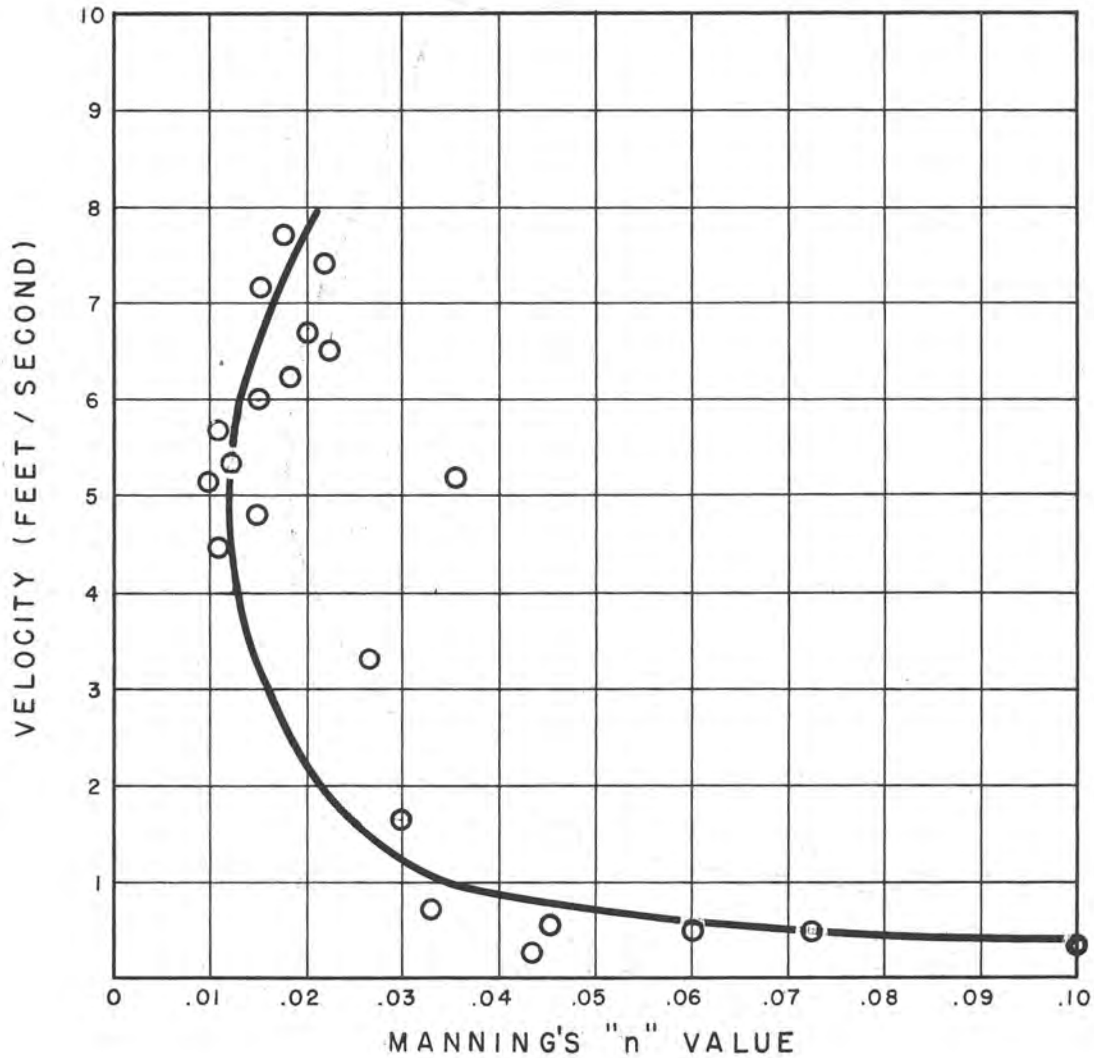


FIG. 9. MANNING'S "n" VALUE VS. VELOCITY
PROTOTYPE DATA

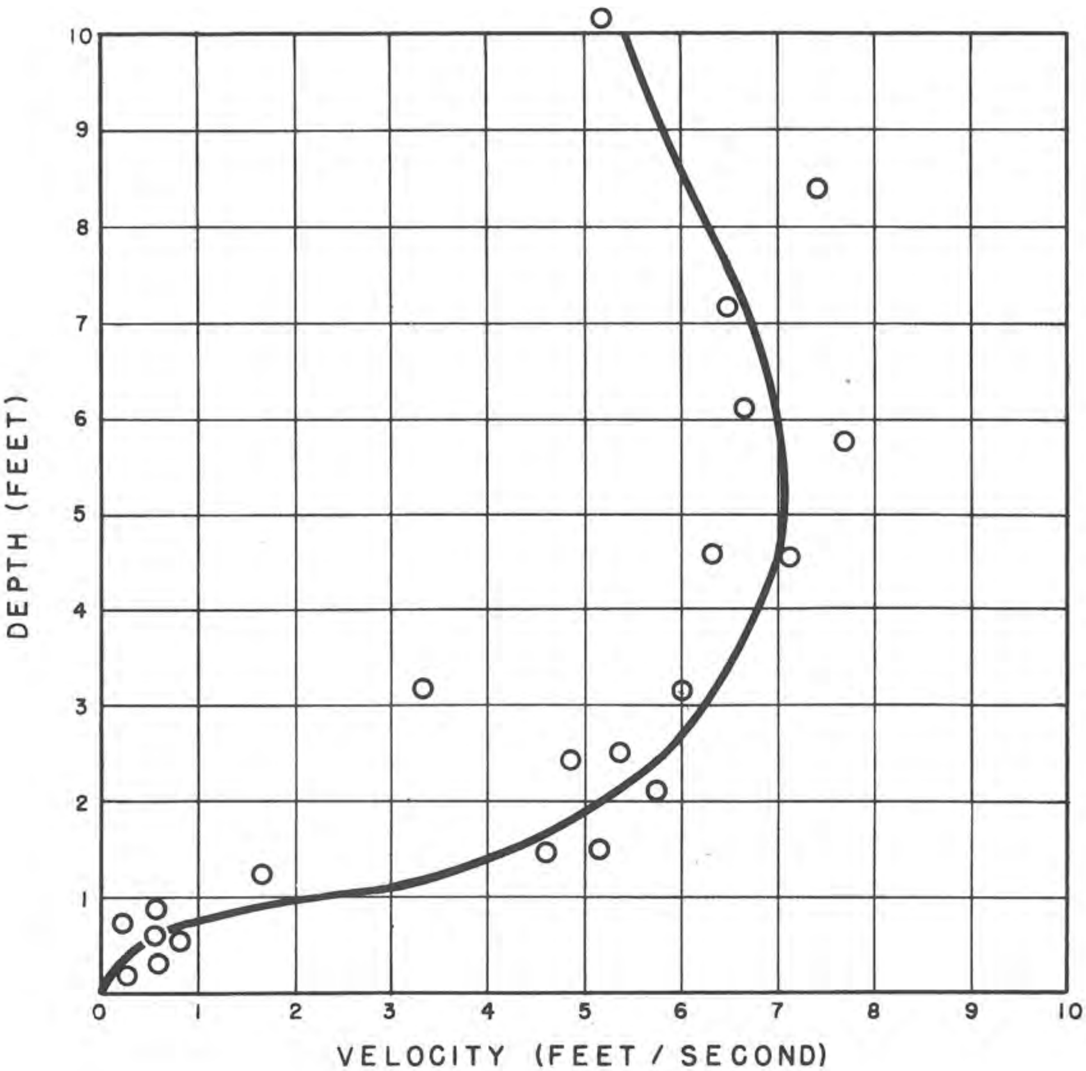


FIG. 10.— DEPTH VERSUS VELOCITY
PROTOTYPE DATA

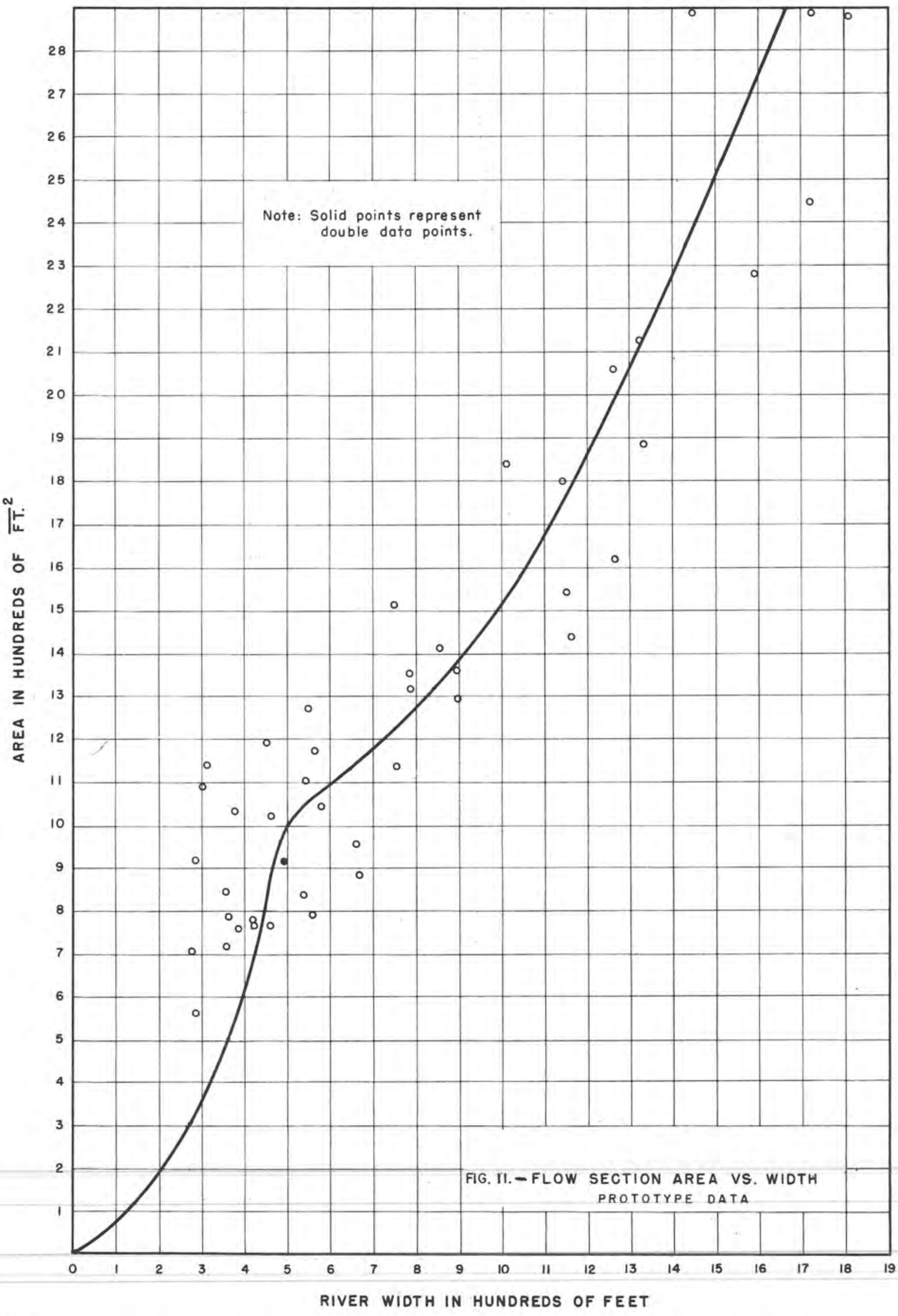


FIG. 11. — FLOW SECTION AREA VS. WIDTH
PROTOTYPE DATA

564

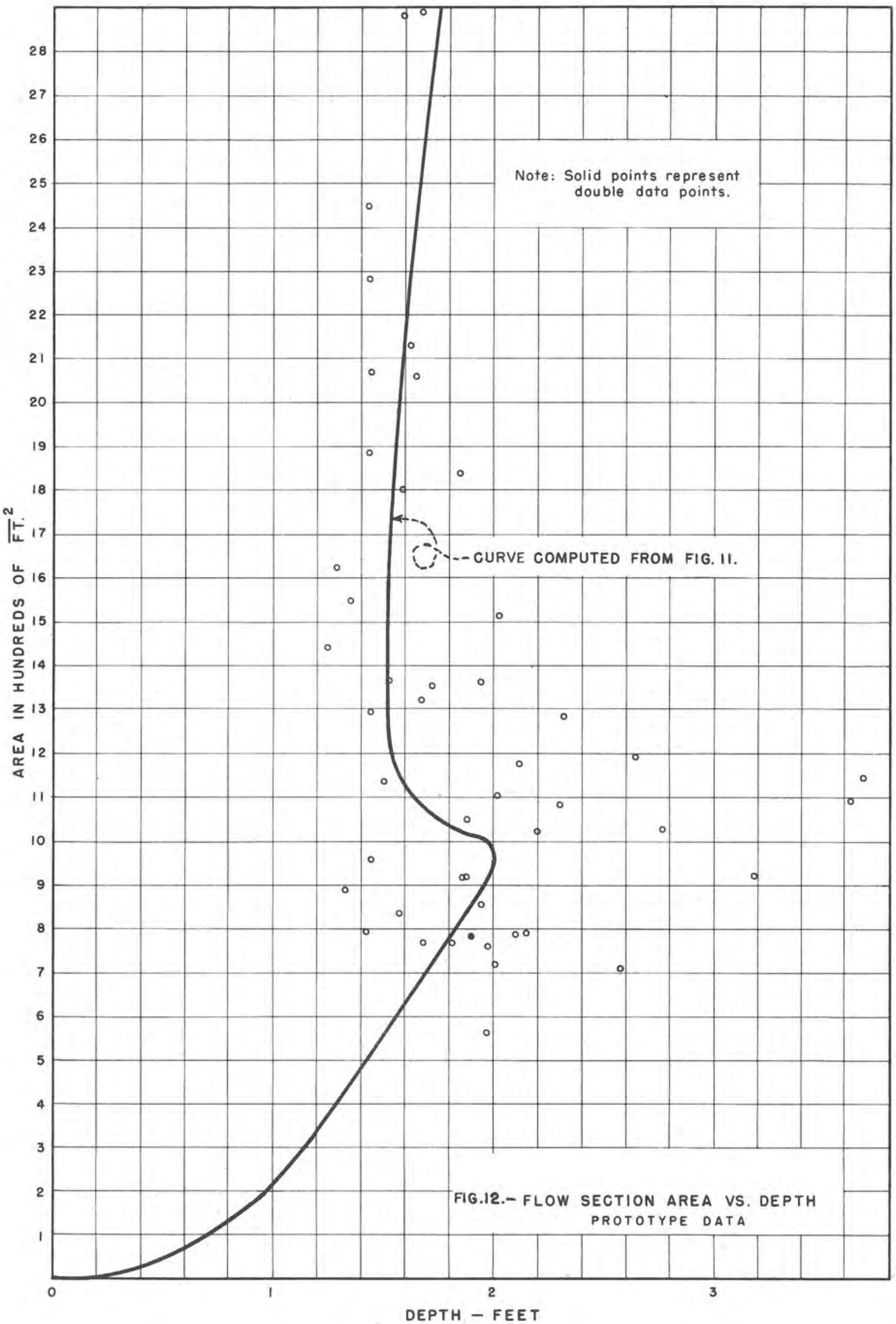


FIG. 12.— FLOW SECTION AREA VS. DEPTH
PROTOTYPE DATA

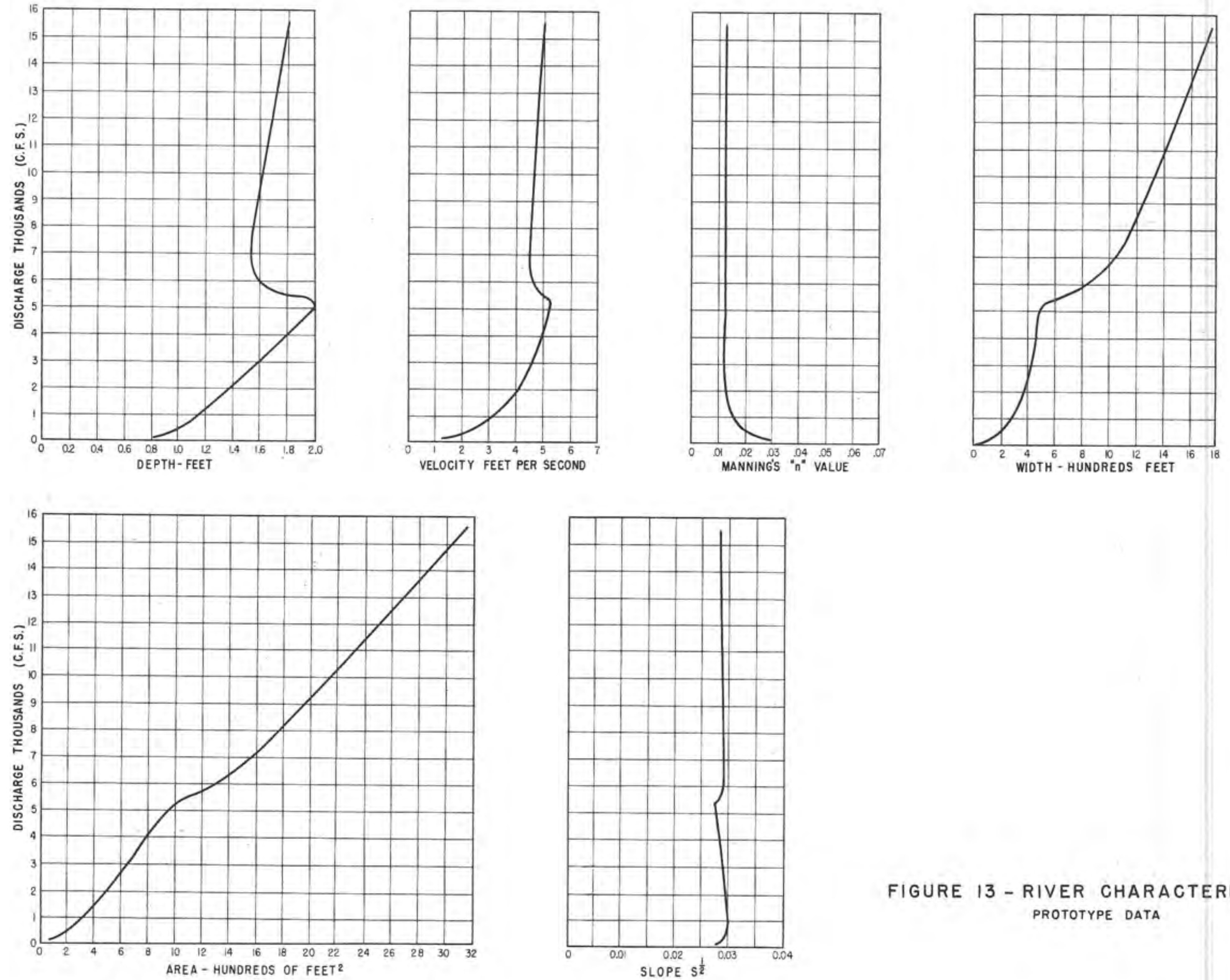


FIGURE 13 - RIVER CHARACTERISTICS
PROTOTYPE DATA

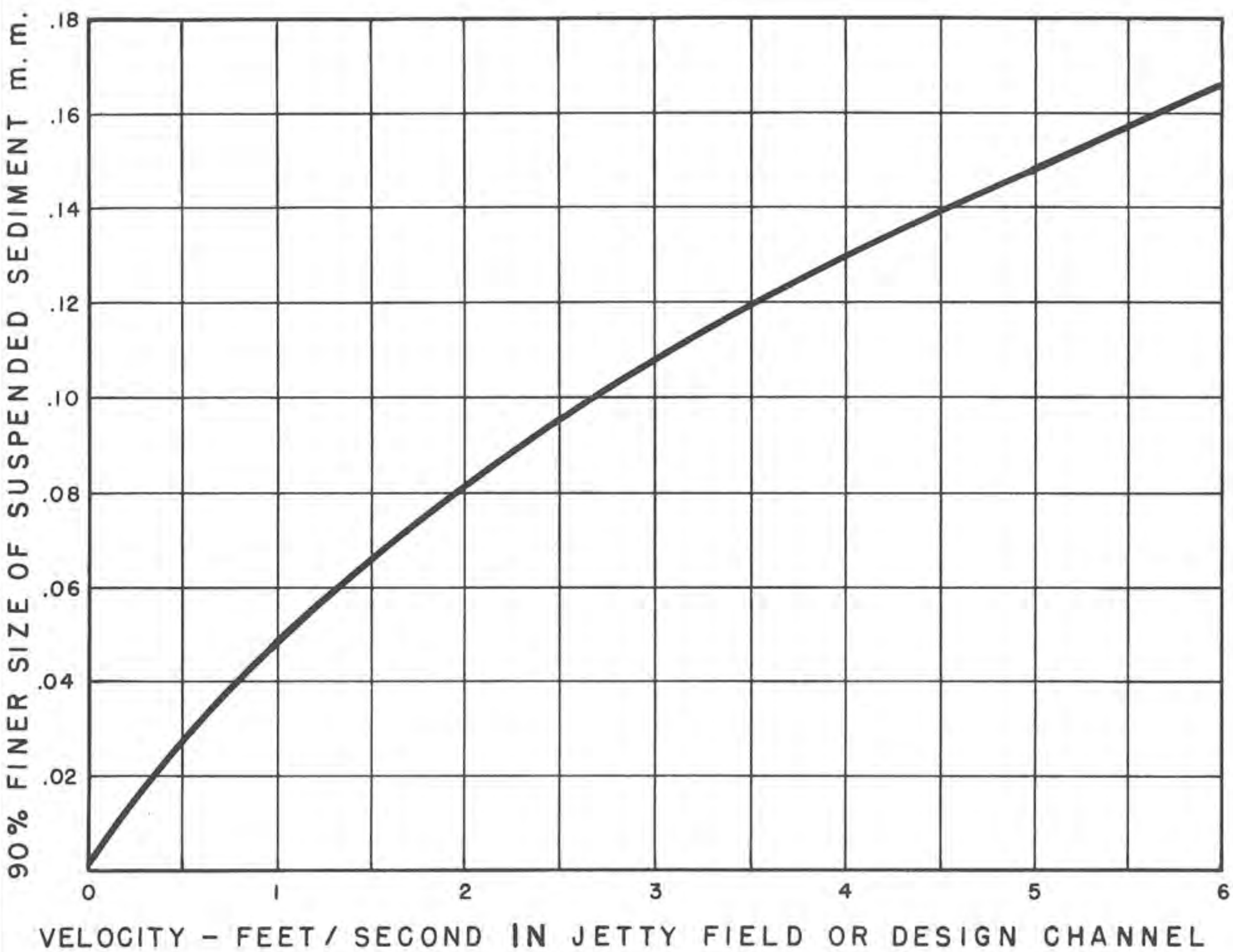


FIG. 14.—90% FINER SIZE OF SUSPENDED SEDIMENT VERSUS VELOCITY OF JETTY FIELD OR DESIGN CHANNEL

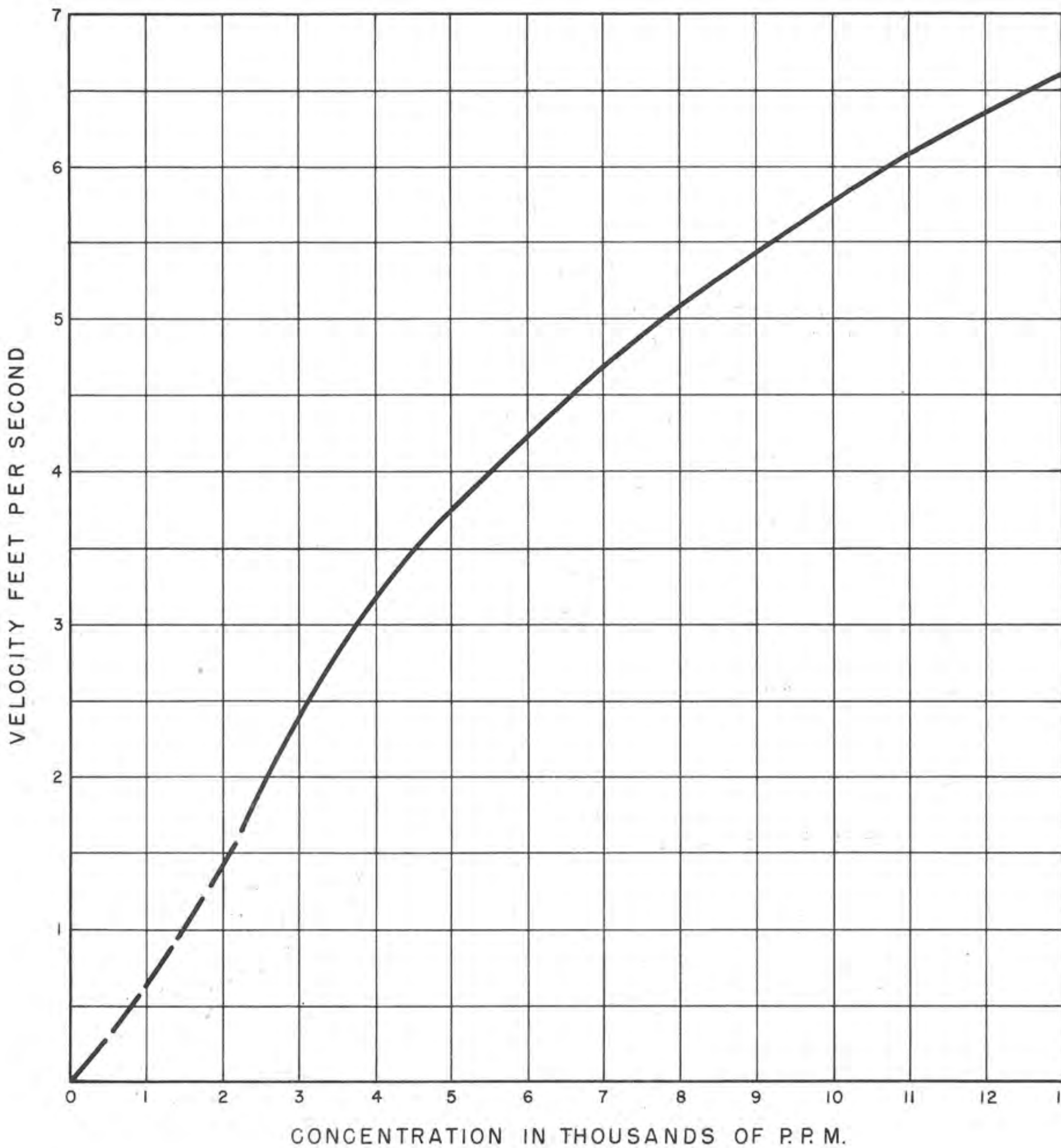


FIG. 15.— VELOCITY VERSUS CONCENTRATION
PROTOTYPE DATA

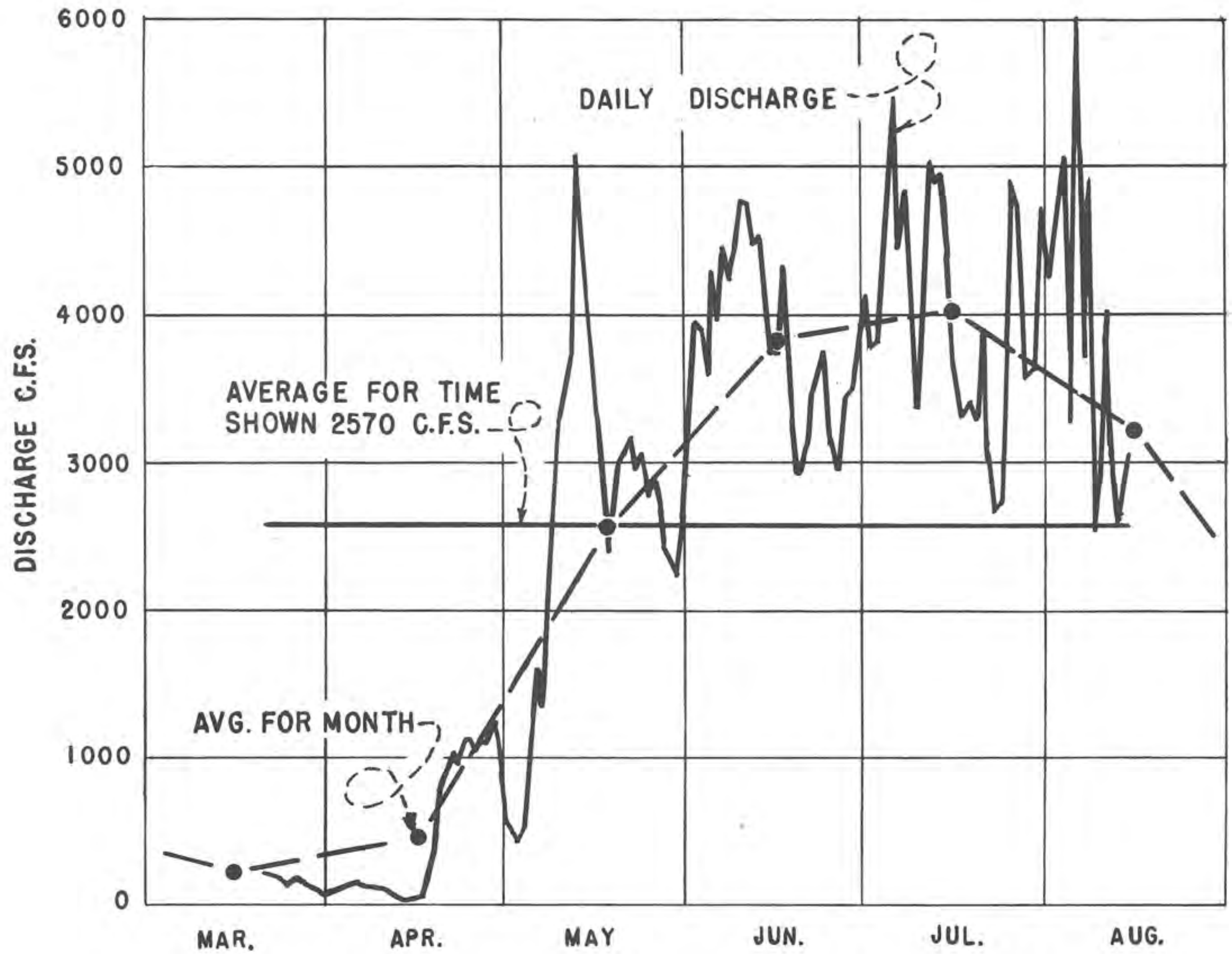
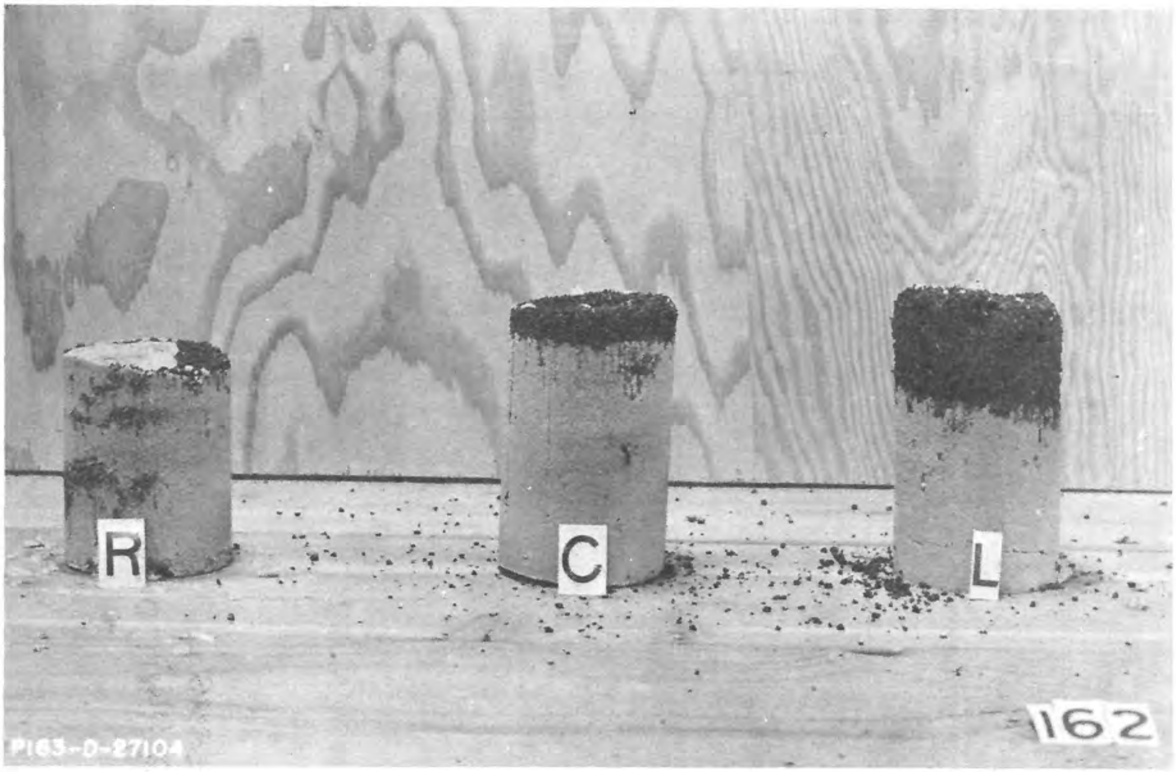


FIG. 16.— 1957 HYDROGRAPH OF RIO GRANDE NEAR BERNARDO N. M.



Outside Center Inside

of Upstream Curve

FIG. 17. --CORE SAMPLES OF BED TAKEN FROM RIGHT JETTY FIELD, DESIGN CHANNEL AND LEFT JETTY FIELD IN UPSTREAM CURVE

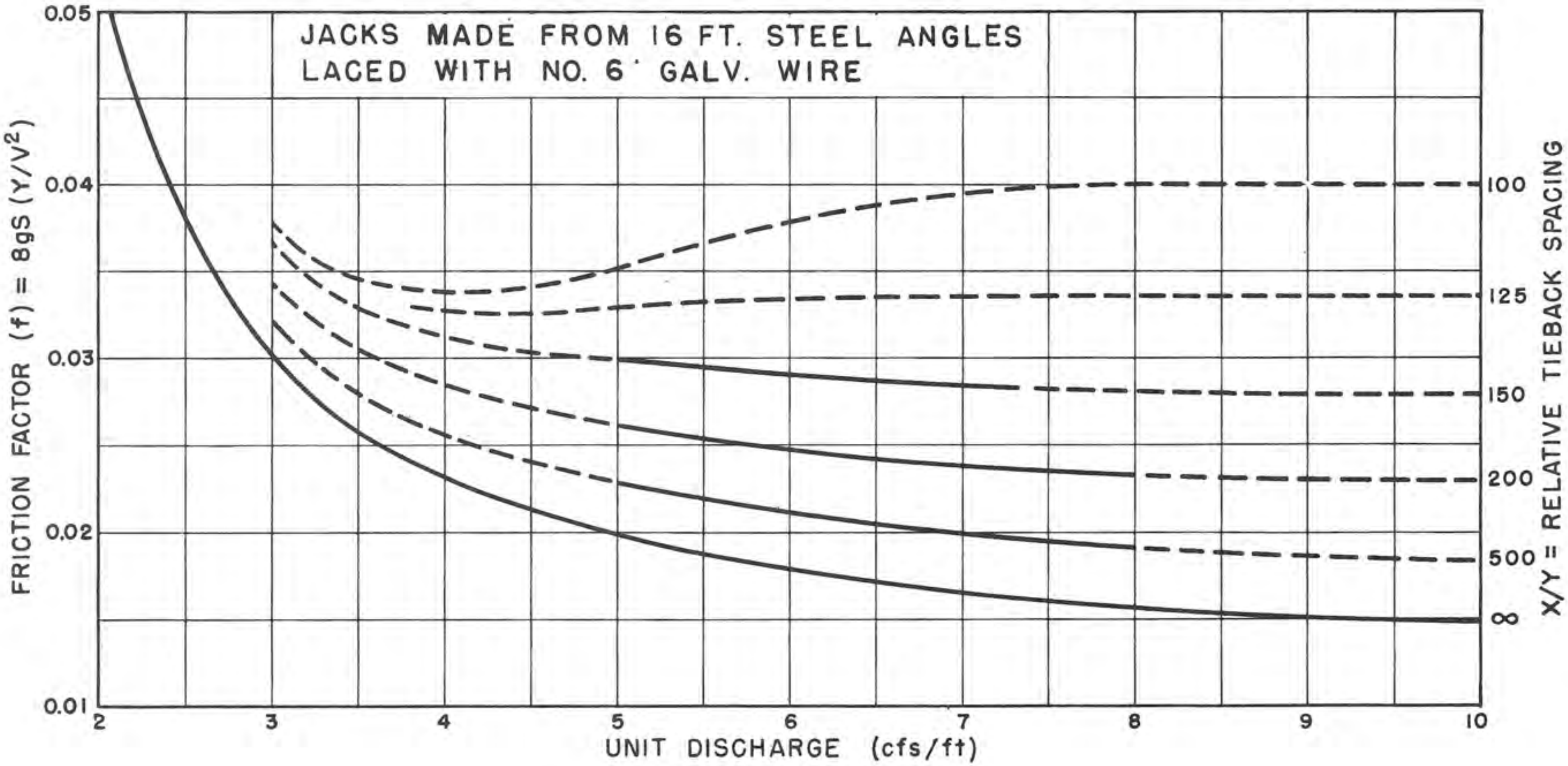


FIG. 18.— FRICTION FACTOR IN TERMS OF RELATIVE TIEBACK SPACING AND UNIT DISCHARGE

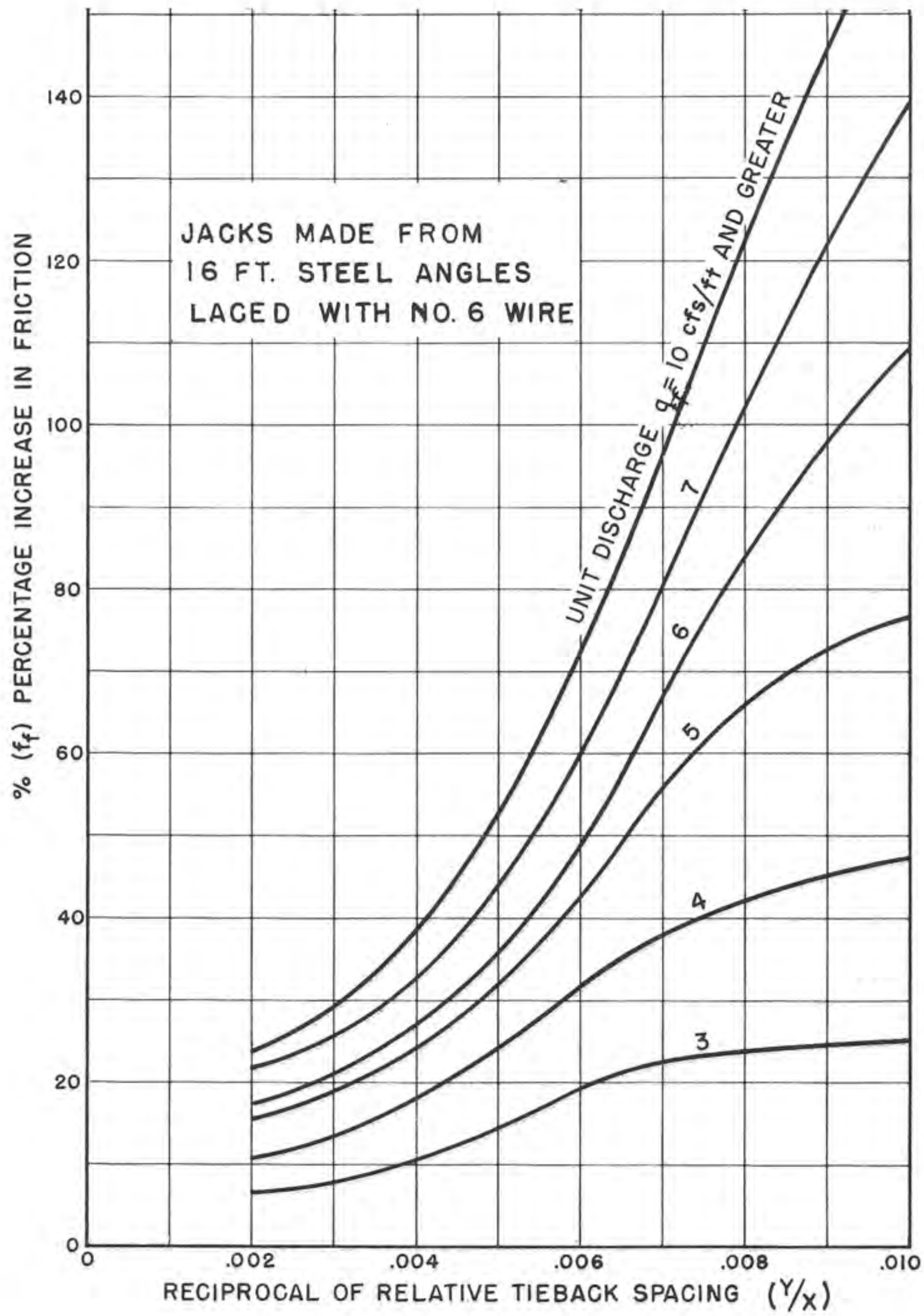


FIG. 19— PERCENTAGE INCREASE IN FRICTION FACTOR IN TERMS OF RELATIVE TIEBACK SPACING AND UNIT DISCHARGE

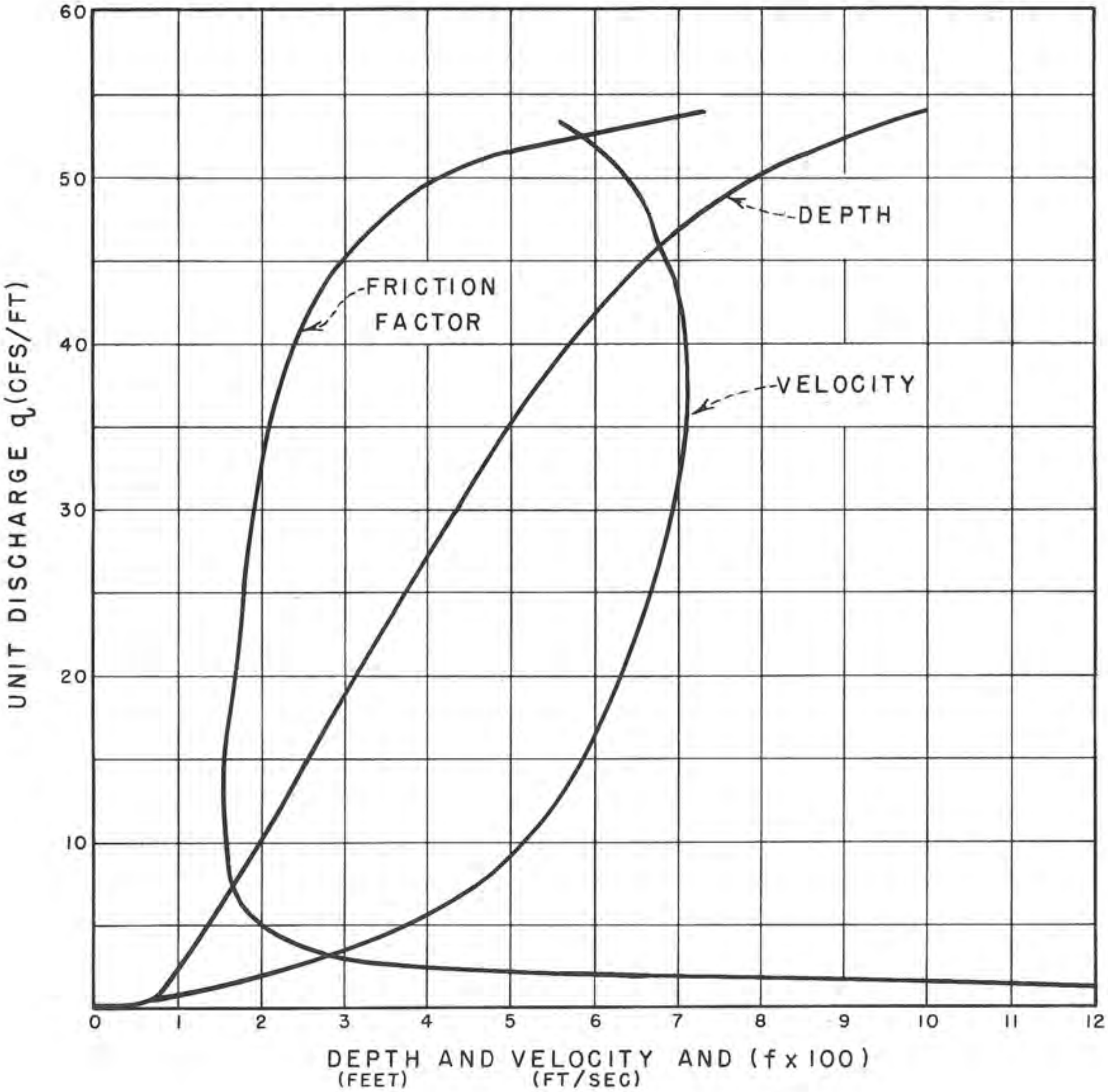


FIG. 20. — DEPTH, VELOCITY AND FRICTION FACTOR VERSUS UNIT DISCHARGE
PROTOTYPE DATA

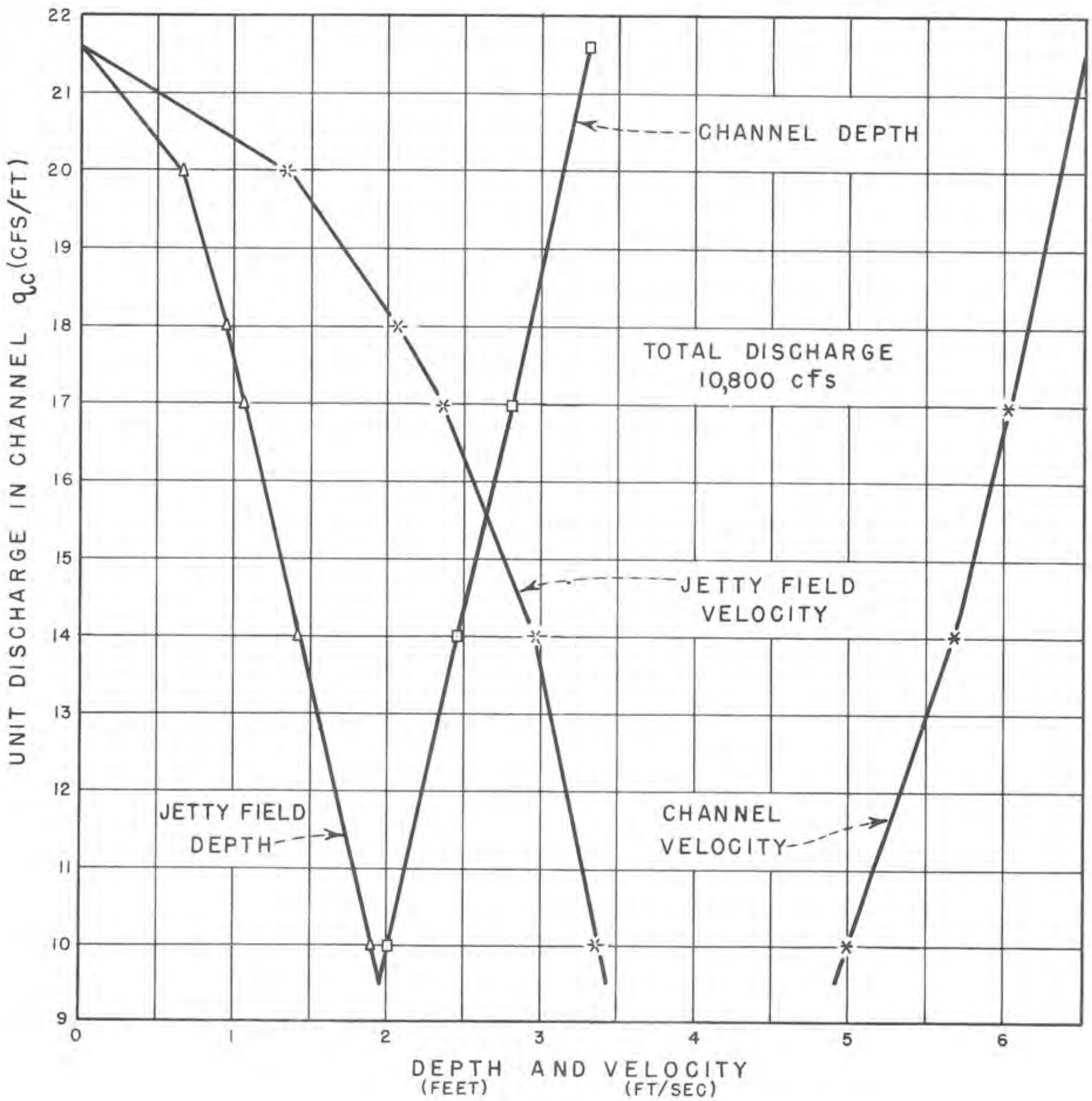


FIG. 21. - CHANNEL AND JETTY FIELD PROPERTIES DETERMINED FROM MODEL AND PROTOTYPE DATA