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Symposium 2.—Sediment in Streams

INTRODUCTION

During the 15 years that have elapsed since the 1947 Sedimentation Conference, some very significant contributions have been made to the knowledge and understanding of sedimentation in streams. Perhaps the impetus for this progress has resulted largely from the pressure placed on the planners and designers by the rapidly expanding river basin and watershed programs.

The establishment of information on alluvial channels and the development of methodology for handling sediment problems in the field has been attained from investigations, evaluations, and experience in the field and concomitant research on specific elements of sediment transport phenomenon in both laboratory and field. The papers presented in this Symposium, reflecting these advances, provide current information on (1) research on the mechanics of sediment transport in alluvial streams; (2) the relationship between channel roughness and sediment and water conveyance; (3) means of applying theories and equations on total sediment transport in aspects of stable channel design; (4) local scour and channel aggradation and degradation; and (5) engineering practice for handling sedimentation in channel rectification and control.

One of the most notable results of the laboratory investigations has been the confirmation of the observed discontinuity in the stage-discharge relations in alluvial channels. Equally significant in advancing the understanding of sediment transport phenomena has been the development of the Einstein bedload function. The interaction between streambed configuration (bed forms), the hydraulic factors, and the sediment transport has been clearly established from both laboratory and field data. Other significant advances reflected in the Symposium papers include (1) increased awareness of the need to consider the stream as a whole (hydraulic factors, discharge pattern, and total sediment load) in stable channel design; (2) development and application of radioisotope techniques in sediment transport research in the laboratory and in the field; (3) improved procedures for determining total sediment load; (4) advanced designs for handling sediment loads at bifurcations; and (5) criteria or guides for design and installation of compatible control works in alluvial channels.

The papers also indicate that a number of important aspects of sediment transport need further attention through research and evaluation. Some of the unanswered questions for additional research are (1) the prediction of the resistance factors in natural streams by application of laboratory findings; (2) accurate measurement or calculation of total sediment transport for the wide variety of situations encountered in the field; (3) techniques for transposition of data and findings from gaged to ungaged channels; (4) prediction of aggradation and degradation; and (5) extension of limited laboratory studies to encompass a greater range of hydraulic factors, grain size, and flow variability.

SEDIMENT DIVERSIONS THROUGH DISTRIBUTARY CHANNELS NORMAL TO A MAJOR RIVER

[Paper No. 26]

By GEORGE A. PRICE, JR., *engineer, U.S. Army Engineer District, New Orleans*

Synopsis

This paper discusses sediment diversions in three normally oriented distributary channels of the Lower Mississippi River.

Old River is a natural distributary, leaving the Mississippi River at river mile 302. The sediment concentration diverted from the Mississippi River is similar to the concentration in the river for the periods studied. The sediment transport capacity of Old River is somewhat greater than the sediment diverted. This diversionary channel has a history of gradual enlargement.

The Old River Control Structure diversion channel is on the right bank of the Mississippi River at mile 312. The diversion will be regulated by a control structure. Data collected during a trial operation of the control structure

indicated that the diversion channel is in approximate equilibrium. The concentration of sediment diverted was similar to, but slightly less than, the concentration in the river, and the transport capacity was also slightly less. No significant channel changes occurred during the observation period.

The Bonnet Carre Spillway is a gated flood outlet located on the left bank of the Mississippi River at mile 128, and is operated infrequently during major floods. Due to its wide and shallow configuration, the sediment transport is considerably less than the sediment diverted; consequently, sediment deposition occurs.

Introduction

Distributary channels that are oriented perpendicular to a sediment-bearing river frequently present the engineer with silting problems that are much more severe than those that are encountered in the parent stream. Such a distributary may divert a heavier sediment load in proportion to the water discharge than is carried by the river. Unless the diversionary channel is capable of transporting the heavier sediment concentrations, rapid shoaling may result.

This paper will discuss three normally oriented distributary channels of the Lower Mississippi River. In two of the examples the transport capacity of the distributary channel is adequate for the diverted sediment load; in one example the transport capacity is insufficient to avoid heavy sediment deposition.

Problems of this nature are important to the design of artificial diversions, as well as to an understanding of natural diversions of large rivers.

Pertinent Factors

The sand load transported by a stream is a function of the availability of material and the capability of the stream to transport it. Usually the supply of material in the bed is ample in comparison with the capability of the stream to transport it. Thus the bed material (sand) that is transported in suspension and as bedload is a function of the ability to transport the sizes of material found in the bed.

The so-called wash load of silt and clay material carried in suspension is almost entirely a function of supply, since the ability of the stream to transport such material is relatively great. As a general rule the wash load that is transported is equal to the wash load entering the reach.

In the case of a diversion channel, the amount of material entrained in the water that is diverted from the river is also of importance. The diverted sediment concentration may differ from the average concentration in the river if most of the flow is drawn from one part of the cross section. In the case of a bank diversion, the slow-moving water near the bank and near the bed is diverted more readily than faster flowing water. The concentration of sediment near the bed is usually higher than elsewhere in the cross section.

If the entrained load matches the transport capacity of the diversion channel, no material will be removed from or deposited on the bed of the diversion channel. However, if the transport capacity of the diversion channel is significantly different, scour or deposit will occur.

Distributary Channels Normal to the Lower Mississippi River

Old River is a major distributary of the Mississippi River, leaving the Mississippi at river mile 302 and joining the Red River 6 miles to the west to form the Atchafalaya River. Old River at present carries about 25 percent of the flow in the Mississippi River above the junction; the other 75 percent is carried by the Lower Mississippi River past Baton Rouge and New Orleans to the Gulf of Mexico. Enlargement of Old River and Atchafalaya River has been so rapid that it was feared that in one or two decades that route would become the principal outlet of the Mississippi River to the Gulf. Old River is scheduled to be permanently closed in the spring of 1963, when the Old River Diversion Channel and Control Structure are put into operation.

The newly built Old River Control Structure is on the right bank of the Mississippi River at mile 312. Its purpose is to regulate the volume of flow into the artificial diversion channel that parallels Old River and empties into Red River. The structure contains a 73-gate overbank section for floodflows and an 11-gate, low-sill section for low and medium flows. The entire structure is designed for a maximum diversion of 700,000 c.f.s. The low-sill structure was operated during several periods in 1961-62 in order to determine operational characteristics.

The Bonnet Carre Spillway is a gated flood outlet located on the left bank of the Mississippi River at mile 128. Its purpose is to protect the city of New Orleans from flooding, and it has been used three times for that purpose. Its design capacity is 250,000 c.f.s. It is about 2 miles wide by 6 miles long and empties through Lake Pontchartrain into the Gulf of Mexico.

All three of the channels described are normally oriented distributaries of the Lower Mississippi River. Pertinent characteristics such as flow, velocity, depth, sediment concentration, and sediment load have been studied and will be described. The geographic locations are shown in figure 1.

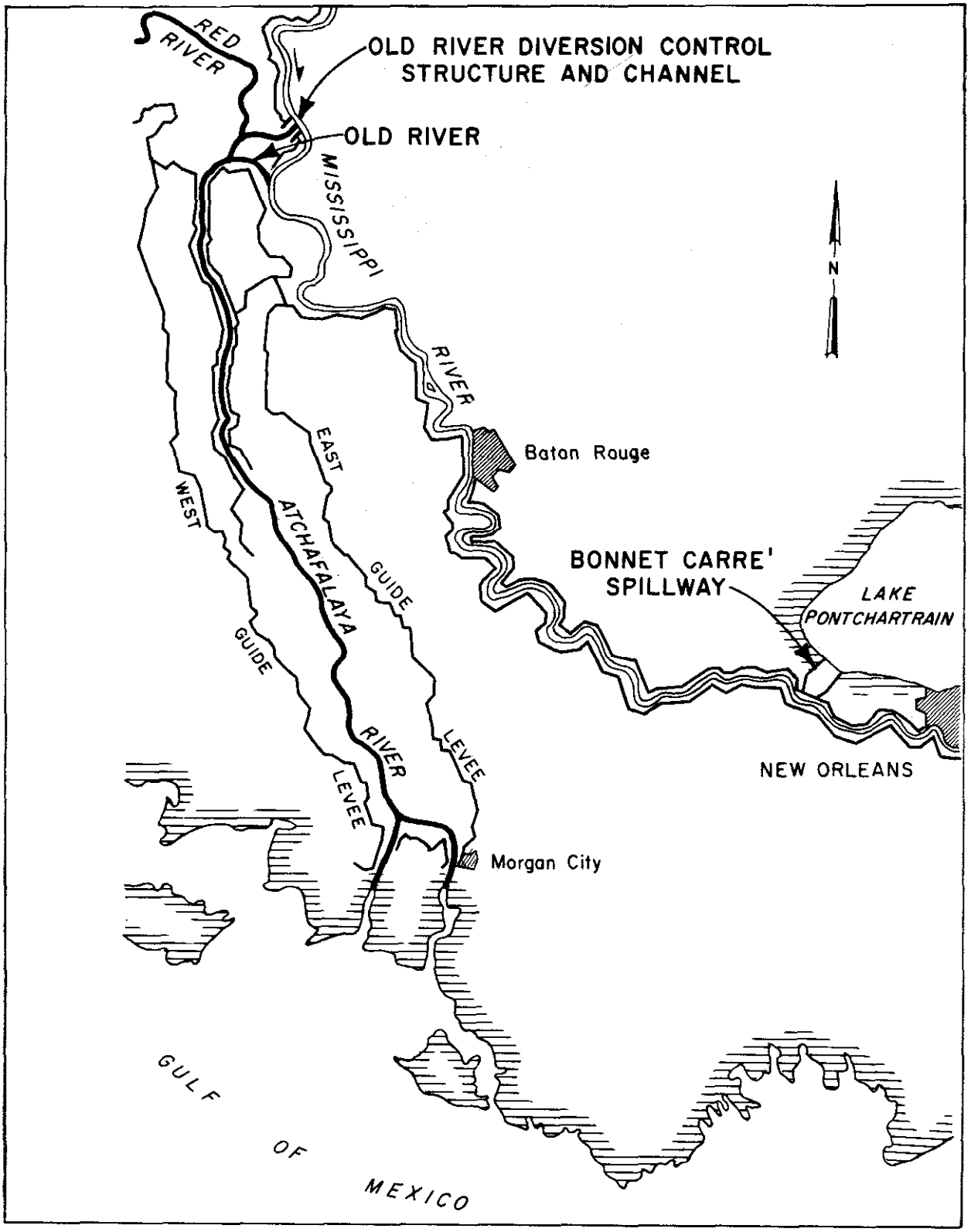


FIGURE 1. — Map showing diversionary channels of lower Mississippi River.

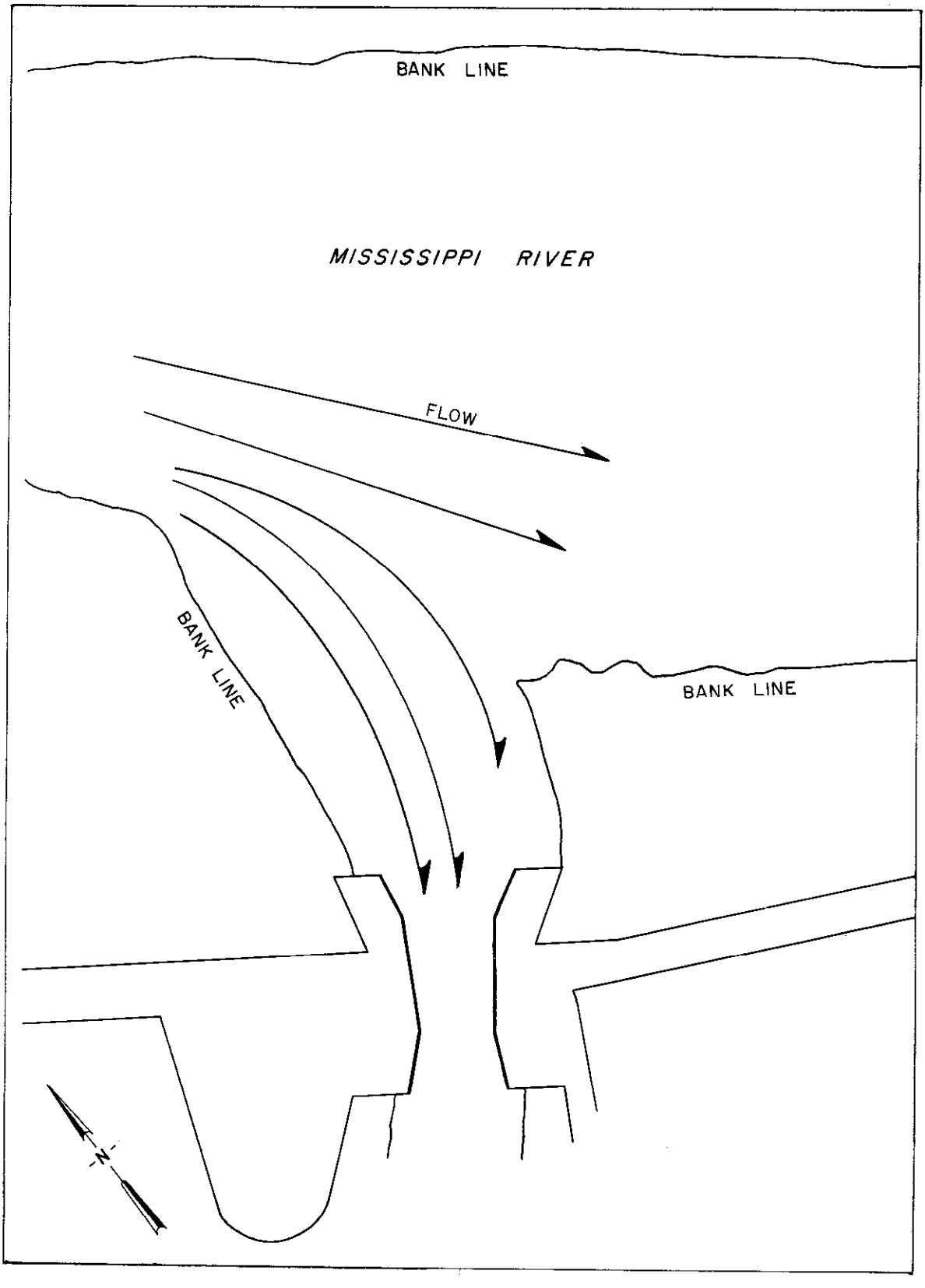


FIGURE 2. — Old River diversion channel and control structure.

Old River Control Structure Diversion Channel

The data selected for study were those obtained in June 1962 during a trial operation of the control structure, and are for flows through only the low-sill diversion structure. A plan view of the diversion is shown in figure 2. At the time of the observations, the Mississippi River was about 50 ft. deep and 3,000 ft. wide, while the diversion channel was about 25 ft. deep and 1,000 ft. wide. The pattern of surface flow into the diversion channel as obtained by float observations is shown in figure 2.

In figure 3 we see the distribution of velocity in the cross section of the Mississippi River above the diversion channel. The elevation of the bottom of the diversion channel is shown for comparison. It may be assumed that flow is diverted from the left one-third of the cross section and from depths extending below the bottom elevation of the diversion channel. The distribution of sediment concentration is also shown in figure 3. The area above the mean concentration line contains lighter-than-average sediment concentrations, whereas the area below the line contains heavier-than-average sediment concentrations. In this example the water diverted appears to come about equally from areas of lighter- and heavier-than-average sediment concentration.

Pertinent observed and computed data are given in table 1. Observed suspended sediment concentrations were obtained with standard field and laboratory procedures and show separately the material suspended as sand (coarser than 0.062 mm.), silt (finer than 0.062 mm.), and total. The volume of water diverted during the period of observation was about 12 percent of the river discharge, and the depth and the velocity in the diversion channel were each

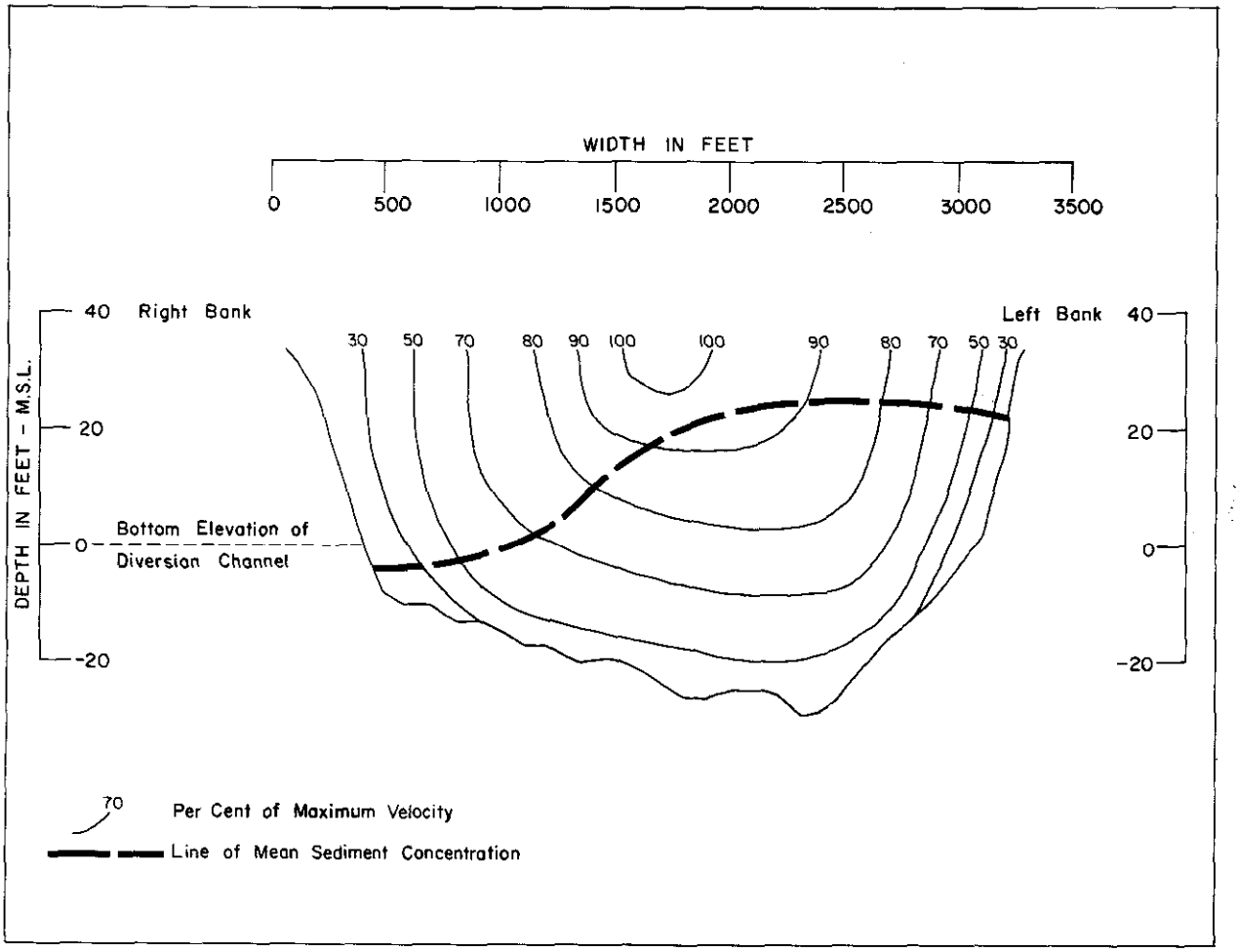


FIGURE 3. — Cross section of Mississippi River above diversion channel.

about one-half the value of that obtained in the river. The sediment loads computed by the simplified Einstein procedure as modified by Colby and Hubbell include suspended load and bedload. The computed concentrations likewise include suspended and bedload material. The computed concentrations were considered to represent the transport capability of the stream.

The data indicate that the concentration of sediment diverted from the river is consistently less, by a small amount, than the concentration in the river. The transport capacity of the diversion channel was also slightly less, due to lesser depths and velocities. Accordingly, the diversion channel appeared to be in approximate equilibrium during the period studied. Surveys confirmed that no overall changes occurred during that period, although there were some localized scour and fill.

TABLE 1.—*Sediment data, vicinity of Old River Diversion Channel, 1962*

MISSISSIPPI RIVER ABOVE DIVERSION, COOCHIE, LA.

Date	Velocity	Depth	Observed concentrations, material in suspension			Computed concentrations, suspended plus bedload [1]		
			Total	Sand	Silt	Total	Sand	Silt
	F.p.s.	*Ft.*	*P.p.m.*	*P.p.m.*	*P.p.m.*	*P.p.m.*	*P.p.m.*	*P.p.m.*
31 May	4.1	51	442	35	407	468	60	408
7 June	4.4	51	389	55	334	423	99	324
14 June	4.8	50	1,148	69	1,079	1,189	117	1,072
21 June	5.0	51	.....	.....	.....	(985)	186	(800)
29 June	4.4	51	688	39	646	730	108	622
DIVERSION CHANNEL IN ENTRANCE TO CONTROL STRUCTURE								
31 May	1.9	20	362	39	323	(385)	60	(325)
7 June	2.2	21	296	28	268	323	58	265
14 June	2.0	23	989	27	962	1,018	64	954
21 June	2.3	30	758	21	737	802	45	757
29 June	2.1	22	.....	.....	.....	(610)	35	(575)

[1] Figures in parentheses are estimated.

Old River

Two periods of data were selected for study, one in 1953 when observations were made at the junction of Old River with the Mississippi River and a period in 1962 with observations at Barbre Landing in Old River, 6 miles west of the junction. (Refer to fig. 1 for the locations.)

The depths and velocities in both streams were similar. Old River diverts about 25 percent of the discharge, and the width is correspondingly narrow. The distribution of velocity in the Mississippi River just above the diversion is shown in figure 4. Flow is diverted from about the left one-third of the cross section. The distribution of sediment concentration is also shown in figure 4. The area above the mean concentration line contains lighter-than-average sediment concentrations, whereas the area below the line contains heavier-than-average concentrations. The water diverted appears to come about equally from areas of lighter and heavier sediment concentrations.

Pertinent observed and computed data are shown in table 2. Data are presented for the two locations in Old River and for Red River Landing on the Mississippi River immediately below the diversion. The observed sediment concentrations are shown for sand (coarser than 0.062 mm.), silt (finer than 0.062 mm.), and total. The computed concentrations are shown for the same classes of material, but include bedloads.

The 1953 observations that were made at the point of diversion show some sediment concentrations higher and some lower in Old River as compared to the Mississippi River, but generally similar. The 1962 observations in Old River at Barbre Landing, 6 miles west of the point of diversion, show generally heavier concentrations in Old River than in the Mississippi River. It was concluded on the basis of the data studied that the sediment diverted at the junction is similar to the concentration in the river, but the sediment transport capacity of Old River is greater than the sediment diverted and additional material is entrained in Old River between the point of diversion and the observation point located 6 miles to the west. The enlargement of Old River has been confirmed by comparative surveys extending over a number of years.

The permanent closure of Old River is scheduled during the spring of 1963. Its function will be replaced by a navigation lock and the controlled diversion structure mentioned earlier. These works were constructed for the express purpose of controlling the flow into and hence the enlargement of the Old River-Atchafalaya River outlet.

TABLE 2.—*Sediment data, vicinity of Old River, 1953 and 1962*
MISSISSIPPI RIVER BELOW OLD RIVER, TED RIVER LANDING, LA.

Date	Velocity	Depth	Observed concentrations, material in suspension			Computed concentrations, suspended plus bedload		
			Total	Sand	Silt	Total	Sand	Silt
	F.p.s.	*Ft.*	*P.p.m.*	*P.p.m.*	*P.p.m.*	*P.p.m.*	*P.p.m.*	*P.p.m.*
23 Jan 1953.....	4.1	27	840	140	700	933	216	717
5 Mar 1953.....	4.6	31	874	385	489	1,006	487	519
28 Mar 1953.....	4.6	36	772	177	595	904	326	578
14 Aug 1962.....	4.0	17	316	16	300	370	69	301
27 Aug 1962.....	3.6	16	264	9	255	369	104	265
10 Sept 1962.....	3.7	17	169	27	142	198	55	143
24 Sept 1962.....	4.1	17	338	28	310	375	62	313
8 Oct 1962.....	4.0	18	266	22	244	311	68	243
OLD RIVER AT JUNCTION								
23 Jan 1953.....	3.4	26	992	200	792	1,098	272	826
5 Mar 1953.....	3.5	34	850	281	569	1,093	495	598
28 Mar 1953.....	3.7	38	875	233	643	988	340	648
OLD RIVER AT BARBRE LANDING, LA.								
16 Aug 1962.....	3.7	18	411	84	327	551	216	335
29 Aug 1962.....	3.2	17	334	35	299	417	107	310
12 Sept 1962.....	3.5	18	189	37	152	254	100	154
26 Sept 1962.....	3.7	22	426	110	316	593	300	293
10 Oct 1962.....	3.3	21	352	95	257	416	149	267

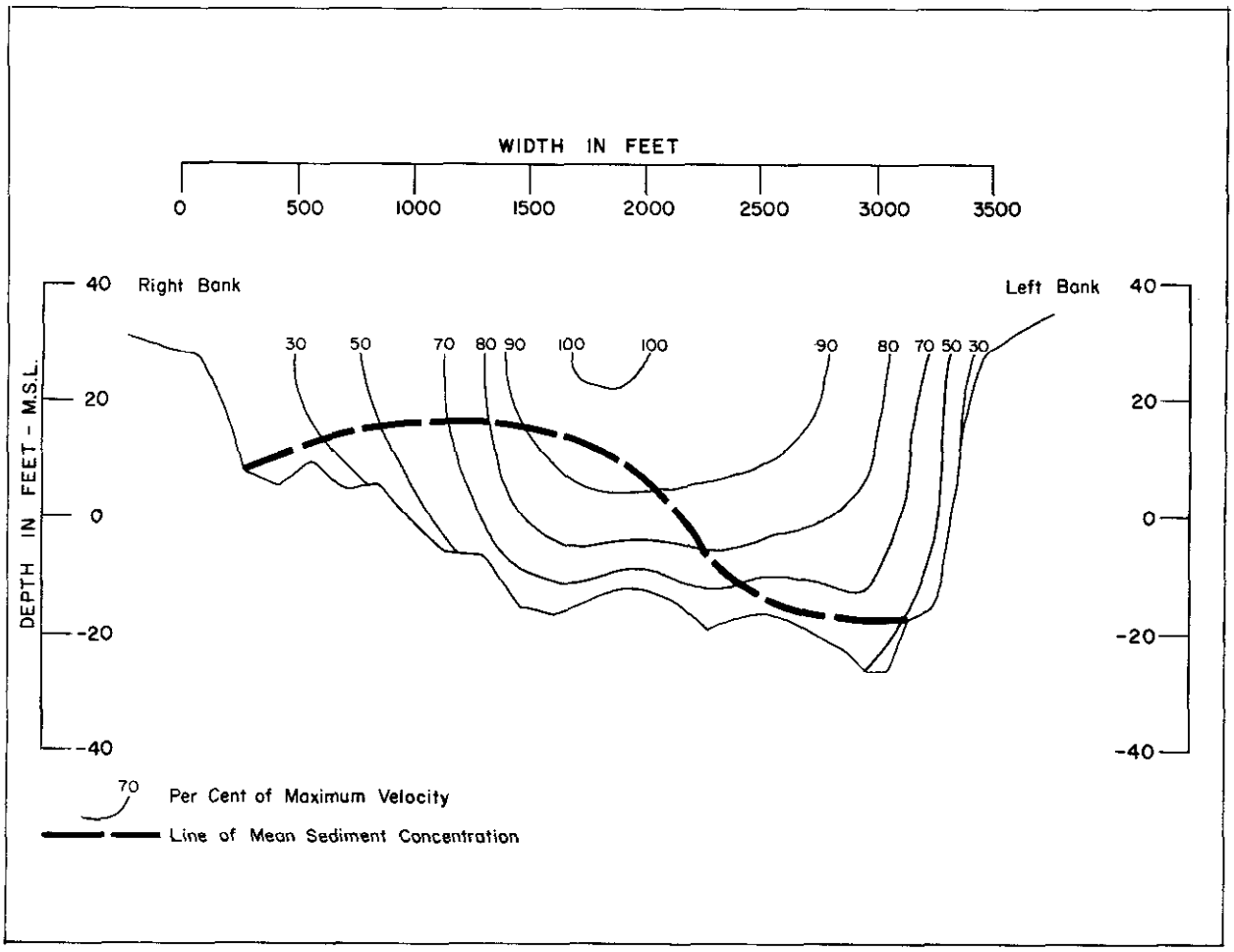


FIGURE 4. — Cross section of Mississippi River above Old River.

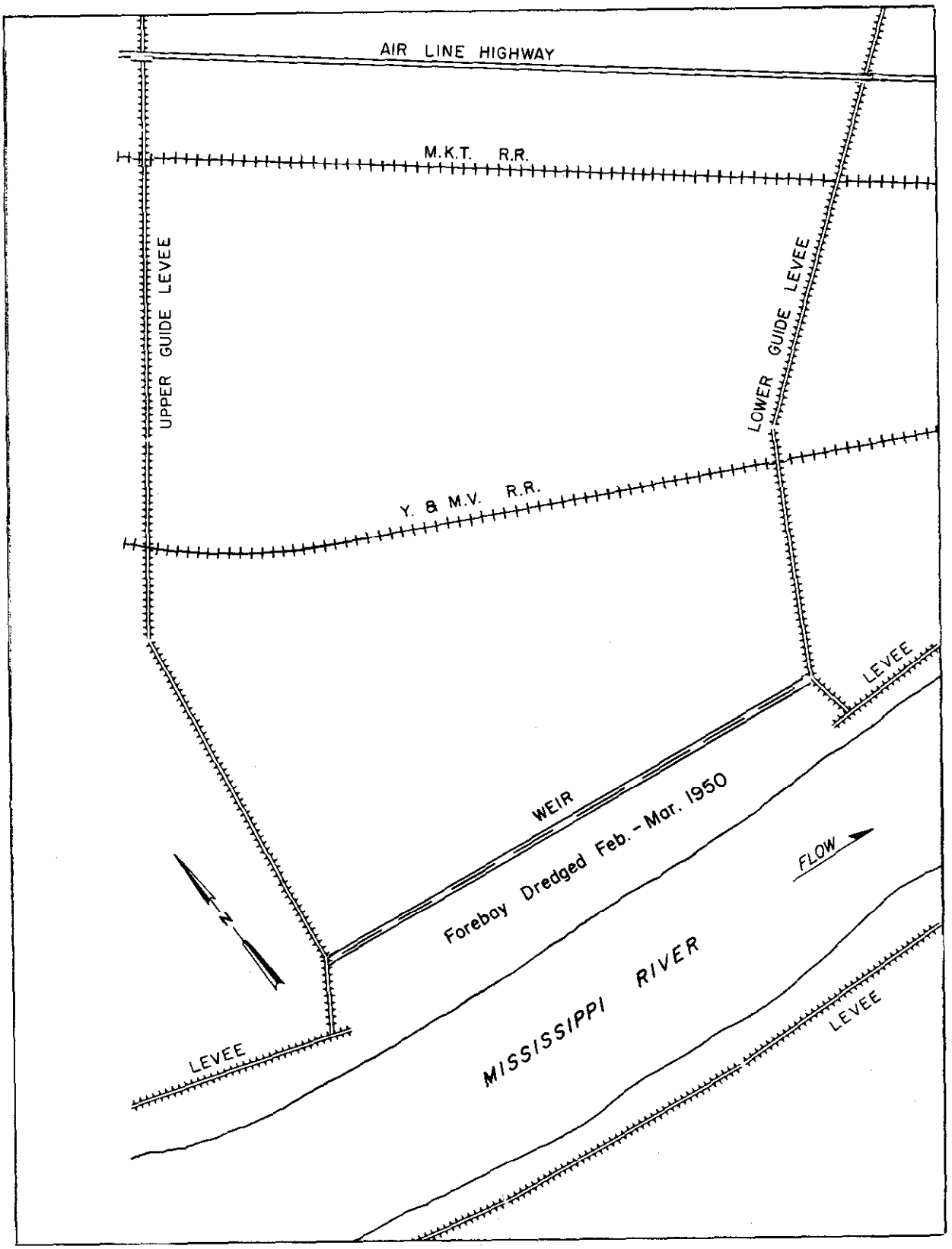


FIGURE 5. — Bonnet Carre Spillway.

Bonnet Carre Spillway

The Bonnet Carre Spillway, described earlier, was operated for a period of 38 days at the height of the Mississippi River flood of 1950. The floodway diversion lowered the flood crest at New Orleans by 2 ft. to assure safe passage of the flood. Over 10 million acre-feet of water was diverted through the floodway. A sketch of Bonnet Carre is shown in figure 5.

The floodway diversion channel was about 2 miles wide and the depth of flow about 10 ft. deep. The Mississippi River just above the diversion was about 2,500 ft. wide and 60 ft. deep. The average velocity in the floodway was 1 to 1.5 f.p.s., and it was about 6 f.p.s. in the river.

The concentration of suspended sediment in the river as measured at Baton Rouge, La., about 100 miles upstream, was 664 p.p.m. average for the 38-day period. Measurements were not made in the Mississippi River directly above the diversion; however, the concentration at the weir at the floodway entrance was 1,068 p.p.m. averaged over the period.

Thus, it appeared that the water diverted carried a heavier suspension of sediment than did the river. However, this conclusion must be qualified by the fact that turbulence at the weir may have carried some bedload into suspension and some material may have been picked up in the forebay. A suction dredge was operating in the forebay, but was discharging its spoil into the river downstream.

Approximately two-thirds of the diverted sediment load was lost in the 6-mile length of the floodway. The concentration at the terminal end was only 400 p.p.m. A total of 5 million cubic yards of deposit remained in the floodway after the operation.

Conclusions

While the data studied are not as complete as could be desired, they indicate:

a. Of the three diversion channels studied, one had a sediment concentration similar to the concentration in the river, one had a somewhat greater concentration, and one a slightly less.

b. In one example the diversion channel was in approximate equilibrium with the diverted sediment load, while one had a greater and one a lesser sediment transport capacity than the diverted load.

c. The distribution of both sediment and velocity in the channel cross section is not uniform, and the pattern of distribution may vary from point to point along the river.

Accordingly, a diversion channel may be in equilibrium with the sediment load carried by the river but may not be in equilibrium with the sediment load diverted from the river. A very thorough study of all pertinent factors is required in the design of a diversion channel that will be in equilibrium, or in determining what changes will occur in a channel that is not in equilibrium. Observations of the distribution of velocity and the distribution of sediment in the river at each location to be considered should be part of the basic study for selecting a suitable diversion site. Such studies will aid materially in obtaining a diverted sediment concentration lighter or heavier than the average in the river, according to the requirements imposed by other factors.

A STUDY OF VARIABLES AFFECTING FLOW CHARACTERISTICS AND SEDIMENT TRANSPORT IN ALLUVIAL CHANNELS

[Paper No. 27]

By D. B. SIMONS[1] and E. V. RICHARDSON, *hydraulic engineers, U.S. Geological Survey, Fort Collins*

Introduction

The U.S. Geological Survey has been studying sediment transport and resistance to flow at Colorado State University, Fort Collins, since 1956. The study has largely been done in laboratory flumes with only limited collection of field data.

The characteristics of the flumes used in the investigations are given in table 1. The principal studies have been completed in the 2-ft. and 8-ft. flumes.

The characteristics (size and gradation) of the bed materials that have been studied, the range of mean velocity, depth, slope, temperature, bed material and fine sediment concentration, and forms of bed roughness investigated and the flume in which each study was made are indicated in table 2.

The investigations conducted in flume 1

TABLE 1.—*Characteristics of recirculating adjustable slope flumes used at Fort Collins, Colo.*

Flume No.	Length	Width	Depth	Range of slope	Range of discharge
	Feet	*Feet*	*Feet*	*Percent*	*Cubic feet per second*
1.....	150	8	2	0–1.5	0–22
2.....	60	2	2.5	0–2	0–7.5
3.....	40	⅔	⅔	0–15	0–1.0
4[1].....	200	8	4	0–3	0–100

[1] Under construction.

[1] Now chief, Civil Engineering Research Section, Colorado State University, Fort Collins.

TABLE 2.—*Flume studies at Fort Collins, Colo.*

Flume No.	Runs	Bed material d_{50}	Gradation coefficient σ	Velocity range	Depth range	Slope-10² range	Bed material concentration range	Fine sediment concentration range	Temperature range	Range of bed forms[1]
	Number	*Mm.*		*F.p.s.*	*Ft.*	*Ft./ft.*	*P.p.m.*	*P.p.m.*	*°C.*	
1	41	0.19	1.30	0.74–4.62	0.43–1.09	0.005–0.95	0–47,300	.....	16–19	B–A
1	21	.27	1.54	.73–4.60	.45–1.13	.005–1.022	0–35,800	.....	11–18	B–A
1	38	.28	1.68	.53–4.93	.40–1.07	.005–1.007	0–42,400	.....	10–17	B–A
1	46	.45	1.60	.65–6.18	.25–1.00	.015–1.01	0–15,100	.....	9–20	B–A
1	54	.47	1.54	1.13–5.32	.30–1.33	.084–.96	1.6–17,700	0–42,000	11–22	R–A
1	44	.93	1.54	1.00–5.86	.43–1.11	.0129–1.28	0–10,200	.....	16–21	B–SW
2	30	.32	1.57	.86–5.73	.51–.63	.054–.983	0–29,600	.....	8–33	B–A
2	39	.54	1.52	.89–6.21	.59–.91	.016–1.928	0–50,000	0–102,000	16–23	B–A
2	15	.33	1.27	1.02–5.93	.49–.52	.016–1.14	0–18,400	.....	19.8–20.3	B–A
2	17	.33	2.07	1.06–6.34	.48–.52	.022–.98	0–22,500	.....	18.3–23.4	B–A

[1] B = beginning of motion; R = ripples; SW = standing waves; A = antidunes.

with the 0.19-, 0.27-, 0.28-, 0.45-, and 0.93-mm. bed materials were conceived principally to provide a better understanding of the gross mechanics of flow and sediment transport when a range of sizes of natural alluvial sands is used with a gradation coefficient σ, ranging from 1.5 to 1.6 where

$$\sigma = \frac{1}{2}\left(\frac{d_{50}}{d_{16}} + \frac{d_{84}}{d_{50}}\right)$$

Special studies have been completed to help understand:

(1) The effect of temperature (viscosity).
(2) The effect of large concentrations of fine sediment (bentonitic and kaolin clays).
(3) The effect of the gradation of bed material.
(4) The effect of specific weight of bed material.

For each of the runs for the regular as well as the special studies, the basic data collected included velocity, depth, slope, the size characteristics of the bed material, water temperature, bed roughness (obtained by direct measurement and by the sonic sounder measurements (*13*)), and bed material discharge.

Forms of Bed Roughness

Based upon the general studies, Simons and Richardson (*16, 18*) suggested the following regimes of flow and forms of bed roughness (fig. 1):

1. Lower Regime
   - Ripples
   - Ripples on dunes
   - Dunes
2. Transition
3. Upper Regime
   - Plane bed
   - Standing waves
   - Antidunes

Resistance to flow is relatively large and bed material discharge is small in the lower regime, and, conversely, resistance to flow is small and bed material discharge large in the upper regime.

The principal reasons for differentiating between ripples and dunes are:

1. Ripples are relatively small, have lengths from 0.25-1.50 ft. and a maximum amplitude of about 0.1 ft., move downstream more or less in the plane of the bed as a result of bedload movement, do not increase in amplitude with increasing depth, have a rather uniform length and amplitude, and do not form under the test conditions when the d_{50} of the bed material is larger than about 0.7 mm.

2. Dunes are similar in appearance to ripples but are much more irregular in form and of considerably greater length and amplitude than ripples. Also, dunes move through a channel at very different elevations with time and are less uniform than ripples. In general, the dimensions of the dunes are quite intimately related to the characteristics of the bed material, particularly fall velocity, but also to slope and depth. Dunes can attain a maximum amplitude approximately equal to the mean depth of flow under ideal conditions. Dunes can occur with any size of bed material so long as the flow brings sufficient tractive forces to bear on the particles to move them at $F_r < 1$. Dunes formed of material with a median diameter of about 2 in., with lengths over 100 ft. and amplitudes of about 3 to 4 ft. have been observed on the lower Rio Grande.

The relation between length of dunes and median fall diameter of bed material for nearly constant depth (fig. 2) shows that the length of dunes increases as size of bed material decreases. Also, the data show that with the smaller sizes of bed material the dunes are less angular.

The resistance to flow caused by the ripples exceeds that for dunes when d_{50} is smaller than about 0.30 mm. under flume conditions. The converse is true for $d_{50} > 0.30$. For field con-

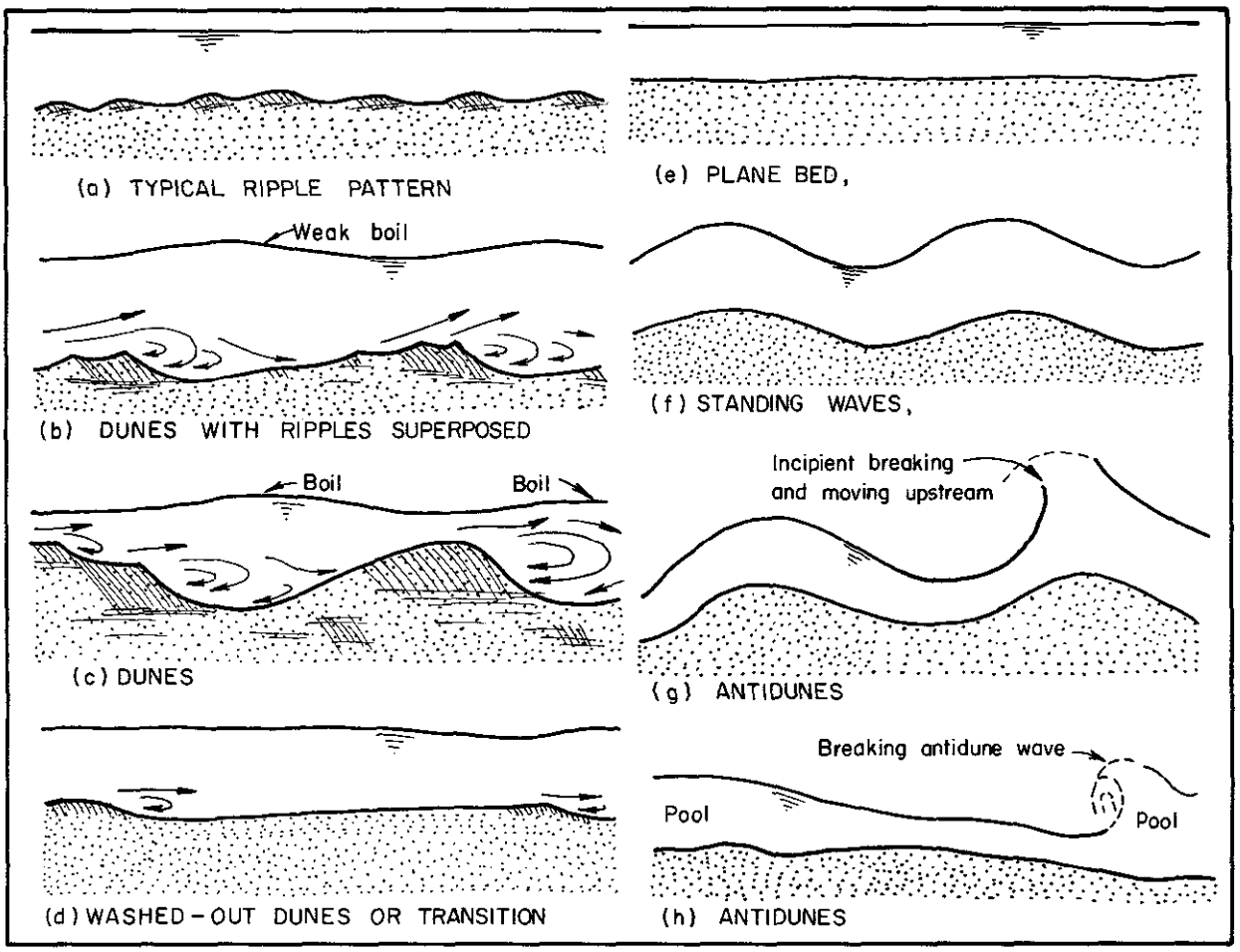


FIGURE 1. — Forms of bed roughness in alluvial channels.

ditions (depths greater than about 2 ft.) the resistance to flow with dunes probably always exceeds that of ripples because of the small relative roughness associated with large depths and ripples. Consequently, resistance to flow caused by ripples and dunes is also a function of their lengths (*21*).

Variables Affecting Bed Roughness

Principal variables that affect the fall velocity and the fall diameter of a given bed material in natural flow, and hence the form of bed roughness, are temperature (viscosity), as reported by Al-Shaikh Ali,[2] and concentration of fine sediment (apparent viscosity), as reported by Haushild, Simons, and Richardson (*10*). Figure 3 indicates the effect of large concentrations of fine sediment (bentonite and kaolin) and temperature on the fall velocity of a river sand with a standard fall diameter of 0.28 mm.

A complete change in form roughness can be accomplished by the introduction of a large concentration of fine sediment (an increase in apparent viscosity of the water-sediment mixture) and, under favorable conditions, by a large reduction in stream temperature (an increase in viscosity of the water). The fact that radical changes in bed form can be caused by large changes in stream temperature or large concentrations of fine sediment, or both, can easily be verified in the laboratory flumes. The same effect has been observed in the field. For example, the Loup River near Dunning, Nebr., has dunes as bed roughness in the summer when the stream fluid is less viscous; whereas, in contrast, the bed is essentially plane during the cold winter months. Similarly, two sets of data collected by Fahnestock[3] in a stable reach of

[2] AL-SHAIKH ALI, KHALID S. INFLUENCE OF TEMPERATURE ON SEDIMENT TRANSPORT AND ROUGHNESS IN ALLUVIAL CHANNELS. Master's thesis. On file, Colorado State University library, Fort Collins. 1961.

[3] FAHNESTOCK, R. K. DATA COLLECTED ON THE RIO GRANDE, 1962. U.S. Geol. Survey, Gen. Hydrol. Written communication.

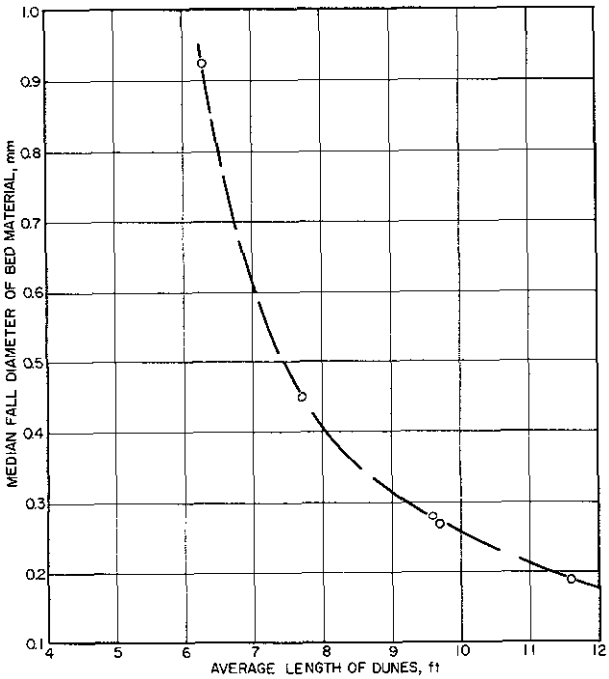


FIGURE 2. — Relation of length of dunes to median fall diameter of bed material.

the Rio Grande at similar discharges show that when the water was cold the bed of the stream was plane, the resistance to flow was small, the depth was relatively small, and the velocity was large; whereas, when the water was warm the bed roughness was dunes, the resistance to flow was large, depth was larger, and the velocity was smaller (table 3).

TABLE 3.—*Rio Grande River data collected on a stable reach for two different stream temperatures at similar discharge*

Temperature	Velocity	Depth	Slope	Bed material	Bed roughness	Manning
F.°	*Ft./sec.*	*Ft.*		d_{50}		*n*
50.....	4.25	2.45	0.00049	0.30	Plane..	.0.014
80.....	2.53	3.66	.00053	.30	Dunes..	.034

Changes in the form of bed roughness and, hence, the resistance to flow are reflected in depth and stage-discharge curves. Figure 4 (*19*) shows a typical break in a depth-discharge relation caused by a change in bed form from dunes to plane bed. The Manning *n* for the dune bed was approximately 0.03; whereas, with the plane bed condition the Manning *n* was 0.012.

[4] DARANANDANA, NIWAT. A PRELIMINARY STUDY OF THE EFFECT OF GRADATION OF BED MATERIAL ON FLOW PHENOMENA IN ALLUVIAL CHANNELS. Doctor of Philosophy dissertation. On file, library of Colorado State University, Fort Collins. 1962.

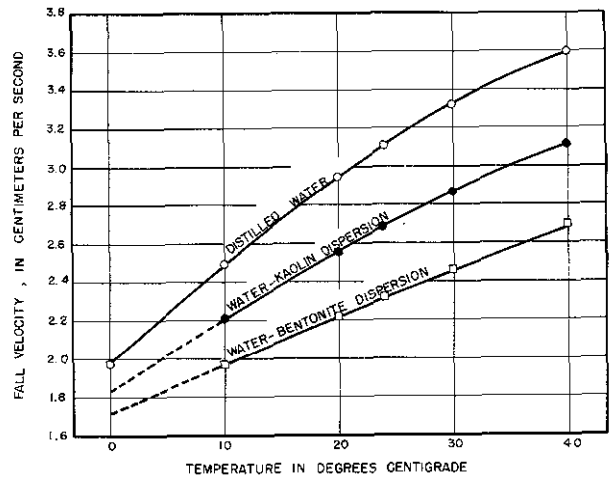


FIGURE 3. — Variation in fall velocity of median particle size of Elkhorn River, Nebr., sand with changes in temperature in distilled water and with 5 percent by weight dispersions of kaolin and of bentonite.

As a further verification of the significance of fall velocity, experiments were conducted in which an expanded clay (idealite) with a specific gravity of about 1.7, a median sieve diameter of 0.7 mm., and a fall diameter of 0.35 mm. was used as a bed material. For a given flow condition this lightweight bed material was shaped into essentially the same bed form as was observed with the use of the 0.33 mm. sand as the bed material.

To determine, in a preliminary way, the role of gradation of bed material with respect to resistance to flow and sediment transport in alluvial channels, two sets of runs were completed by Daranandana, 1962,[4] in flume 2, using two different bed materials and holding depth and temperature constant. The two bed materials had the same median fall diameter, but one was

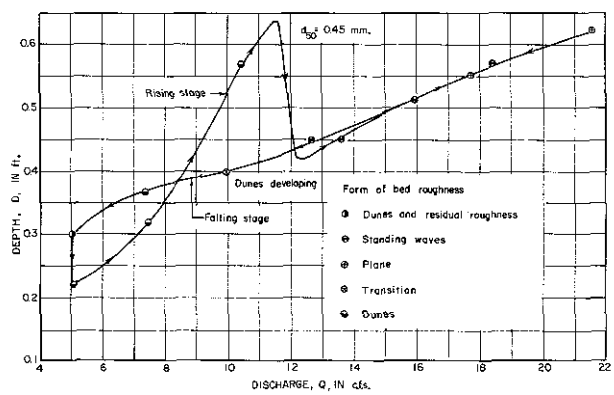


FIGURE 4. — Typical depth-discharge relation involving lower and upper regime flow.

quite uniform ($\sigma = 1.27$) and the other was graded ($\sigma = 2.07$). The effects of gradation on form of bed roughness are qualitatively illustrated in figure 5.

To illustrate further the effect of gradation of bed material on resistance to flow in alluvial channels, figures 6 and 7 (after Daranandana (1962)) relate f_b to V and f_b to S, respectively, for the runs made with the uniform and the graded bed material, $d_{50} = 0.33$ mm. These relations follow logically from figure 5. The large scatter in these relations, when the bed form is ripples and dunes and the bed material is graded, indicates that continual sorting and remixing takes place with time. This suggests that the representative fall velocity and gradation of the bed material also varies with time, which adds to the complexity of the mechanics of flow and transport in alluvial channels.

Prediction of Form of Bed Roughness

A completely satisfactory method of predicting form of bed roughness has apparently never been developed. Various methods of predicting form roughness [Albertson, Simons and Richardson (*1*); Simons and Richardson (*16*); and Garde [5]] have been proposed; however, none of the methods are applicable for both laboratory and field conditions. A relatively simple relation (fig. 8) that relates stream power, median fall velocity of bed material, and form roughness gives an indication of the form of bed roughness one can anticipate if the depth, slope, velocity, and fall diameter of bed material are given. Flume data were utilized to establish the boundaries separating (1) plane bed and ripples, (2) ripples and dunes for all sizes of bed material, and (3) dunes and transition for the 0.93-mm. bed material. The lines dividing (1) dunes and transition and (2) transition and upper regime are based upon the following field data: (1) Elkhorn River (Beckman and Furness (*2*)); (2) Rio Grande 20 miles above El Paso (see table 3); (3) Middle Loup River at Dunning (Hubbell and Matejka (*11*)); (4) Rio Grande at Cochiti; (5) Rio Grande near Bernalillo; (6) Rio Grande—Angostura heading (Culbertson and Dawdy (*7*)); (7) the canal

[5] GARDE, R. J. TOTAL SEDIMENT TRANSPORT IN ALLUVIAL CHANNELS. Doctor of Philosophy dissertation. On file, library, Colorado State University, Fort Collins. 1959.

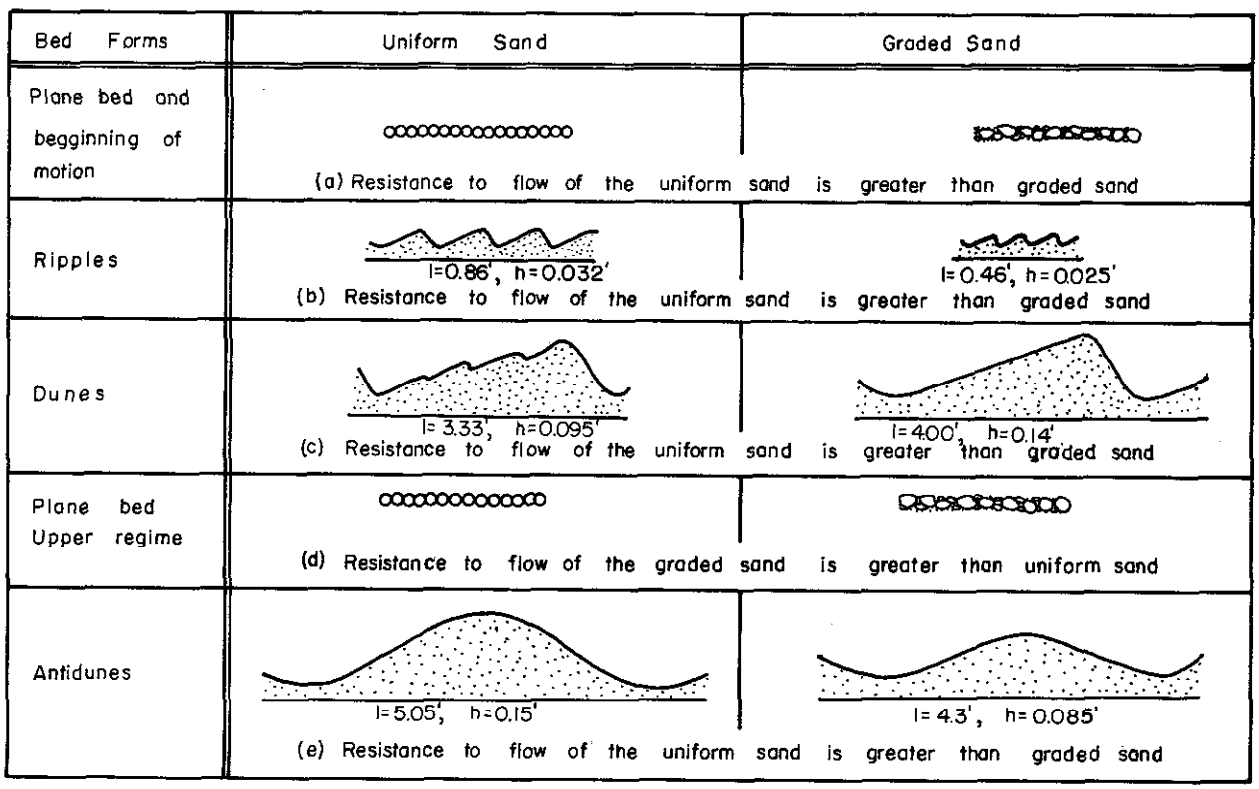


FIGURE 5. — Comparison of resistance to flow for various bed forms for both sands.

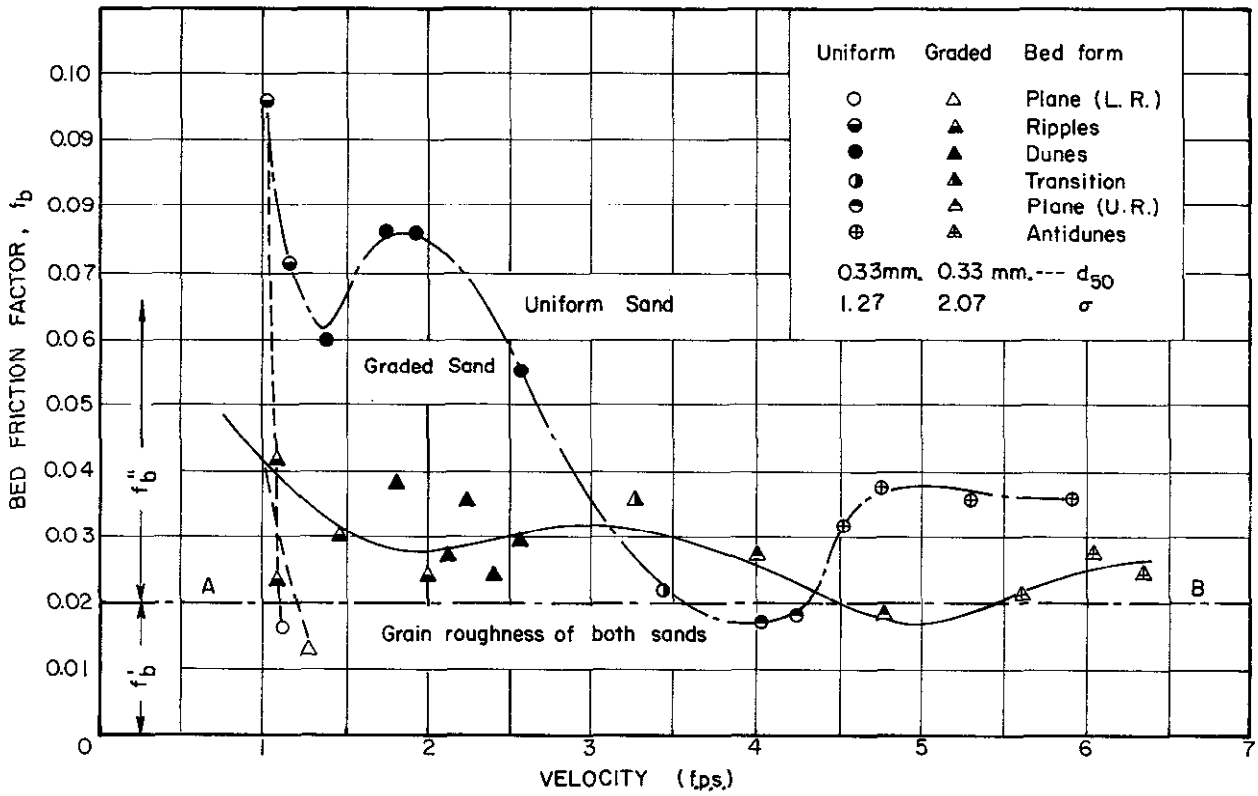


FIGURE 6. — Comparison of total bed friction factor, friction factor pertaining to grains, and friction factor pertaining to form roughness for uniform and graded bed material sands.

data reported by Simons;[6] and (8) canal data collected in Pakistan by Tipton and Kalmbach,[7] Denver, and Harza Engineering Co., Chicago, during 1961-62. If only the flume data were used, the dividing line between dunes and transition would occur at about 10 percent less stream power than the field data indicates.

In figure 8, note that the range of stream power for which dunes occur becomes smaller with decreasing fall diameter of bed material. Hence, when the median fall diameter of the bed material is relatively small, stable channels must be designed with care. For example, if large resistance to flow is anticipated, stable channel design concepts indicate a relatively small average velocity, large depth, and steep slope. However, the channel may not function as anticipated. If the stream power is sufficiently large, the bed form will not be dunes but will be plane, which will result in a small resistance to flow, a high average velocity, a small depth, and an unstable channel with high transport capacity. A good example is the Marala-Ravi Canal in Pakistan.[7] This canal has bed material with a median diameter of 0.29 mm. and was designed to carry 15,500 c.f.s. When it was put into operation its bed depth was 9.1 ft., its slope was 1/3,460, with an average velocity of 4.51 ft./sec. The stream power for these conditions was 0.71, which was sufficient for a plane bed, channel instability, and a bed material discharge of 132 tons per day per foot of width.

The measurement, as well as the prediction of average velocity in alluvial channels, is a complex problem. When the velocity data are reviewed it is immediately noted that error can be introduced into the measurement by any change in bed configuration during the measurement. Also, as the water-sediment mixture flows over the crests of the ripples and dunes, a separation occurs (see fig. 1). Taking measurements of velocities within the zone of separation shows that upstream velocities exist that are one-half to two-thirds average stream velocity and the boundary shear is sufficient to form ripples that are oriented opposite to the direction of the flow in the channel.

Prediction of Average Velocity

Various methods of determining average velocity and resistance factors for alluvial channels have been suggested. The Manning and

[6] SIMONS, D. B. THEORY AND DESIGN OF STABLE CHANNELS IN ALLUVIAL MATERIALS. Doctor of Philosophy dissertation. On file, library, Colorado State University, Fort Collins. 1957.

[7] Data collected by Tipton and Kalmbach, Inc., Engineers, Denver, Colo., written communication. 1961-62.

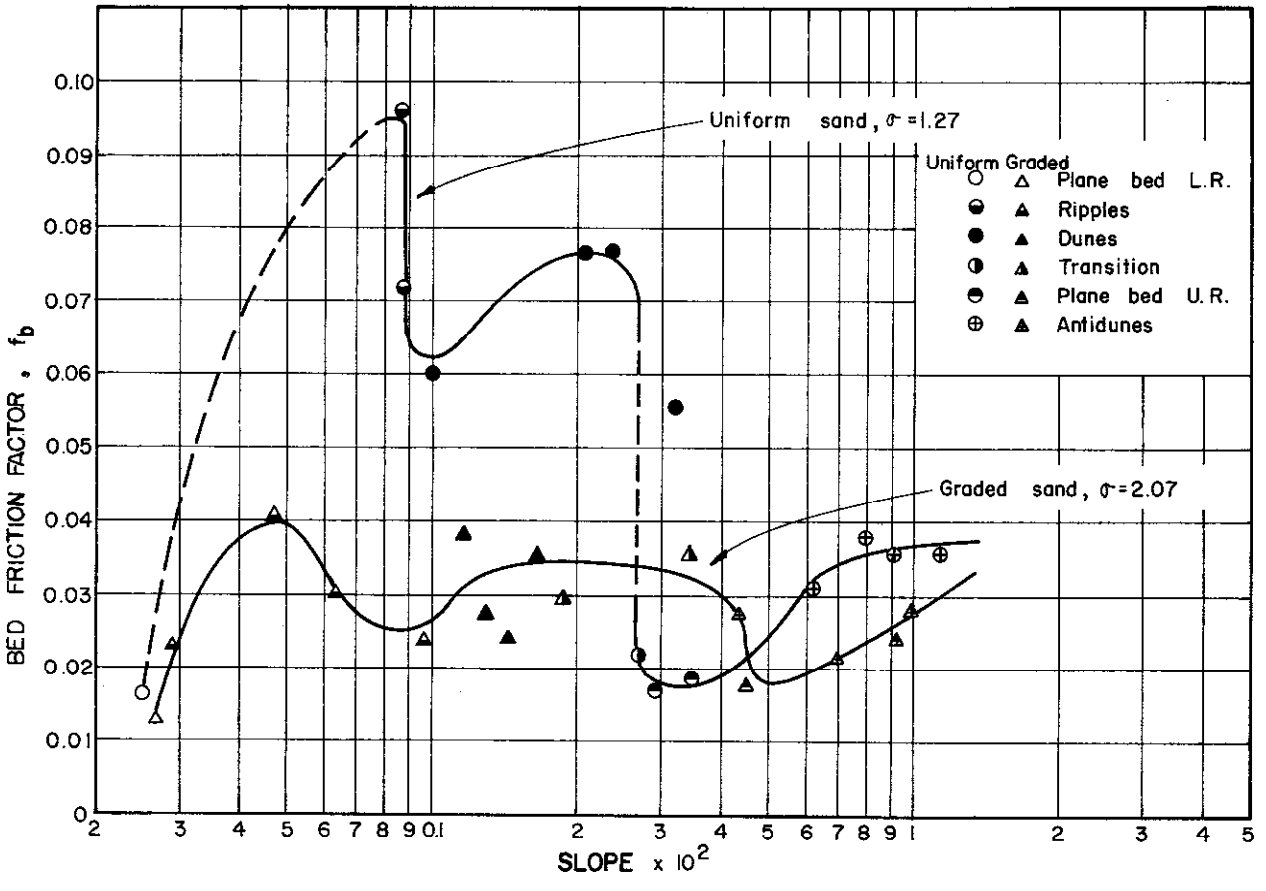
Chezy equations developed for channels with rigid boundaries, the regime equations (*12*) quite commonly referred to when attempting to design stable alluvial channels, and Einstein's and Barbarosa's treatment (*9*) of alluvial river channel roughness have all been used to estimate channel resistance and average velocity. The latter two methods are probably the most successful of those cited.

The flume data referred to in table 2 follow Einstein's river curve, which relates V/V_*'' to ψ', with reasonable success within the range of ψ' values for which ripples and dunes occur. However, for both large and small values of ψ', the data departs somewhat systematically and radically from the proposed curve. Hence, some modification is required when it is applied to laboratory data (fig. 9).

The occurrence of the separation zones downstream of the ripples and dunes suggests that the cross sectional area through which the flow is assumed to occur should be reduced to eliminate the separate zones. This reduces the actual depth D to an effective depth D' and increases the average velocity V to an effective velocity V'. Using these concepts of depth and velocity, Simons and Richardson (*17*) devised a useful method of predicting the average velocity in alluvial channels. The method involved establishing a relation between V and V_* for the plane bed runs for each median fall diameter and neglecting the difference in resistance to flow and relative roughness with and without sediment movement for the plane bed. The resultant relation (fig. 10a), which is essentially a straight line that passes through the origin, implies that for plane bed

$$V \alpha V_* \text{ and } V = \frac{C}{\sqrt{g}} \sqrt{g\,DS}$$

With forms of bed roughness other than plane bed, the points relating V and V_* fall to the right of the relation for plane bed, indicating a greater resistance to flow, the magnitude of which is proportional to the displacement to the right of the line. Assuming that this dis-



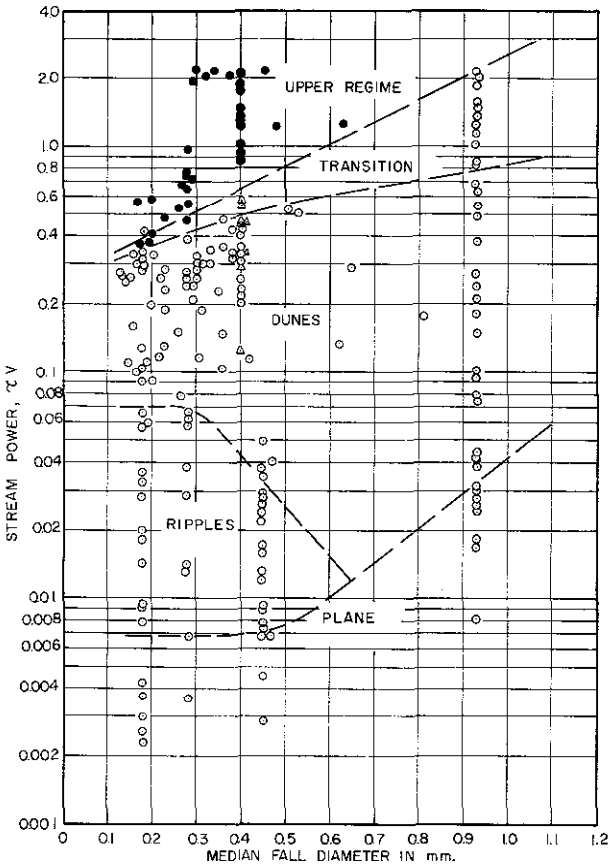


FIGURE 8. — Relation of stream power and median fall diameter of bed material to form of bed roughness.

placement is related to the volume of the flow within the separation zones downstream of the ripples or dunes, a correction to depth and velocity can be made based upon the assumed magnitude of the separation zones which yields an effective depth D' and velocity V'. Replotting the relation between V and V_* for each run with similar bed roughness and using V' in place of V and $\sqrt{g\,D'S}$ in place of $\sqrt{g\,DS}$ puts the new points essentially on the line representing plane bed conditions. This indicates that if the depth correction $\triangle D$ can be determined, making $D' = D - \Delta D$ and $V' = V\,D/D'$, a useful method of predicting average velocity results, provided the form of bed roughness and median fall diameter of the bed material are known. The complexity of predicting form roughness has already been discussed. Also, the usefulness of this method is limited when dealing with a situation where a small change in stream power will cause a change in bed roughness and a large change in average velocity.

The method applied to stable irrigation canals with dunes having median fall diameters of about 0.28 mm. is illustrated in figure 10a. The straight line passing through the origin is based upon plane bed flume runs of bed materials with median fall diameters of 0.27 and 0.28 mm. Figure 10b relates the depth D to the correction ΔD, which is assumed to be related to the volume of the separation zones relative to the total flow and slope of energy gradient. The method of estimating average velocity, using figure 10, is illustrated in table 4. Also in table 4 the estimated average velocity is given for comparison with the measured average velocity. For most of the canals, agreement is within 10 percent and illustrates the effectiveness of the method.

Certainly the source of considerable error in determining velocity and sediment transport is in evaluating the magnitude and effect of each significant variable — only a few have been discussed here. Future fundamental studies of the variables, to define more precisely their effects on flow in alluvial channels, are essential.

Bed Material Discharge

The mechanics of sediment transport are very complex — many empirical and semi-empirical methods of estimating bed material discharge have been proposed. Of these many methods, two of the most commonly used are the Einstein method (*8*) and the modified Einstein method (*6*). Although the modified method is more accurate, its usefulness is limited because computations are based on actual suspended sediment and velocity data; hence, it is not particularly useful for design problems.

The relation between measured bed material discharge and the bed material discharge computed by the Einstein (*8*) procedure is given in figure 11. It was found that agreement between the computed and measured sediment discharge varied appreciably with size of bed material and the form of bed roughness. Best results occurred in the transition zone connecting lower and upper regime flow. The largest variation occurred within the lower regime, particularly near beginning of motion and with ripples where bed material discharge was relatively very small. Success hinges on beginning with a known average velocity for each run. For design purposes, where it is necessary to estimate the average velocity, results would be less impressive.

A study of the Einstein method by Bishop,[8] in which the flume data cited in table 2 was used, resulted in a rather simple method of

[8] BISHOP, A. A. SEDIMENT TRANSPORT IN ALLUVIAL CHANNELS, A CRITICAL EXAMINATION OF EINSTEIN'S THEORY. Doctor of Philosophy dissertation. On file, library, Colorado State University, Fort Collins. 1961.

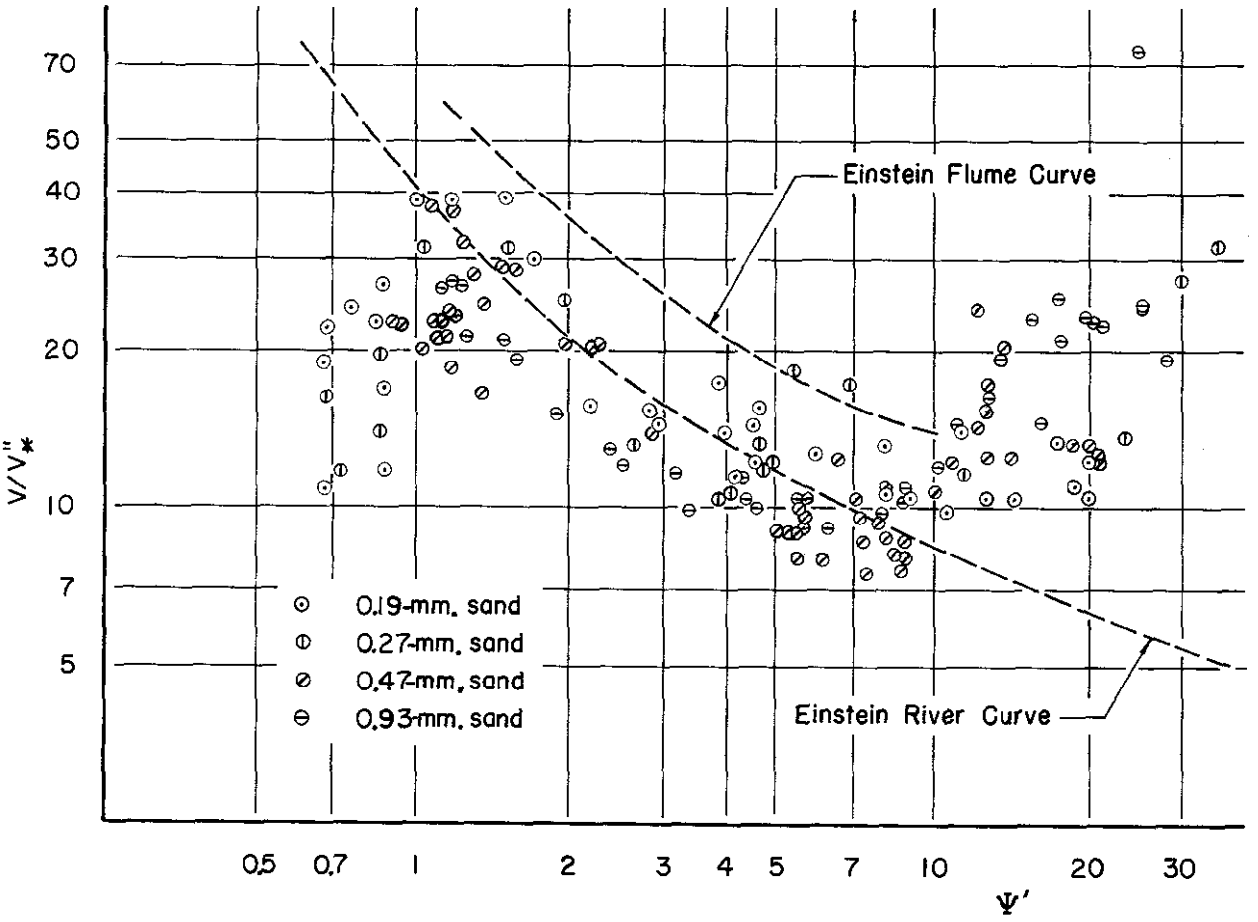


FIGURE 9.—Flume resistance data for various sands in relation to Einstein's bar resistance diagram.

estimating bed material discharge. It involves:

1. The use of the first nine steps as described by Einstein (*8*) to determine ψ', except that R' was determined by a more direct graphical method developed by Vanoni and Brooks (*20*) where

$$\psi' = \frac{\rho_s - \rho_f}{\rho_f} \frac{d_{35}}{R' S}$$

2. Values of ϕ_T in relation to ψ' can be determined from figure 12, from which the total bed material discharge q_T can be estimated from the relation

$$q_T = \phi_T \rho_s g d_{50}^{3/2} \left(\frac{\rho_s}{\rho_f} - 1 \right) 43.2 w$$

The $\phi_T - \psi'$ curves (fig. 12) are very similar to Einstein's $\phi_* - \psi_*$ curves, except that the transport intensity factor

$$\phi_T = q_s \Big/ \rho_s g^{3/2} d_{50}^{3/2} \left(\frac{\rho_s}{\rho_f} - 1 \right)^{1/2}$$

is calculated from the total bed material load, if i_B and i_b are assumed equal to unity. The lower leg of each $\phi_T - \psi'$ curve has been fit by the probability function defined by Einstein. However, A_* and B_* were found to vary systematically with median fall diameter of bed material as shown in figure 13. The upper legs of each of the $\phi_T - \psi'$ curves do not follow the probability relation and have been extended to fit the data empirically. The bed material discharge computed accordingly is compared with the measured bed material discharge in figure 14. The relation is consistently improved throughout the entire range of flow conditions investigated.

To illustrate the possible utility of Bishop's method[9] for estimating bed material discharge in canals and rivers, the total bed material discharge was computed for the (1) canals investigated by Simons,[10] (2) the Niobrara River, (3) the Loup River, (4) the Colorado River, (5) the Solomon River, and (6) the Rio Grande. The computed values are related to a compar-

[9] See footnote 8.

[10] See footnote 6.

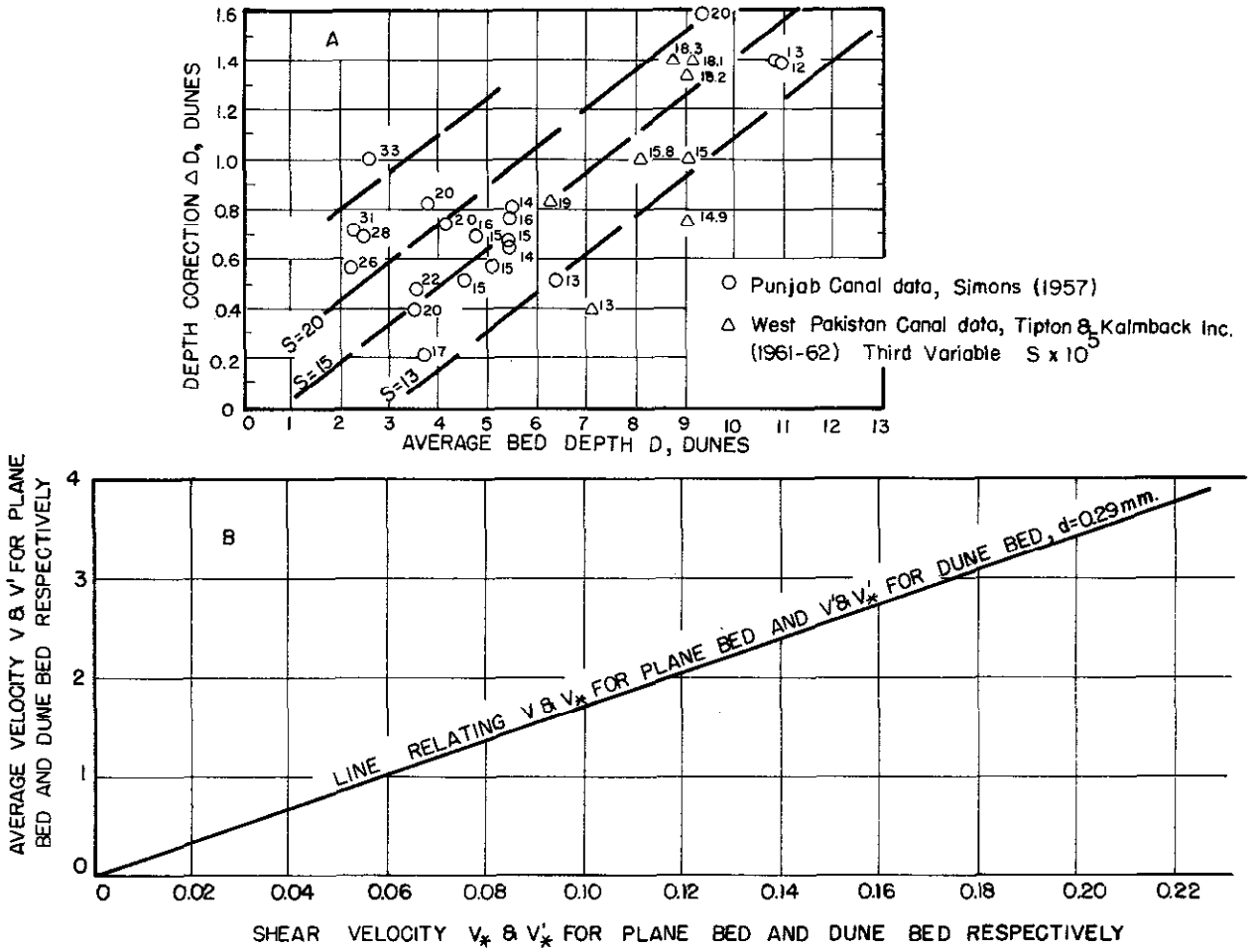


FIGURE 10. — Relation of V and V' to V_* and V'_*, respectively, for bed material with a median fall diameter of about 0.29 mm. for plane bed and dunes.

able total bed material discharge obtained by (1) direct measurement of the total bed material load in the canals, (2) direct measurement at a contracted section for the Niobrara River, (3) by direct measurement at the turbulence flume (*3*) on the Loup River and by the modified Einstein method for the Colorado River, the Solomon River and the Rio Grande (fig. 15). The agreement is best for shallow streams but offers good possibilities for all streams. However, as is the case for the flume data the success achieved, when the method to field conditions is applied, is dependent on the accuracy with which the stream velocity can be estimated or measured.

Future Studies

A series of experiments are being considered, oriented toward:

1. A study of the mechanics of the formation of bed roughness in alluvial channels and a statistical analysis of their occurrence.
2. Determination of the magnitude of lift and drag on sediment particles.
3. Determination of fall velocity of gravel particles.
4. Measurement and evaluation of the role of turbulence on resistance to flow and sediment transport in alluvial channels.

The study of bed roughness will be initiated in flume 3 with a lightweight bed material (Vestyron) used with a specific gravity of 1.04-1.06. When this facility is used, dunelike roughness elements can be generated by the flow at very small Reynolds numbers. Actually, the individual motion of a single particle can be followed and studied over an appreciable distance.

The magnitude of the lift and drag forces on particles is being studied, using individual beads mounted on a fine wire. The force on the bead is measured with calibrated semi-conductor

TABLE 4.—*Comparison of measured average velocity to computed average velocity, using figure 10 for stable canals with dune bed and median diameter of bed material of approximately 0.29 mm.*

Depth D	Slope $S\times10^3$	ΔD	D'	$\sqrt{gD'S}$	V'	Computed V	Measured V	Percent error
PUNJAB DATA, SIMONS (see footnote 6)								
2.6	0.33	0.9	1.7	0.135	2.44	1.59	1.4	+13.6
3.45	.22	.75	2.70	.138	2.40	1.88	2.16	−12.9
2.24	.31	.76	1.48	.122	2.10	1.39	1.53	− 9.15
2.20	.26	.50	1.70	.119	2.05	1.58	1.50	+ 5.3
3.74	.17	.54	3.20	.132	2.30	1.97	2.28	−13.6
5.10	.15	.66	4.44	.146	2.50	2.17	2.27	− 4.4
5.42	.15	.70	4.72	.151	2.60	2.26	2.30	− 1.8
5.43	.14	.52	4.91	.154	2.65	2.40	2.25	+ 6.7
4.97	.15	.64	4.33	.145	2.50	2.18	2.14	+ 1.9
4.53	.15	.57	3.96	.138	2.40	2.10	2.14	− 1.9
6.36	.13	.53	5.83	.156	2.70	2.47	2.49	+ .8
5.44	.16	.75	4.69	.155	2.70	2.33	2.32	+ .4
2.46	.28	.70	1.76	.126	2.20	1.57	1.56	+ .7
3.50	.20	.67	2.93	.137	2.40	2.01	2.16	− 7.0
3.78	.20	.71	3.07	.140	2.40	1.95	1.88	+ 3.7
5.47	.14	.54	4.93	.149	2.60	2.34	2.15	+ 8.8
4.16	.20	.77	3.39	.148	2.55	2.08	2.12	− 1.9
4.73	.16	.65	4.08	.145	2.50	2.16	2.12	+ 1.9
10.80	.13	1.25	9.55	.20	3.45	3.08	2.84	+ 8.5
10.90	.12	1.20	9.70	.194	3.35	2.99	2.87	+ 4.2
WEST PAKISTAN DATA, TIPTON AND KALMBACH, INC. (see footnote 7)								
7.0	0.132	0.63	6.37	0.169	2.92	2.66	2.74	− 2.9
10.0	.149	1.40	8.60	.203	3.51	3.02	3.36	−10.1
9.0	.15	1.27	7.73	.193	3.34	2.87	3.02	− 5.0
8.1	.158	1.15	6.95	.188	3.25	2.79	2.86	− 2.4
8.7	.183	1.39	7.31	.208	3.60	3.03	2.97	+ 2.01
6.0	.193	1.04	4.96	.175	3.03	2.51	2.67	− 6.0
9.1	.181	1.46	7.64	.211	3.65	3.06	3.10	− 1.3
9.0	.182	1.45	7.55	.210	3.64	3.06	3.10	− 1.3
9.3	.20	1.57	7.73	.223	3.85	3.20	3.15	+ 1.6

strain gages. This method of measuring drag on a wire was originated by Sharp (*15*) to determine velocity in pipes and open channels.

The measurement of turbulence in water has been a difficult task because of many factors — principally inadequate turbulence measuring equipment. However, preliminary work done in flume 3, using the electrokinetic probe reported by Chuang and Cermak (*5*), indicates the feasibility of measuring turbulence in both pipes and open channels. The probe by Chuang (*4*) has also been used in flume 3 to measure the frequency of shedding of vortices downstream of cylinders. The results thus obtained are in excellent agreement with those obtained by Roshko (*14*) in a wind tunnel with a hot wire anemometer.

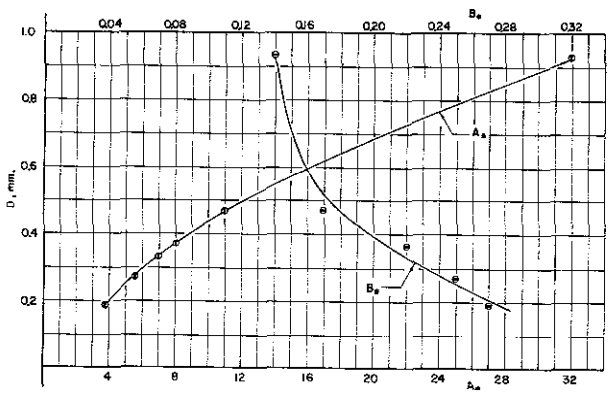


FIGURE 11. — Measured sediment discharge compared to sediment discharge computed by Einstein's function.

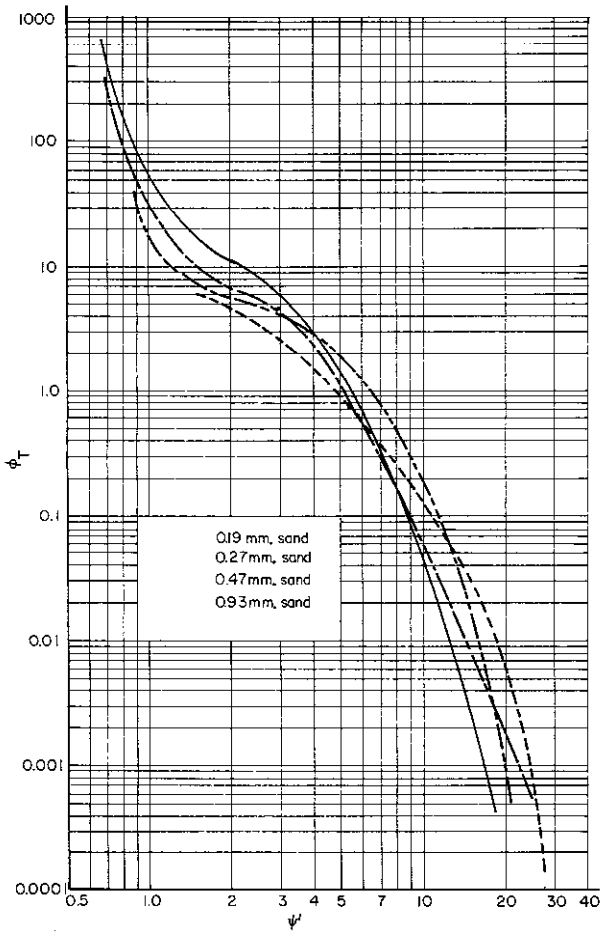


FIGURE 12. — Composite ϕ_T-ψ' curves for various sands from flume data.

Symbols and Units

Symbol	Definition	Units
A_*	Constant for ϕ_T	...
B_*	Constant for ψ_*	...
$C/\sqrt{g}$	Chezy coefficient	...
D	Water depth	Feet
ΔD	Correction to depth	Feet
D'	Effective depth $(D-\Delta D)$	Feet
d_{50}	Size of bed material for which 50 percent is finer, etc.	Feet
f_b	Friction factor of the bed	...
F_r	Froude number	...
i_b	Fraction of the bed material in a given size range.	...
i_B	Fraction of bedload in a given grain size.	...
q_s	Corresponding suspended load rate.	Pounds per second per foot
q_T	Total sediment discharge	Tons per day
R	Hydraulic radius	Feet
R'	Hydraulic radius with respect to the grain.	Feet
R''	Hydraulic radius with respect to channel irregularities.	Feet
S	Slope	...
V	Average velocity	Feet per second
V'	Effective velocity	Feet per second
V_*	Shear velocity $\sqrt{gDS}$	Feet per second
V'_*	Effective shear velocity	Feet per second
w	Width of channel	Feet
ρ_s	Density of fluid	Slugs per cubic foot
ρ_f	Density of solids	Slugs per cubic foot
τ	Shear stress	Pounds per square foot
ϕ	Intensity of sediment transport.	...
ϕ_*	Intensity of transport for individual grain size.	...
ϕ_T	Intensity of transport for total bed material load.	...
Ψ'	Intensity of shear on representative particle.	...
Ψ_*	Intensity of shear for individual grain.	...

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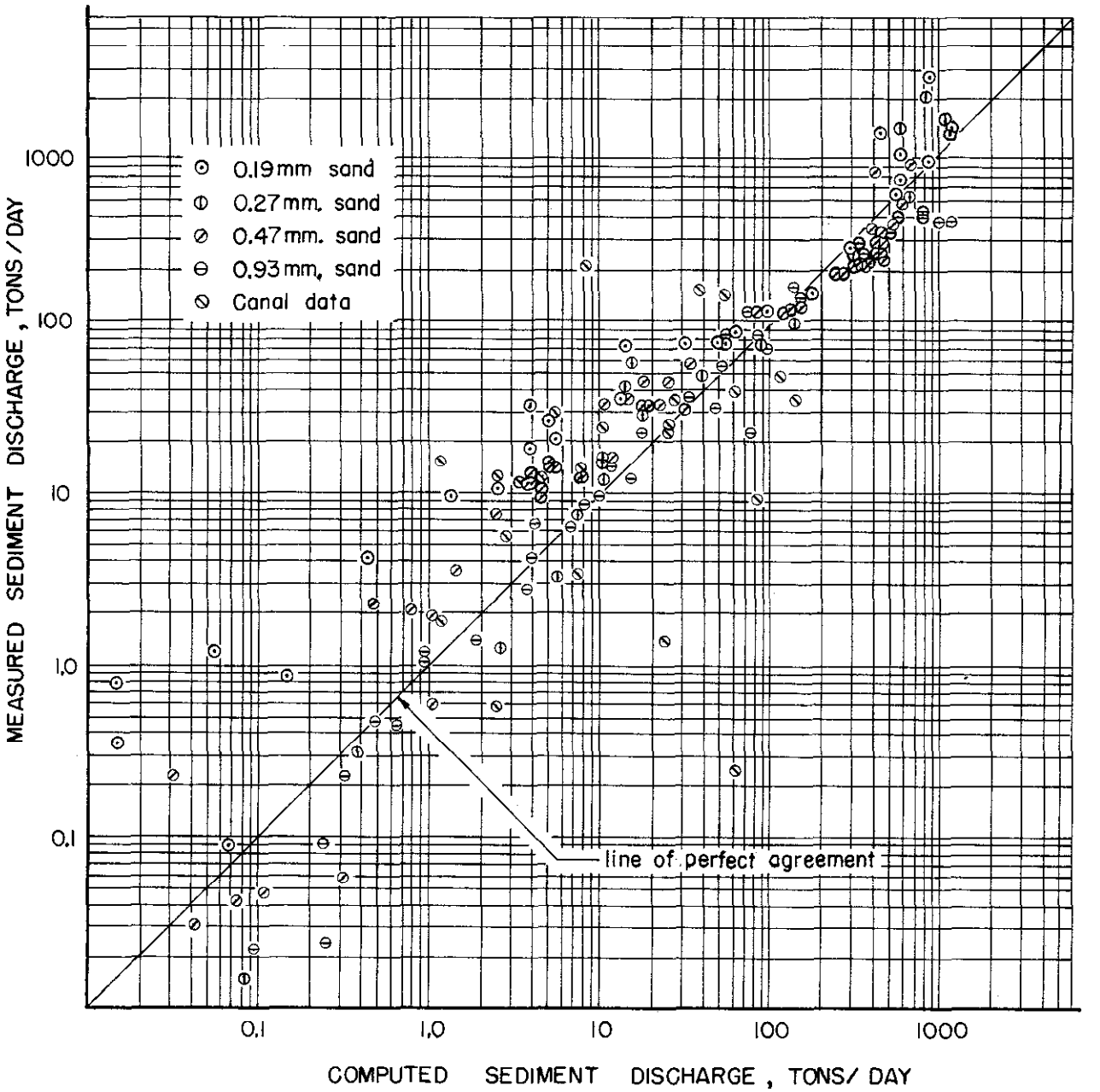


FIGURE 13. — Values of A_* and B_* for theoretical probability curves from laboratory data.

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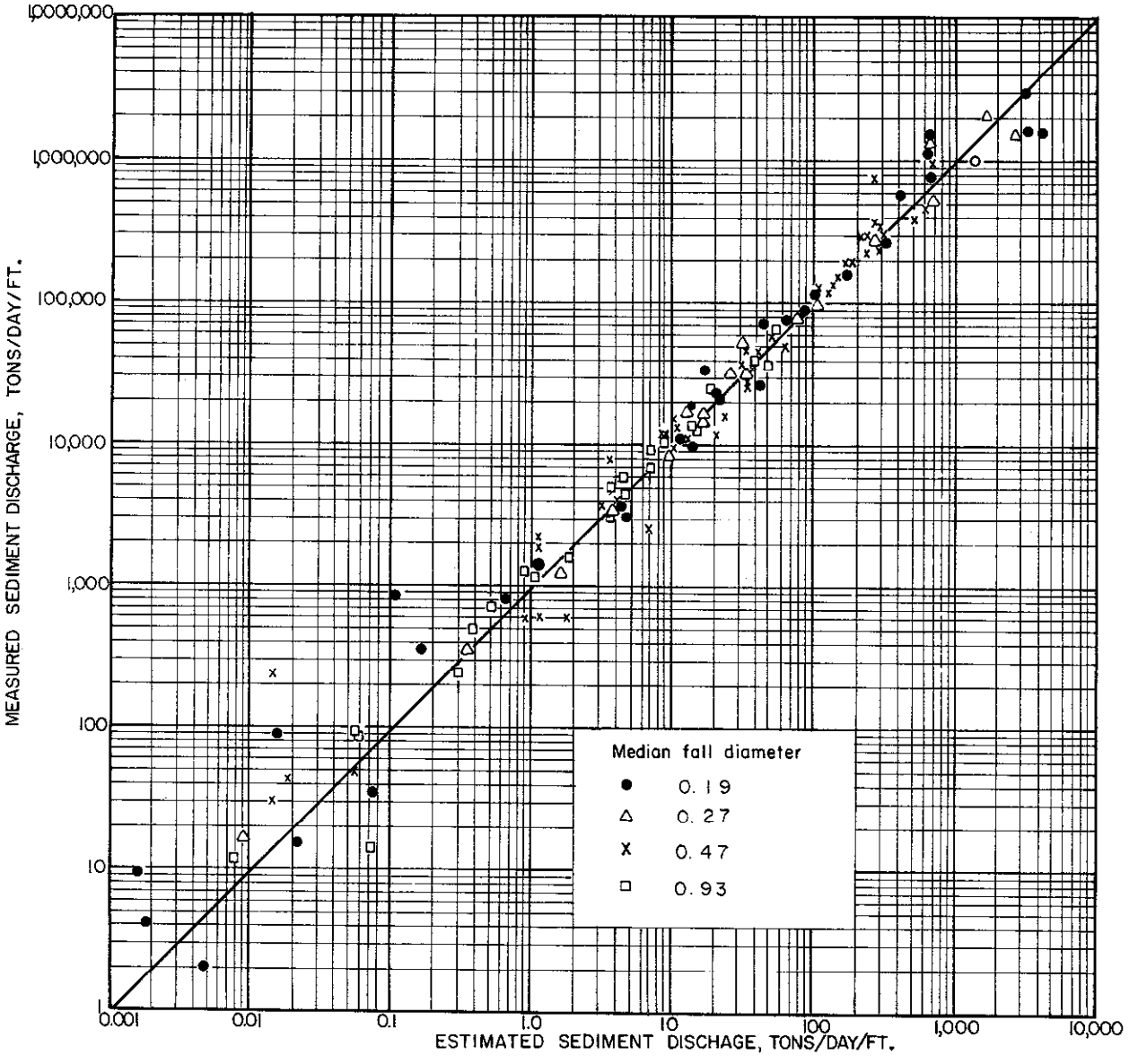


FIGURE 14. — Measured sediment discharge compared to sediment discharge computed using modified ϕ_T-ψ' relation.

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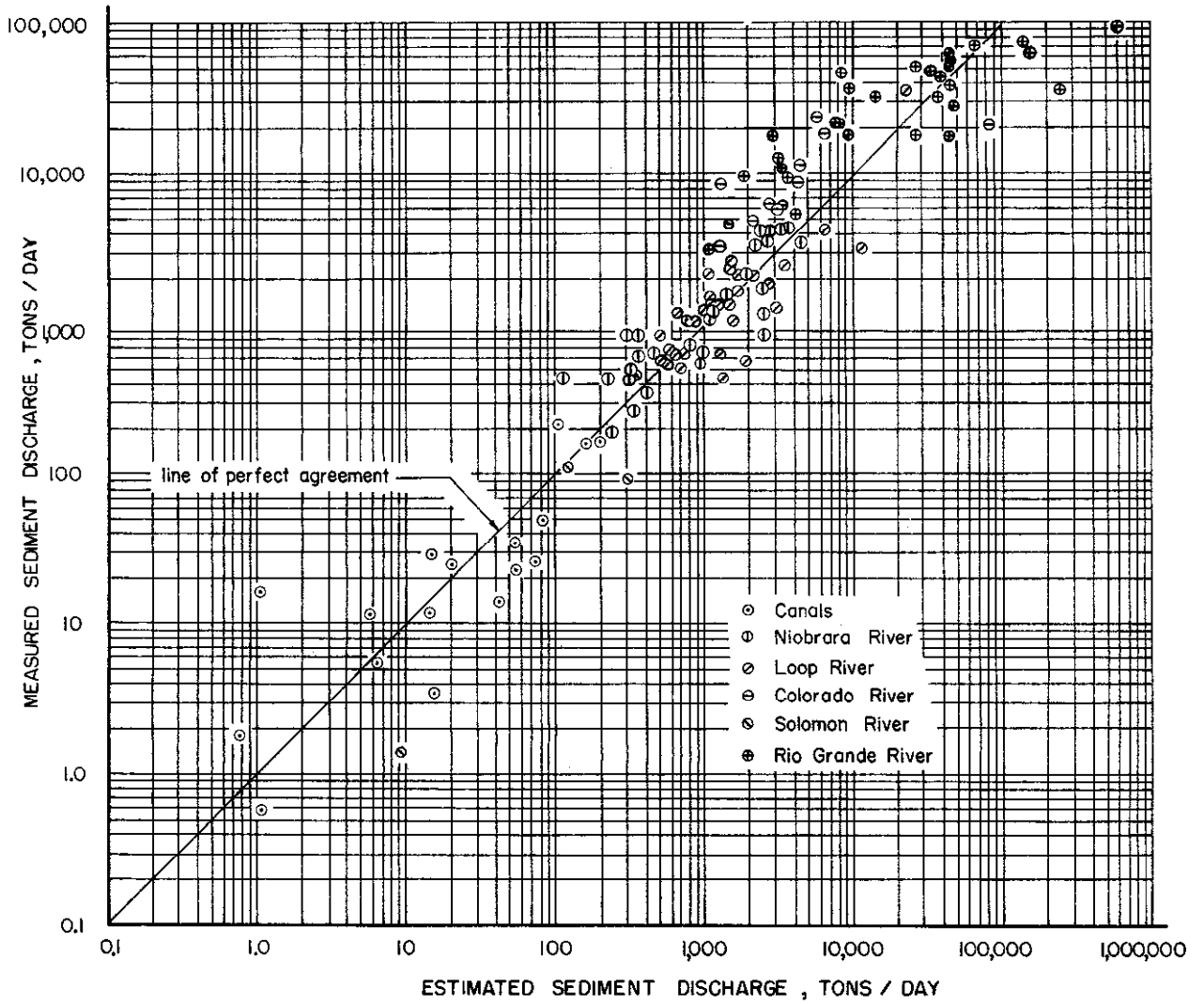


FIGURE 15.—Comparison of bed material load estimated from laboratory curves with measured bed material load.

DEEP RIVER VELOCITY AND SEDIMENT PROFILES AND THE SUSPENDED SAND LOAD

[Paper No. 28]

By FRED B. TOFFALETI, *U.S. Army Engineer Division, Lower Mississippi Valley, Vicksburg, Miss.*

Synopsis

A large number of velocity measurements have been made in the Atchafalaya River at Simmesport and in the Mississippi River at Vicksburg to define the vertical velocity distribution. Examination of these data shows the average vertical velocity profile of the two streams to be almost identical at the respective locations. However, a comparison of this measured vertical velocity distribution with that derived by a formula in common usage in analytical sedimentation studies shows these two to be quite different.

Suspended sediment measurements are also made at regular intervals in the Atchafalaya River at Simmesport. Sampling procedure is in

accordance with the Luby method for point measurement. An evaluation of the accuracy of this method is made by a study of 11 sets of measurements covering a wide range of flow conditions. The study indicates that the silt and clay portion of the suspended load was satisfactorily measured, but that there was a significant deficiency in measurement of the sand portion. A detailed study of the fine sand fraction of the suspended load in the 11 examples indicated that the Luby sampling procedure gave fine sand loads averaging about 30 percent less than the apparent actual load. In some instances a part of this deficiency is attributable to abnormal or atypical samples, but even so, economic reasons forbid an adequate number of Luby point samples to measure accurately the sand load in large, deep rivers. The procedure used herein for evaluating the accuracy of sediment measurements would be useful in routine analyses for detecting abnormal or apparently erroneous samples and to approximate the quantity of unmeasured load below the lowermost Luby measuring point.

Introduction

In recent years a considerable number of velocity and suspended sediment measurements have been made in the Atchafalaya and Lower Mississippi Rivers, two large and deep streams in the Lower Mississippi Valley. This paper presents the results of some of these measurements and a token comparison with theoretical and empirical procedures developed to provide this information analytically.

Vertical Velocity Distribution

In the period 1955 through 1960 several hundred vertical velocity profiles were measured in the Atchafalaya River at the Simmesport discharge range (fig. 1). Similar measurements were made in the Mississippi River at the Vicksburg discharge range in the period 1949 through 1956. The results of these measurements in terms of velocity coefficients, or the relation of the mean velocity in the vertical to that at any given depth, are shown in figure 2 and table 1. These data indicate that the average vertical velocity profiles of the two streams are practically identical at the respective locations.

TABLE 1.—*Average velocity coefficient* U/U_y

Proportionate depth	Mississippi River at Vicksburg[1]	Atchafalaya River at Simmesport[2]
[3]*S*	0.915	0.917
0.1	.902	.906
.2	.905	.912
.3	.914	.922
.4	.931	.938
.5	.957	.960
.6	.994	.991
.7	1.045	1.042
.8	1.121	1.120
.9	1.244	1.233
[4]*B*	1.461	

[1] Determined from 500 measurements.
[2] Determined from 473 measurements.
[3] Surface measurement taken at 0.5-ft. depth.
[4] Bottom measurement taken at 1.8-ft. above bottom.

These data were obtained over a period of several years in all seasons and include measurements over a wide range of stage, discharge, and water temperature. A grouping of the data in accordance with conditions of similarity, i.e., season and stage, provided no evidence of distinguishing characteristics attributable to these variables. This approach indicated that the natural scatter or variations due to turbulence obscured the discernment of any seasonal or other effects. Table 2 shows the degree of variation in the 500 sets of Vicksburg measurements by deviations from the average for each tenth of depth. In this tabulation the scatter is more pronounced in the upper and lower regions of flow, particularly the latter. As will be seen later, this greater scatter at the extremities of the velocity profile apparently affects sediment sampling in these areas.

A log-log plot of the velocity coefficients in table 1 applied to any value of mean velocity gives the reasonably accurate vertical velocity distribution formula

$$U_y = 1.15\,U\left(\frac{y}{d}\right)^{0.155} \qquad (1)$$

in which y is distance above the bed, d is the depth of a mean-depth section, U is the average velocity of flow in the mean-depth section, and U_y is the point velocity at y distance from the bed. This velocity distribution for any values of U and d shows that m, the number of velocity units per cycle of 10 of either y or y/d, can be expressed as

$$m = \frac{U}{3.40} \qquad (2)$$

The Von Karman coefficient expressed as $k = \frac{2.3u_*}{m}$ then becomes

$$k = \frac{44.4\sqrt{Sd}}{U} \qquad (3)$$

in which S is the water-surface slope and u_*, the shear velocity, equals $\sqrt{Sdg}$, g being the acceleration due to gravity.

A comparison of the vertical distribution given by formula 1 above with that from formula 3 in USDA Technical Bulletin 1026 [1] shows the two to be quite different. USDA formula 3 gives velocities ranging from about 40 per-

[1] H. A. EINSTEIN. THE BED-LOAD FUNCTION FOR SEDIMENT TRANSPORTATION IN OPEN CHANNEL FLOWS. U.S. Dept. Agr. Tech. Bul. 1026. 1950.

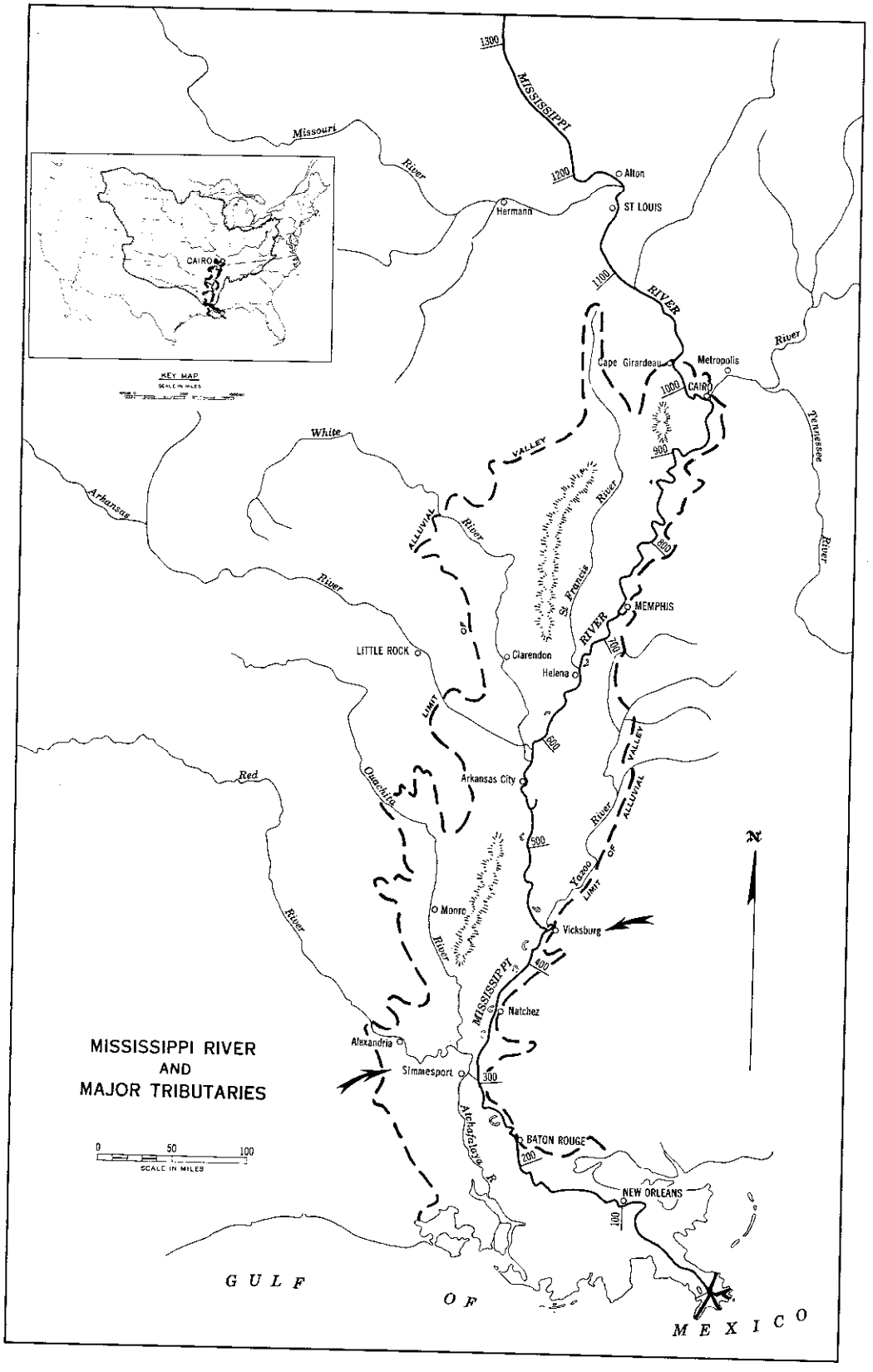


FIGURE 1. — Location of the Simmesport and Vicksburg stream gaging stations.

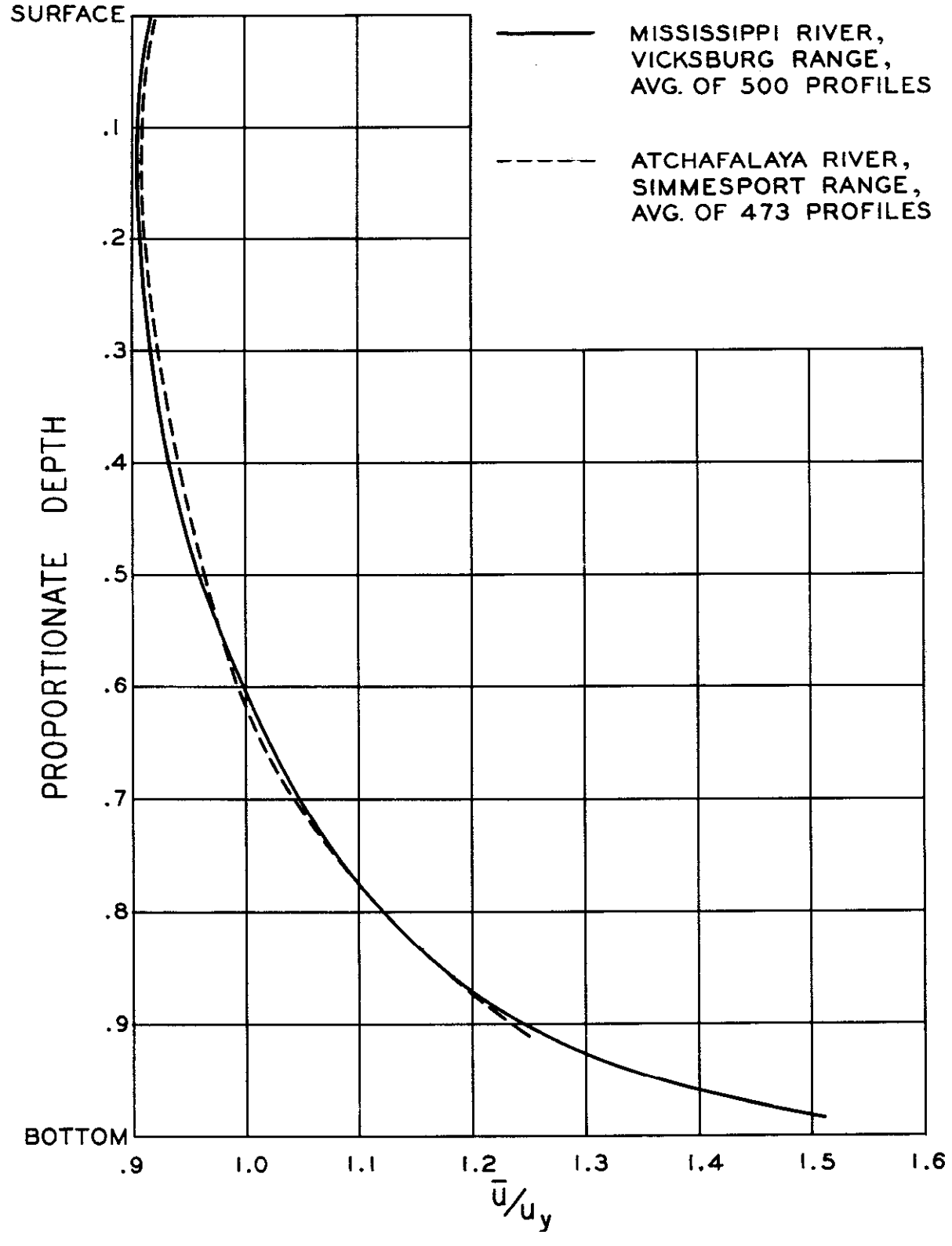


FIGURE 2. — Velocity profiles at Simmesport and Vicksburg ranges.

TABLE 2.—*Summary of deviations from the average velocity coefficient at each tenth of depth for the 500 measurements in the Mississippi River at Vicksburg*

Percent deviation from average	Proportionate depth										
	S [1]	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	B [2]
−20>−20										4	44
−19											6
−18										2	9
−17	1									1	7
−16											15
−15	2							1	3	3	9
−14	1	2						1		7	11
−13	2		1					1	3	7	11
−12	3								5	18	15
−11	5	2	1				1	5	3	7	13
−10	10	1	1				2	3	6	22	13
−9	14	5	2	2		1		4	8	17	10
−8	18	10	3	2		2	4	8	11	13	16
−7	20	17	4	7	2	4	3	14	15	15	10
−6	26	24	14	6	6	9	10	13	23	26	12
−5	29	41	28	17	12	10	22	13	37	20	15
−4	34	37	38	25	16	18	31	37	29	18	11
−3	36	35	50	49	33	32	30	37	32	23	18
−2	26	44	53	72	60	57	59	39	27	26	16
−1	31	33	52	58	77	80	55	45	41	27	22
0	19	35	48	55	67	77	58	44	29	33	19
+1	29	30	43	53	81	76	76	53	34	29	13
+2	35	38	32	54	61	55	54	54	33	22	14
+3	28	23	26	34	35	33	34	39	37	15	11
+4	17	31	29	28	20	25	23	28	28	19	9
+5	24	20	28	15	12	11	15	24	24	25	16
+6	17	24	17	7	6	4	13	15	24	12	12
+7	18	16	12	8	5	4	4	8	12	12	9
+8	13	12	7	3	2	2		8	10	10	13
+9	7	6	3	2	1		2	1	8	9	11
+10	9	3	4		4		1	2	6	15	5
+11	7	5	2				1		1	12	7
+12	5	2	1	1			1	1	4	6	6
+13	2	1					1			2	10
+14	2	1		2				2	2	3	2
+15	1	1							2	5	6
+16	1										3
+17									2	4	4
+18	2									1	5
+19	1								1		2
+20>+20	5	1	1							10	40

[1] Surface measurement taken at 0.5 ft. depth.
[2] Bottom measurement taken at 1.8 ft. above bottom.

cent higher at the surface to about 100 percent higher near the bed. In this comparison the Technical Bulletin 1026 formula 3 was used in the form

$$\frac{U_y}{u_*} = 8.50 + 5.75 \log_{10}\left(\frac{y}{\Delta}\right)$$

If this formula is stated as

$$\frac{U_y}{u_*} = C + 5.75 \log_{10}\left(\frac{y}{\Delta}\right)$$

it is found that to reproduce the vertical velocity distribution determined by formula 1 would require C to be a negative number. Further, C would not be a constant range of y values, bottom to surface.

Although the procedure outlined in Technical Bulletin 1026 for determining sediment loads has previously been used in Atchafalaya River studies, the preceding comparison of formulas indicates that use of the vertical velocity distribution formula 3 for sediment studies of either the Atchafalaya or Lower Mississippi River would give loads greatly in excess of measured loads.

Sediment Sampling and the Total Load

Suspended sediment sampling was initiated in 1949 on the Lower Mississippi River at Baton Rouge, and in 1951 on the Atchafalaya River at Simmesport. In 1958 the Baton Rouge operation was transferred to Red River Landing, about 72 miles upstream. Sampling is generally at 2-week intervals, more frequent during floods when possible, and less frequent during the low-water season. The Luby method for point sampling was adopted for these stations, with 40 samples taken in Mississippi River cross sections, a sample from 5 levels in each of 8 verticals, and 25 samples in the Atchafalaya

River at 5 levels in 5 verticals. Bed material samples are also taken at each river station at each vertical of suspended sediment measurement.

Over the period of observations, which included all types of water years, the measured suspended sediment in the Lower Mississippi River averaged 84 percent finer than 62 microns at Baton Rouge and 71 percent at Red River Landing. In the Atchafalaya River at Simmesport the average was 76 percent. Bed material samples from the Baton Rouge and Simmesport stations show the dominant material to be fine and very fine sand, and very little of the sand portion of the measured suspended sediment at these stations exceeded ¼ mm. in size. The characteristics of these suspended sediments, then, are such that the Luby five-point method would be expected to give an overall total-load accuracy within 1 or 2 percent. A summary of the results of observations at the three stations discussed above is shown in table 3.

Evaluation of Measured Load

The discussion that follows attempts to evaluate the accuracy of suspended sediment measurements and load determinations of the Atchafalaya River at Simmesport.

The laboratory analyses of the suspended sediment samples indicate generally that the material finer than 62 microns is fairly evenly distributed in the vertical, and that a major portion of this fraction of the load is in the size range of very fine silts and clays. As these fine materials comprise about 76 percent of the total load on a long-term basis, it is considered that this major portion of the total load is accurately measured. For an estimate of accuracy of measurement of the suspended sands, the fine sand fraction (¼ to ⅛ mm.) was selected for study. This is generally the dominant bed material and is found in transport at all levels in the channel section except at extremely low flows.

Figure 3, *A* to *K*, presents 11 examples of fine sand distribution as determined from analyses of the point samples. These examples cover a wide range of discharge with water depths at the verticals varying from 83 to 46 ft. on the deep side of the channel, and from 54 to 26 ft. on the shallow side. The figures show measured data for six levels in the verticals. The five measurements and average thereof on the extreme right in each figure show the results of measurements made at 3 ft. from the bed in each vertical. This is a measurement somewhat below the lowest Luby point in each case and is not used in load computation. Its purpose is to attempt to define sediment distri-

TABLE 3.—*Summary of measured suspended sediment loads* [1]

Water year (Oct.-Sept.)	Total measured sediment load (in 1,000 tons)	Sand-silt ratio				Water-year discharge (1,000 d.s.f.)	Average sediment concentration (p.p.m.)
		Sand		Silt			
		In 1,000 tons	Percent	In 1,000 tons	Percent		
ATCHAFALAYA RIVER AT SIMMESPORT, LA.							
1951–52	196,460	48,890	25	147,570	75	80,800	900
1952–53	135,230	28,440	21	106,790	79	56,960	880
1953–54	54,130	13,110	24	41,020	76	31,980	627
1954–55	93,360	24,080	26	69,280	74	50,425	686
1955–56	67,175	15,450	23	51,730	77	49,080	507
1956–57	225,474	55,700	25	169,774	75	74,059	1,126
1957–58	214,390	48,082	22	166,308	78	89,413	887
1958–59	83,230	20,944	25	62,286	75	55,729	553
1959–60	131,878	24,153	18	107,725	82	69,333	704
1960–61	133,372	40,524	30	92,848	70	76,814	643
LOWER MISSISSIPPI RIVER AT BATON ROUGE AND RED RIVER LANDING, LA. [2]							
1949–50	548,330	107,770	20	440,560	80	245,200	828
1950–51	575,280	67,600	12	507,680	88	224,810	947
1951–52	408,390	73,820	18	334,570	82	200,660	754
1952–53	212,580	28,920	14	183,660	86	142,200	552
1953–54	107,730	14,090	13	93,650	87	88,660	449
1954–55	211,490	39,930	19	171,550	81	137,460	570
1955–56	161,220	25,920	16	135,300	84	127,530	468
1956–57	291,388	53,043	18	238,345	82	172,875	624
1957–58	325,774	95,203	29	230,571	71	195,653	616
1958–59	230,504	78,693	34	151,811	66	129,253	660
1959–60	318,234	77,219	24	241,015	76	163,850	718
1960–61	231,754	71,471	31	160,283	69	168,133	510

[1] The sand fraction is the material retained on the No. 230 sieve (0.062 mm.). The silt fraction includes all of the fine material passing the No. 230 sieve.

[2] Sampling at Red River Landing after 1 January 1958.

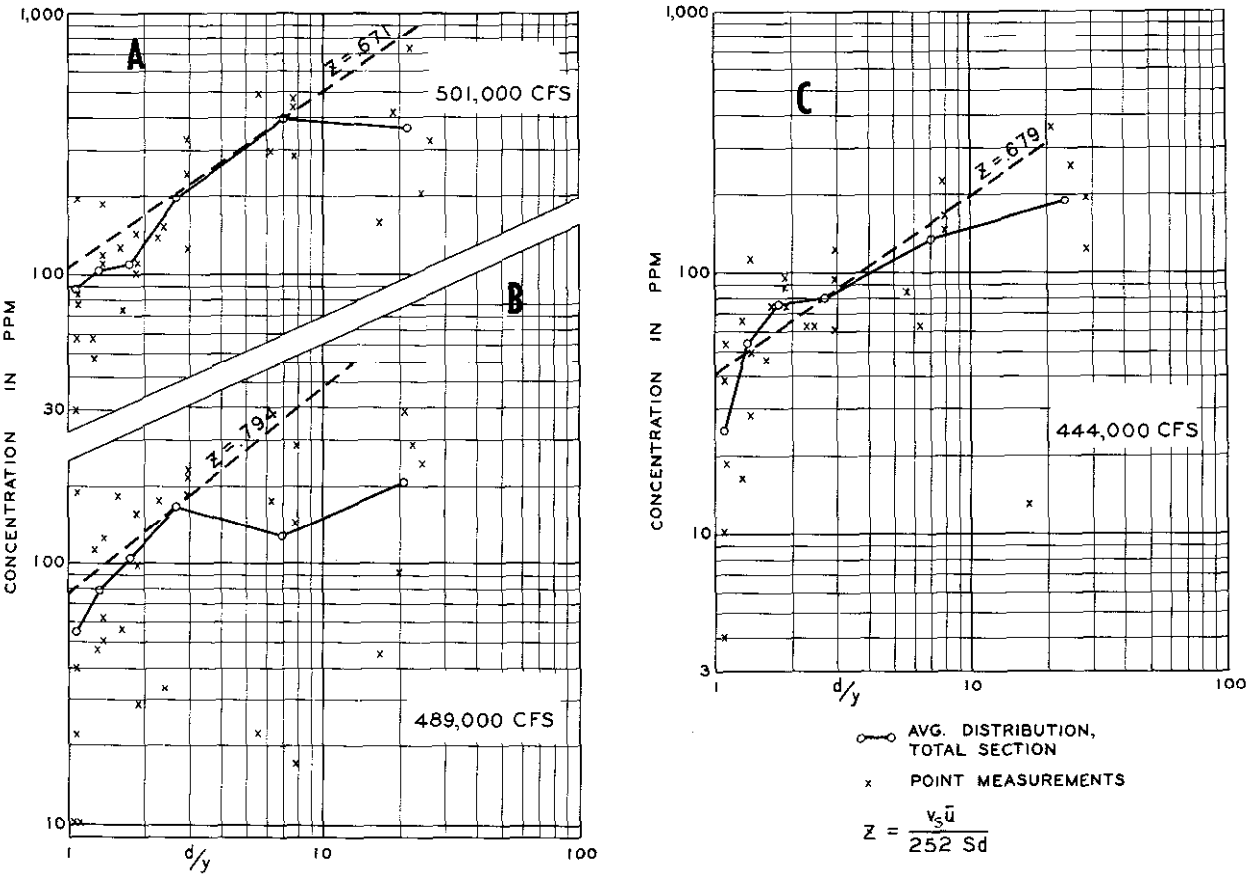


FIGURE 3. — Measured suspended fine sand at 6 levels at each of 5 verticals in cross section of Atchafalaya River at Simmesport, La.: *A*, for 501,000 c.f.s.; *B*, for 489,000 c.f.s.; *C*, for 444,000 c.f.s.

bution nearer the bed than dictated by the 5-point Luby method. Although, for the purpose stated, it would be desirable to obtain samples closer to the bed than 3 ft., experience in sampling in deep turbulent flow has shown this to be the lowest practicable sampling level from considerations of possible loss or damage to instruments. The plot in these figures is concentration versus d/y rather than $\frac{d-y}{y}$, a more common procedure. This was done for convenience of load computation, but it was also found that the vertical distribution could be defined by d/y as well as, or better than, by the conventional plot. The distribution is not satisfactorily definable by a single line in either case.

The plot of measured points in figure 3 shows a wide scatter in fine sand concentration values at all levels in each example, but in most of the examples the scatter appears to be somewhat less at and near middepth. The reason for this may be due to fluctations in the velocity pattern as indicated in table 2. The velocity coefficient is relatively stable at and near middepth, but fluctuates to an increasing degree toward each extremity of the vertical profile.

In order to better define the vertical distribution of fine sand, each common level of measurement across the channel was averaged. In essence, this produced an average vertical distribution profile in a mean depth channel section. For comparison with the measured average vertical distribution, a theoretical vertical distribution of fine sand is indicated by the dashed line in each example. The evaluation of the coefficient Z for this theoretical distribution was determined by the expression

$$Z = \frac{v_s}{k u_*} = \frac{v_s}{k\sqrt{Sdg}} \qquad (4)$$

in which v_s is the settling velocity of fine sand grains in feet per second. By substitution for k from formula 3

$$Z = \frac{v_s U}{252\, Sd}$$

In about two-thirds of the examples presented, the measured data approximate this theoretical distribution from about middepth to 0.85 depth,

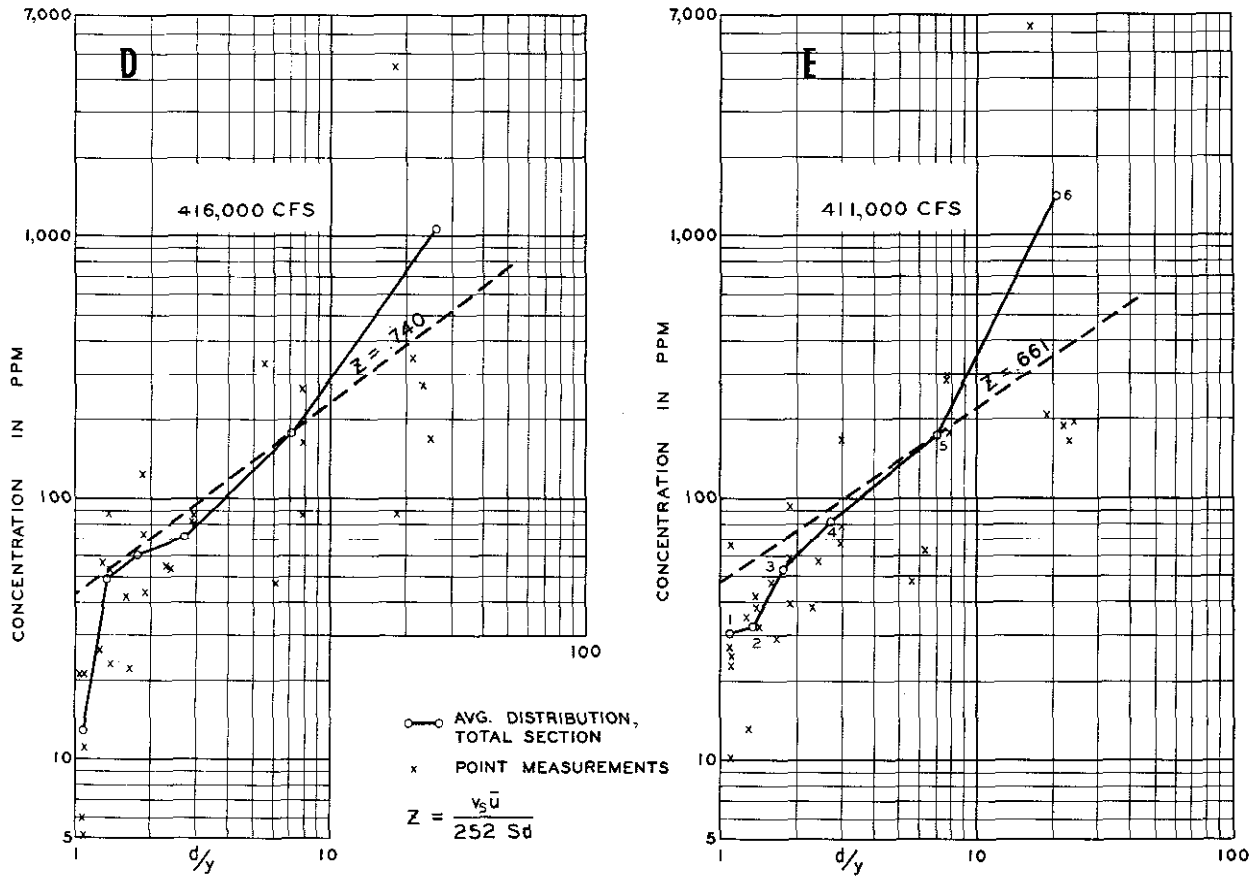


FIGURE 3 (continued). — Measured suspended fine sand at 6 levels at each of 5 verticals in cross section of Atchafalaya River at Simmesport, La.: *D*, for 416,000 c.f.s.; *E*, for 411,000 c.f.s.

or about one-third of the total depth. Several of the other examples, especially as shown in parts *B* and *K* of figure 3, have apparent sampling errors, or at least the questionable samples are not typical if a logical order of sediment density from surface to bottom is an accepted fact. In this regard it is not readily understood why the concentration at the points 3 ft. above the bed should be generally out of line on the low side. In figure 3, *D*, *E*, and *F* the average is high, but in each of these cases the high average is caused by a single sample of very high concentration.

In evaluating the accuracy of the fine sand measurements, the average profiles of figure 3 were each described by three lines. In fitting these lines to the average profiles it was assumed that the profiles approximated the true sediment distribution within the limits of the Luby points, except in cases such as shown in figure 3, *B* and *K*. One of the lines was fitted to points representing the profile in the approximate upper half of the channel section; another was fitted to profile points from approximate middepth to the level $d/y=11.24$; and a third line extended from $d/y=11.24$ to near bed at $y=3.68D_{65}$. The slope of the line to near bed was set as the equivalent of that of a $\frac{d-y}{y}$ versus concentration plot of the middle line. Any slope or position of the middle line plotted with d/y values will intersect a $\frac{d-y}{y}$ plot at $d/y=11.24$. The lower limit of suspended load was assumed to be at the level $y = 3.68\ D_{65}$. For cases such as shown in figure 3, *B* and *K*, the middle line was assumed to be the dashed line, with extension of this line in the case of figure 3, *K*, to include all distribution from $d/y=11.24$ to $d/y=1$. To illustrate this, in figure 3, *E*, the sediment distribution in the upper portion of the channel section would be described by a line passing midway between points 1 and 2 and through point 3. Between this line and a point on $d/y=11.24$, the distribution would be described by a line passing through points 4 and 5. From a point on this line at $d/y=11.24$, the distribution to the lower limit of suspended material is

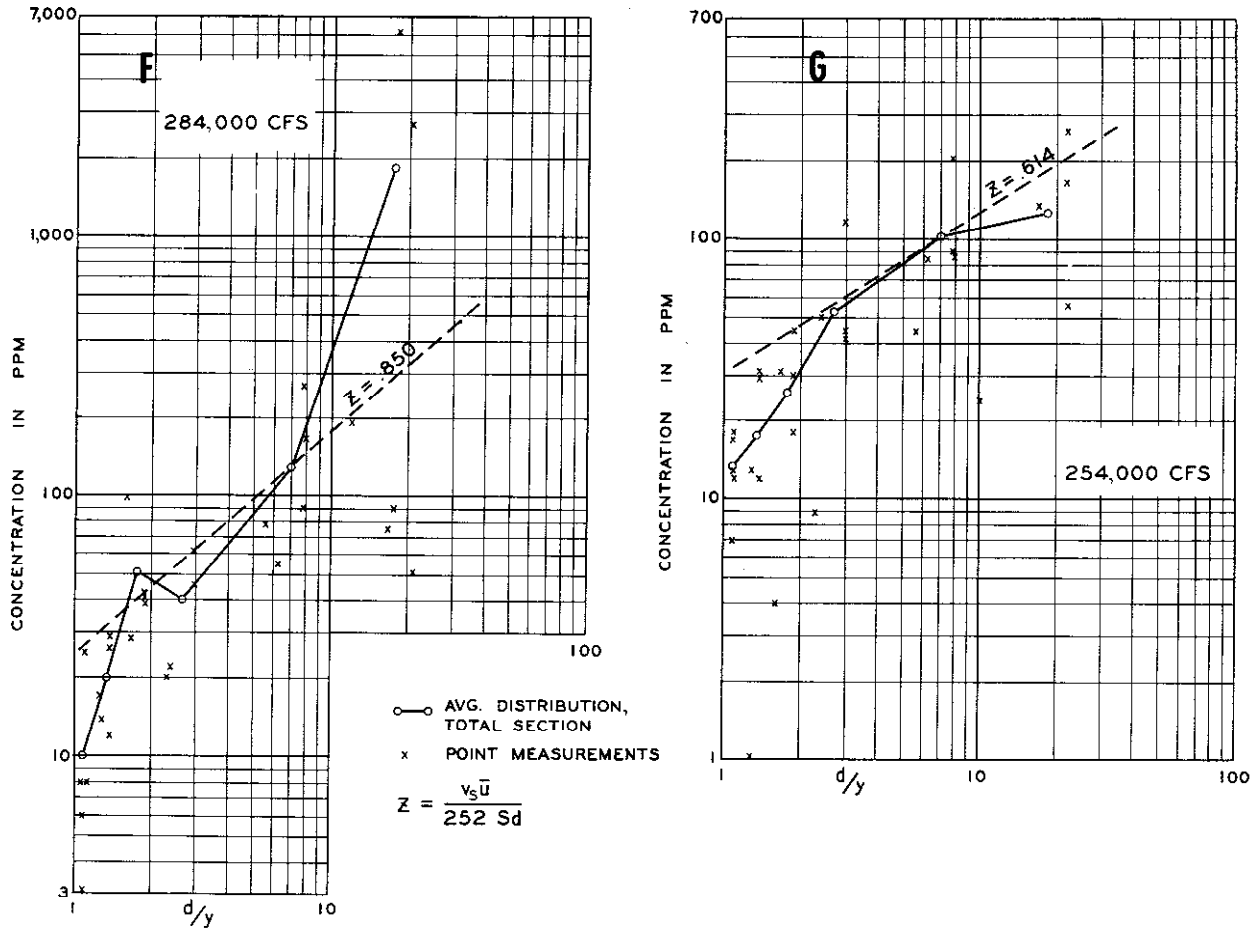


FIGURE 3 (continued). — Measured suspended fine sand at 6 levels at each of 5 verticals in cross section of Atchafalaya River at Simmesport, La.: *F*, for 284,000 c.f.s.; *G*, for 254,000 c.f.s.

on a line having a slope 0.756 times the slope of the preceding intersecting line. This distribution line near the bed will then be at a slope equivalent to a $\frac{d-y}{y}$ plot of the line passing through points 4 and 5.

The above-described procedure permits each of the sediment distribution lines to be formulated in terms compatible with formula 1 for vertical velocity distribution. It is then a rather simple mathematical process to compute the total fine sand load. The results of these computations and a comparison with the loads determined by the measurements alone are shown in figures 4 through 14. These graphs show the respective fine sand load in tons per day per 0.10-ft. increments of depth at any level in the vertical over the entire width of a mean depth channel section. This type of plot is of special interest in that indicated loads from abnormal measurements are quite obvious at first glance. Also, it is immediately noticeable that there is a significant deficiency in sand load as determined from the five-point Luby sampling procedure. This deficiency is in the region near the bed, and is due to the relatively great distance of the lowest Luby point from the bed. In the 11 examples presented, the individual bottom samples are taken from 5 to 13 ft. above the bed.

The procedure for computing the suspended fine sand load is shown by the following example:

In figure 3, *E*, between the levels $y=3.68$ $D_{65}=0.0042$ ft. and $d/y=11.24$ or $y=4.3$ ft., the fine sand concentration C_1 at any point is

$$C_1=59\left(\frac{d}{y}\right)^{0.588}=577/y^{0.588}.$$

From level $y=4.3$ ft. to $d/y=2.1$ or $y=23.0$ ft. the fine sand concentration C_2 is

$$C_2=37.5\left(\frac{d}{y}\right)^{0.778}=767/y^{0.778}, \text{ and}$$

from level 23.0 ft. to the surface the fine sand concentration C_3 is

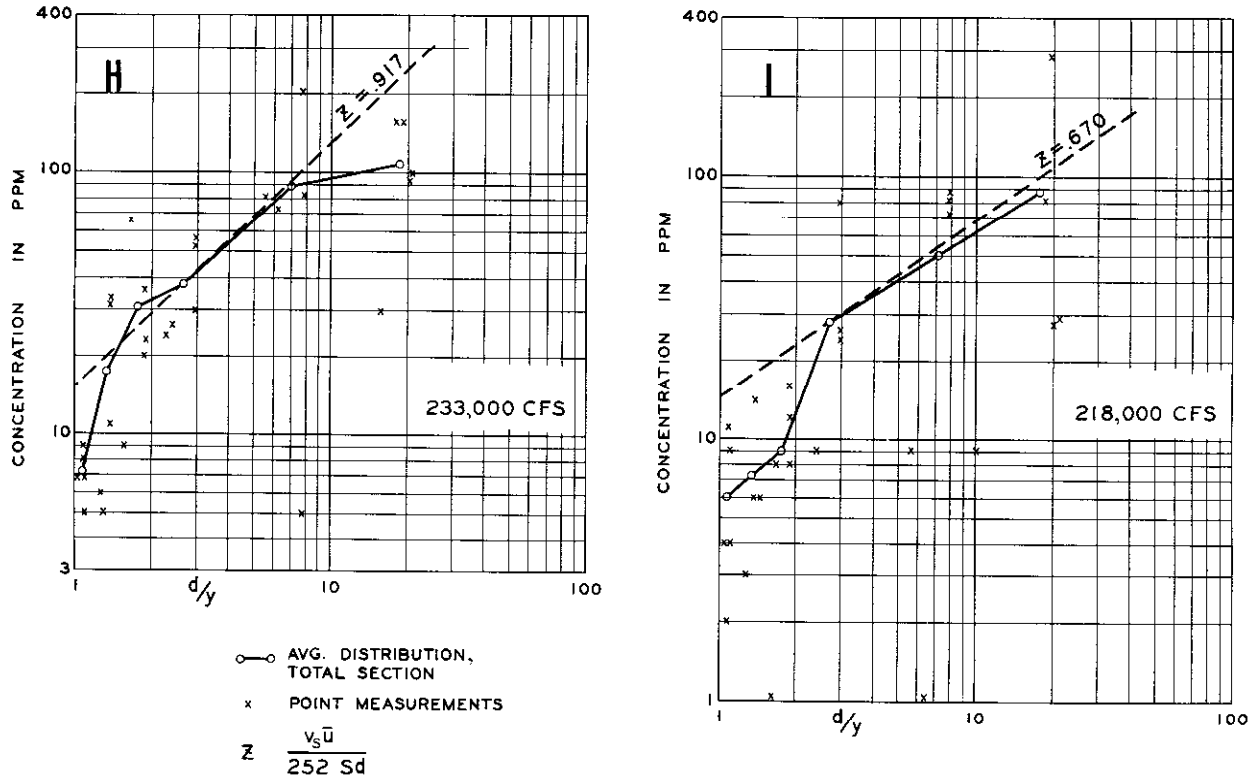


FIGURE 3 (continued).—Measured suspended fine sand at 6 levels at each of 5 verticals in cross section of Atchafalaya River at Simmesport, La.: *H*, for 233,000 c.f.s.; *I*, for 218,000 c.f.s.

$$C_3=23.8\left(\frac{d}{y}\right)^{1.38}=\frac{23.8\,d^{1.38}}{y^{1.38}}$$

These expressions for distribution of fine sand in parts per million are each multiplied by the velocity distribution formula 1, which for this case is $Uy=3.48\,y^{0.155}$, and then multiplied by 4.146, to convert the distribution to tons per day over the width of channel. The resulting expressions are

$$C_1U_y=\frac{8.325}{y^{0.433}};\ C_2U_y=\frac{11{,}066}{y^{0.623}};\ C_3U_y=\frac{343.4\,d^{1.38}}{y^{1.225}}$$

We can now determine the total or incremental sediment loads within the prescribed limits of each by integration. Resultant expressions are:

$$\text{Sediment load } S_1=14{,}683\,y^{0.567}\Big]\begin{matrix}y=4.3\\ y=0.0042\end{matrix}$$

$$\text{Sediment load } S_2=29{,}353\,y^{0.377}\Big]\begin{matrix}y=23.0\\ y=4.3\end{matrix}$$

$$\text{Sediment load } S_3=134{,}613\left(\frac{d}{y}\right)^{0.225}\Big]\begin{matrix}y=23.0\\ y=48.4\end{matrix}$$

To check the accuracy of fitting the distribution lines to measured data, the computed load above the lowest measured value coincident with the computed load graphs in figures 4 through 14 was added to the load encompassed by the measured load graph below that point. This load in every case was within 3 percent of the total measured load, which indicates a satisfactory alinement of distribution lines above the lowermost Luby point. As the analytical treatment of the load below this level logically assumes that the concentration of sand will increase toward the bed, any significant difference in measured and computed load is generally in that portion of the load between the lowest Luby point and the bed. It is thus indicated in the 11 examples presented that an upward adjustment of the measured fine sand loads in amounts from 8 to 84 percent is warranted. The overall average for the 11 cases is +32 percent. If these examples, which were selected at random, are typical and it is assumed that the total meas-

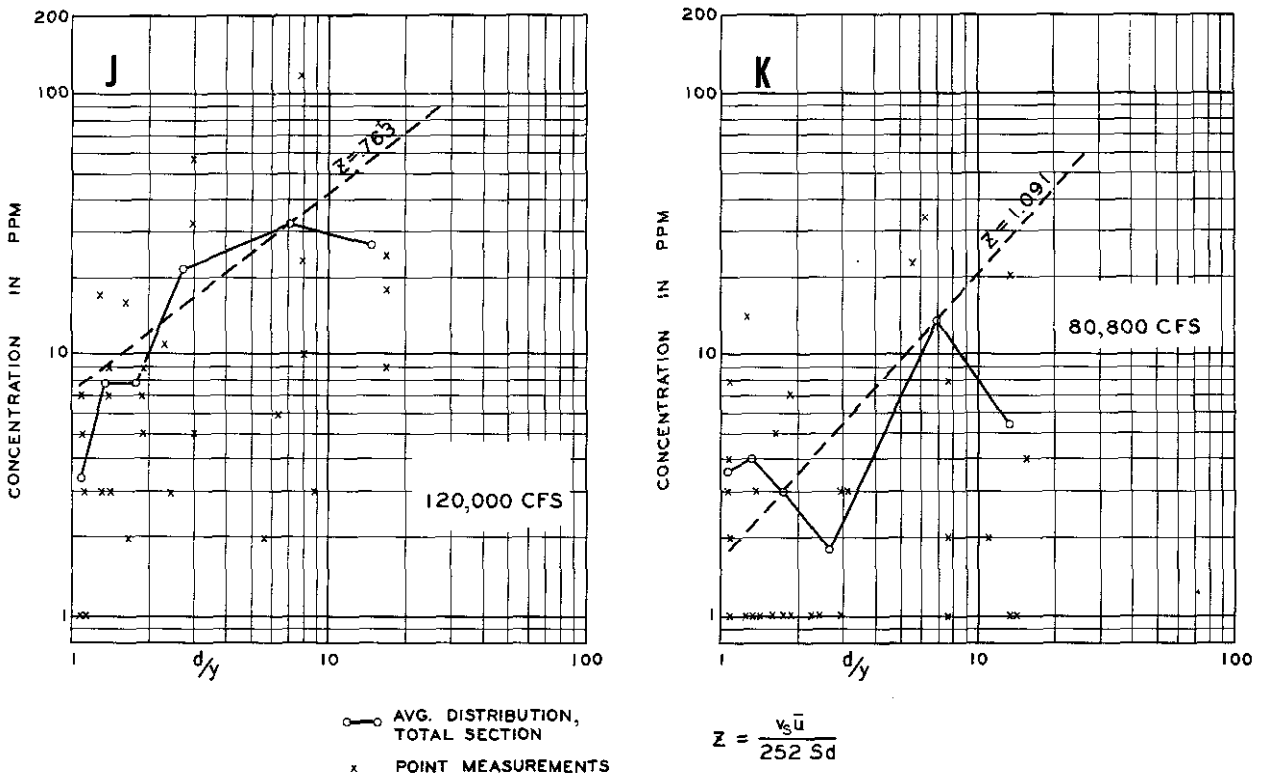


FIGURE 3 (continued).— Measured suspended fine sand at 6 levels at each of 5 verticals in cross section of Atchafalaya River at Simmesport, La.: *J*, for 120,000 c.f.s.; and *K*, for 80,800 c.f.s.

ured sand load in suspension is equally deficient, the unmeasured sand load in the 10-year period would total 106 million tons. The examples used in this study could well be typical, and it is reasonable to assume that measurements of all the sand load are equally short.

Discussion

It must be realized that in field operations an abnormal or atypical sample cannot be recognized. Also, when the samples are received in the laboratory a settling period of at least 2 weeks is required before an analysis can be made. Therefore, since it is several weeks after a set of samples is taken before a load determination can be made, averaging is depended on to equalize the deviations. The unreliability of such dependence is exemplified in the measurements at a flow of 489,000 c.f.s., as shown in figs. 3, *B*, and 5. In this case two samples at the lowermost Luby point in two of the five verticals are apparently in error and in sufficient amount to affect greatly the evaluation of measured load. The total sediment content of each of these samples was about one-half of that in the next two samples 13 ft. farther from the bed. The sand contents were 15 and 22 percent, respectively, of those in the next higher sampling level. It is considered that of the 117,774 tons per day difference between the computed and measured fine sand load, about 55,000 tons is attributable to the two abnormal samples.

Conclusions

The examination and analysis of the velocity and sediment profiles selected for presentation in this paper suggest the following conclusions:

(1) The vertical velocity distribution formula 3 in U.S. Department of Agriculture Technical Bulletin 1026 is not applicable to large rivers, such as the Lower Mississippi River and the Atchafalaya River.

(2) In studies involving the velocity and sedimentation characteristics of large rivers, the treatment of entire channel sections as a unit is a satisfactory procedure.

(3) Measurements of the suspended sand load by the Luby method are deficient.

(4) As a supplement to point sampling by the Luby method, additional samples are required near the bed to define sediment transport in that region. A basic weakness in the Luby method

is that, regardless of the number of points measured, more samples will always be taken above middepth than below. Therefore, an excessive total number of samples would be required for adequate sampling below middepth, and the cost would be prohibitive.

(5) The accuracy of point sampling should be evaluated by a plot of data as presented herein, and the measured loads adjusted as necessary to describe the apparent actual load.

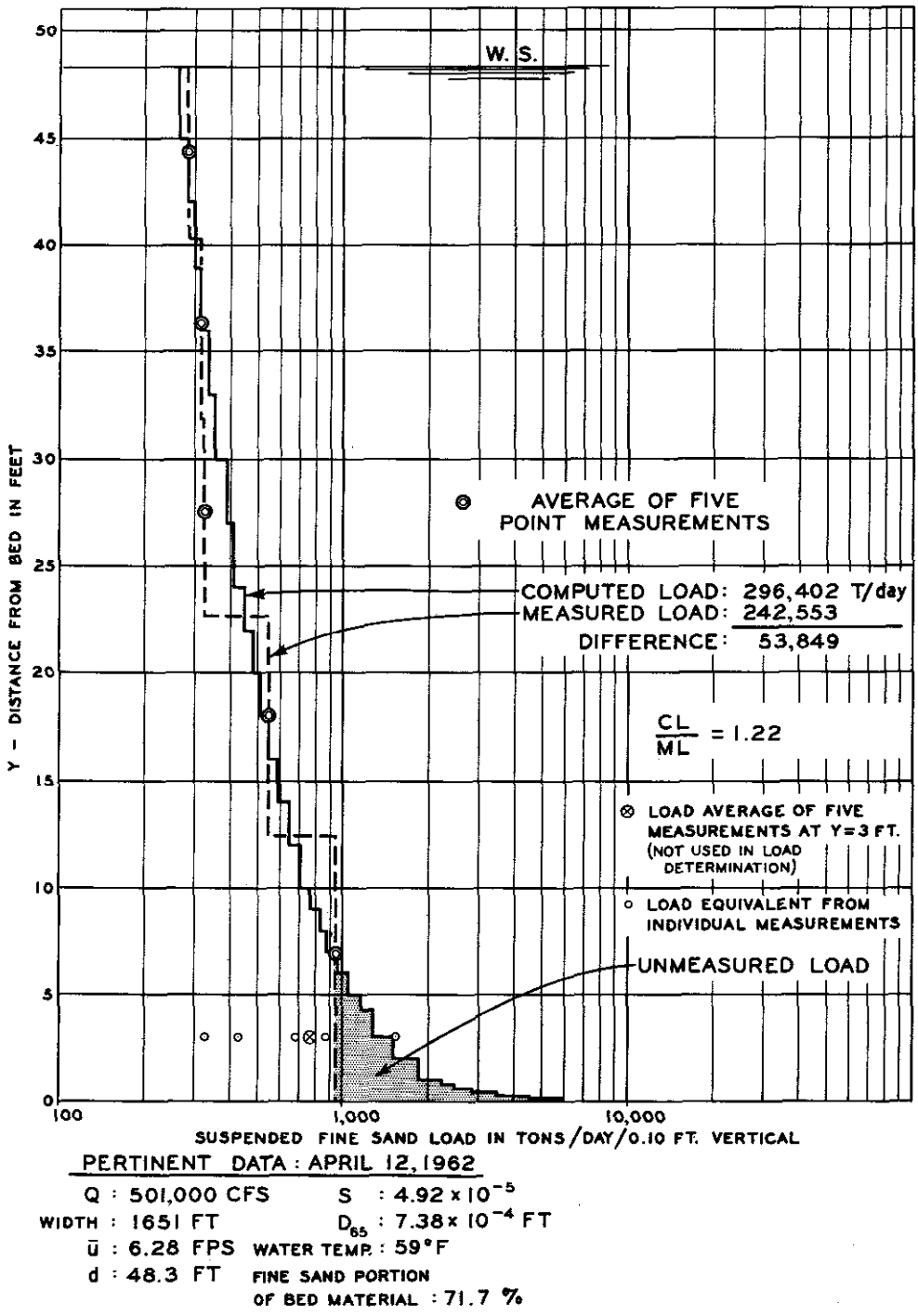
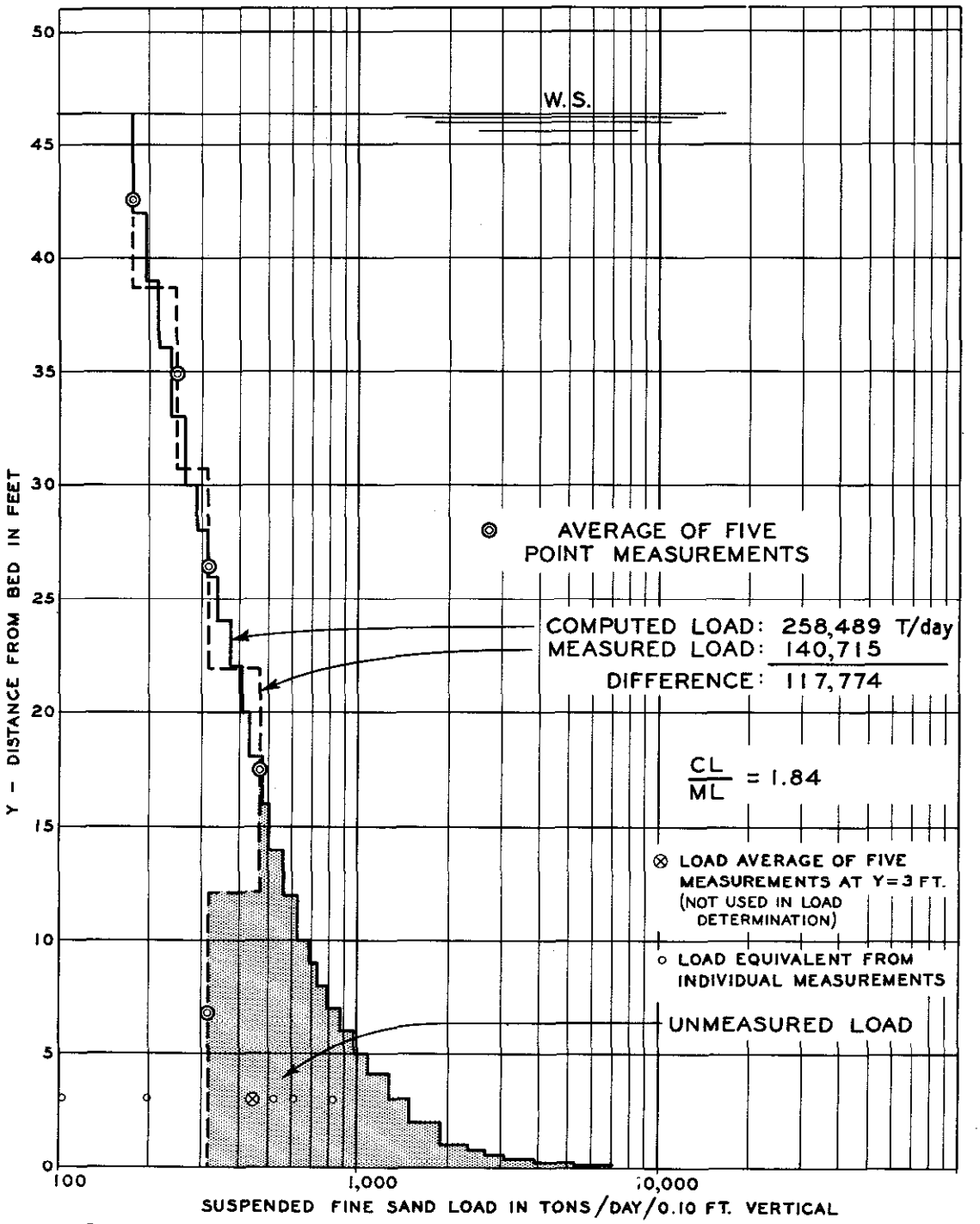


FIGURE 4. — Computed, measured, and unmeasured suspended fine sand load at Simmesport, Atchafalaya River, April 12, 1962.



PERTINENT DATA : JUNE 5, 1961

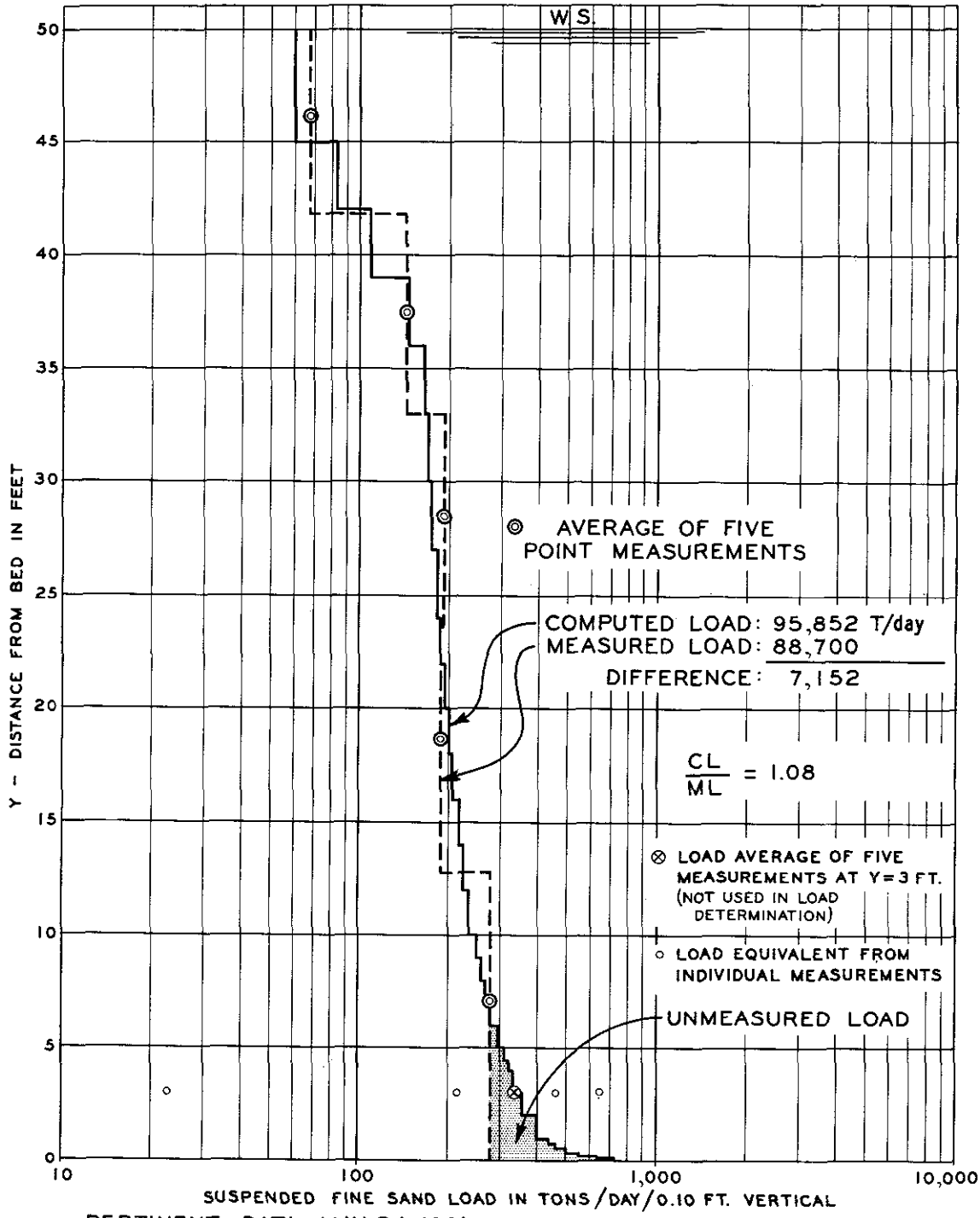
Q : 489,000 CFS S : 5.03×10^{-5}

WIDTH : 1587 FT D_{65} : 6.75×10^{-4} FT

$\bar{u}$: 6.66 FPS WATER TEMP. : 70°F

d : 46.3 FT FINE SAND PORTION OF BED MATERIAL : 81.6 %

FIGURE 5.—Computed, measured, and unmeasured suspended fine sand load at Simmesport, Atchafalaya River, June 5, 1961.



PERTINENT DATA : MAY 24, 1961

Q	: 444,000 FT	S	: 4.65×10^{-5}
WIDTH	: 1562 FT	D_{65}	: 6.98×10^{-4} FT
$\bar{u}$	: 5.68 FPS	WATER TEMP.	: 70°F
d	: 50.0 FT	FINE SAND PORTION OF BED MATERIAL	: 77.1 %

FIGURE 6. — Computed, measured, and unmeasured suspended fine sand load at Simmesport, Atchafalaya River, May 24, 1961.

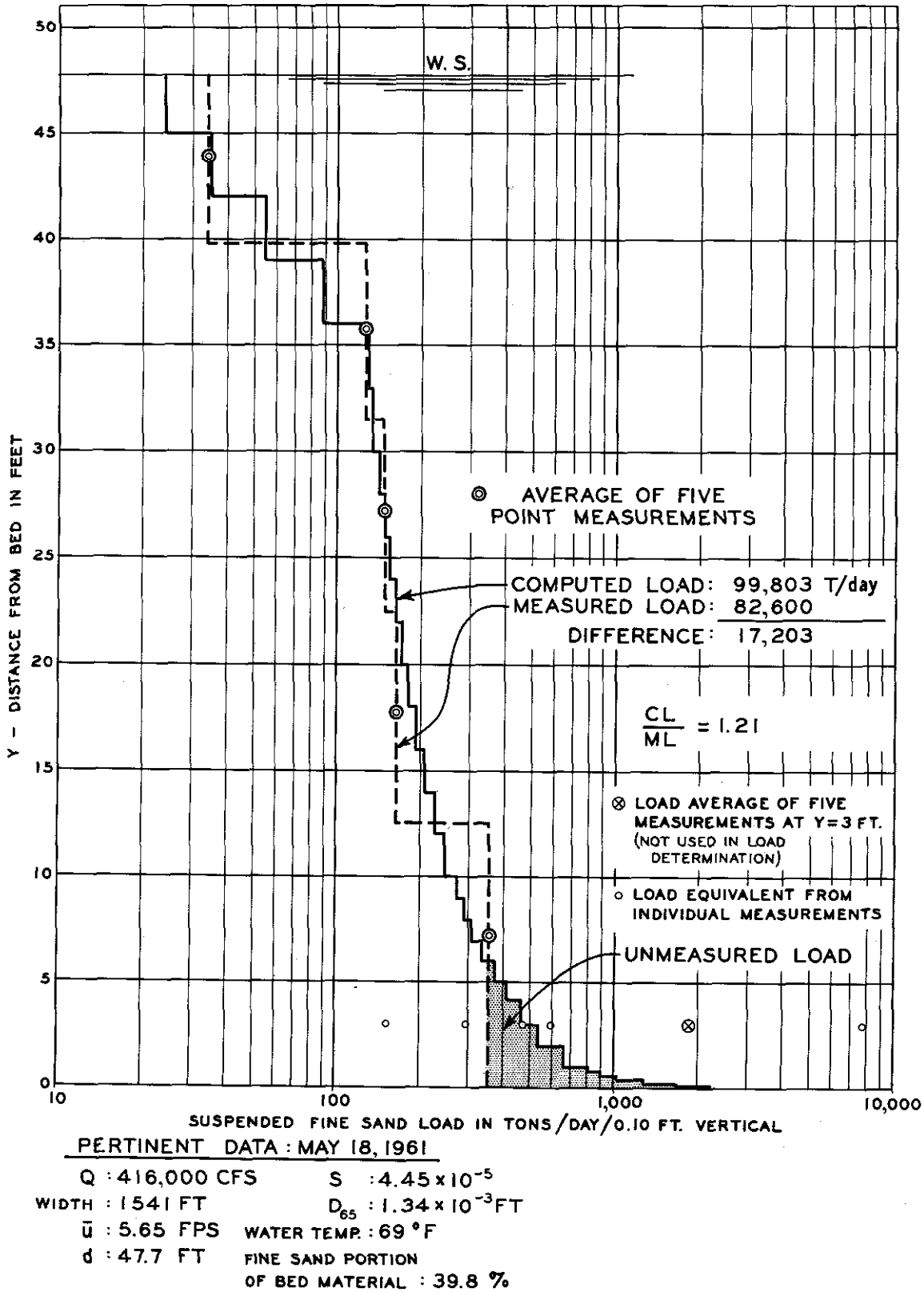
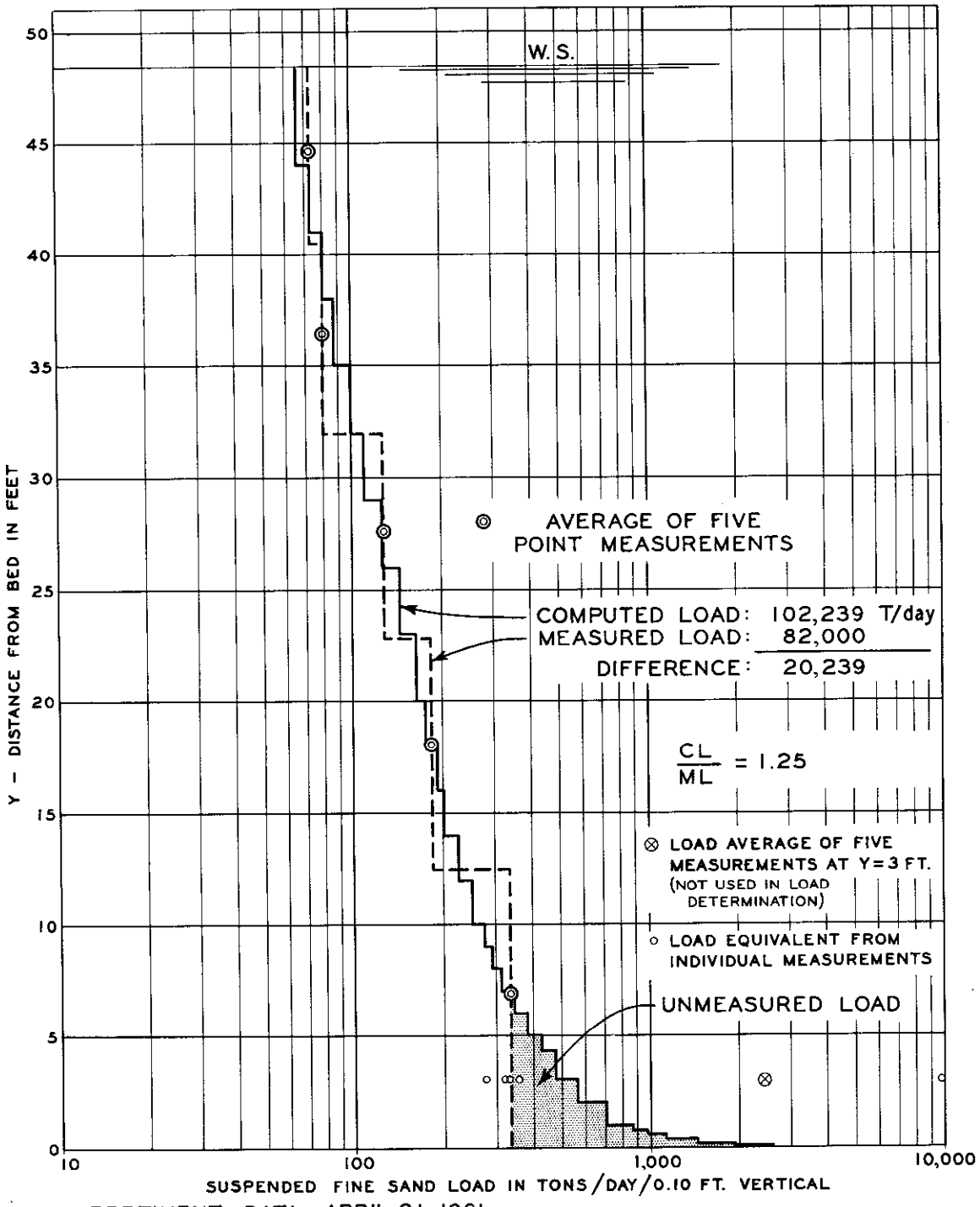


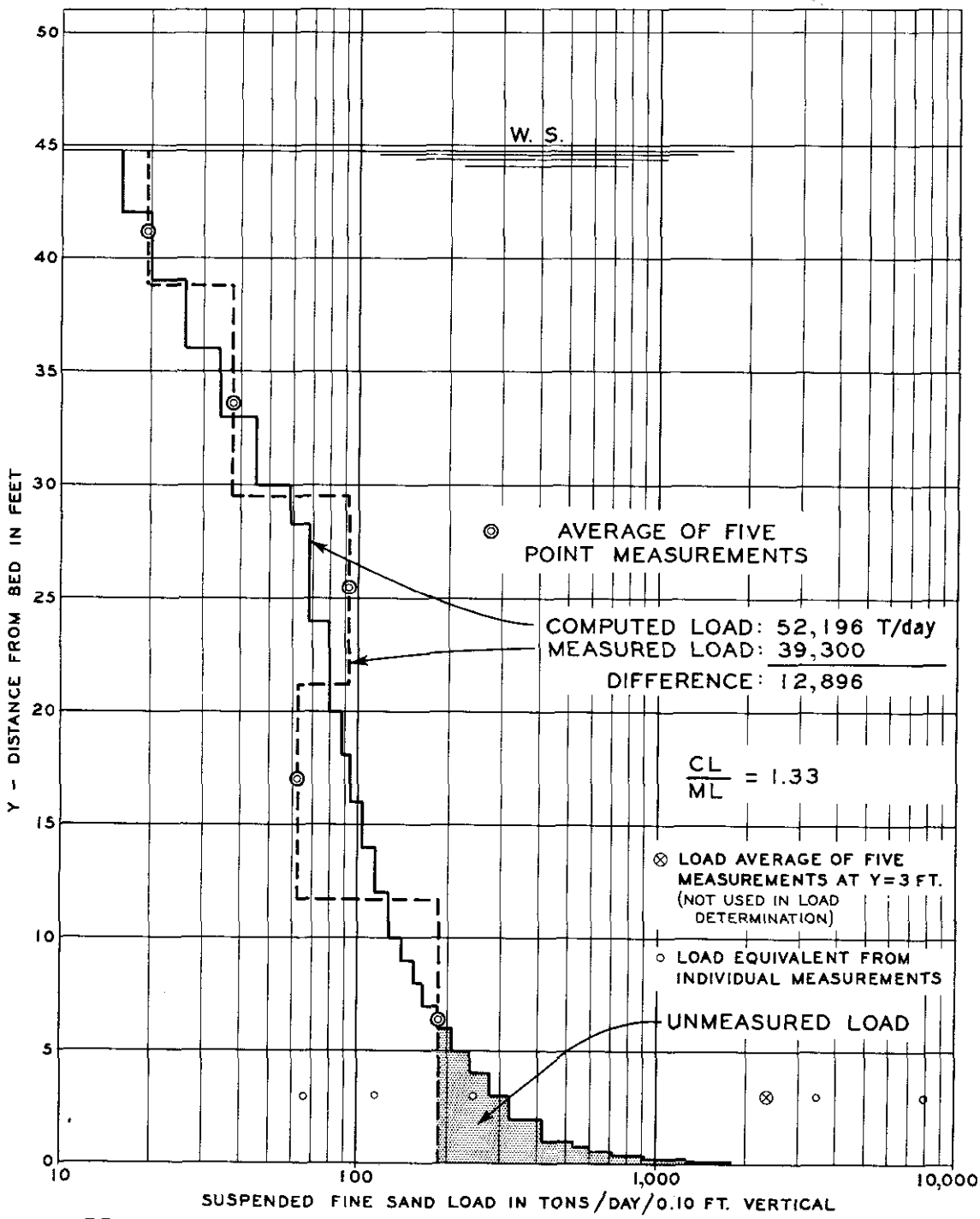
FIGURE 7.— Computed, measured, and unmeasured suspended fine sand load at Simmesport, Atchafalaya River, May 18, 1961.



PERTINENT DATA : APRIL 21, 1961

Q	: 411,000 CFS	S	: 4.45×10^{-5}
WIDTH	: 1538 FT	D_{65}	: 1.13×10^{-3} FT
$\bar{u}$	: 5.52 FPS	WATER TEMP.	: 61 °F
d	: 48.4 FT	FINE SAND PORTION OF BED MATERIAL	: 41.6 %

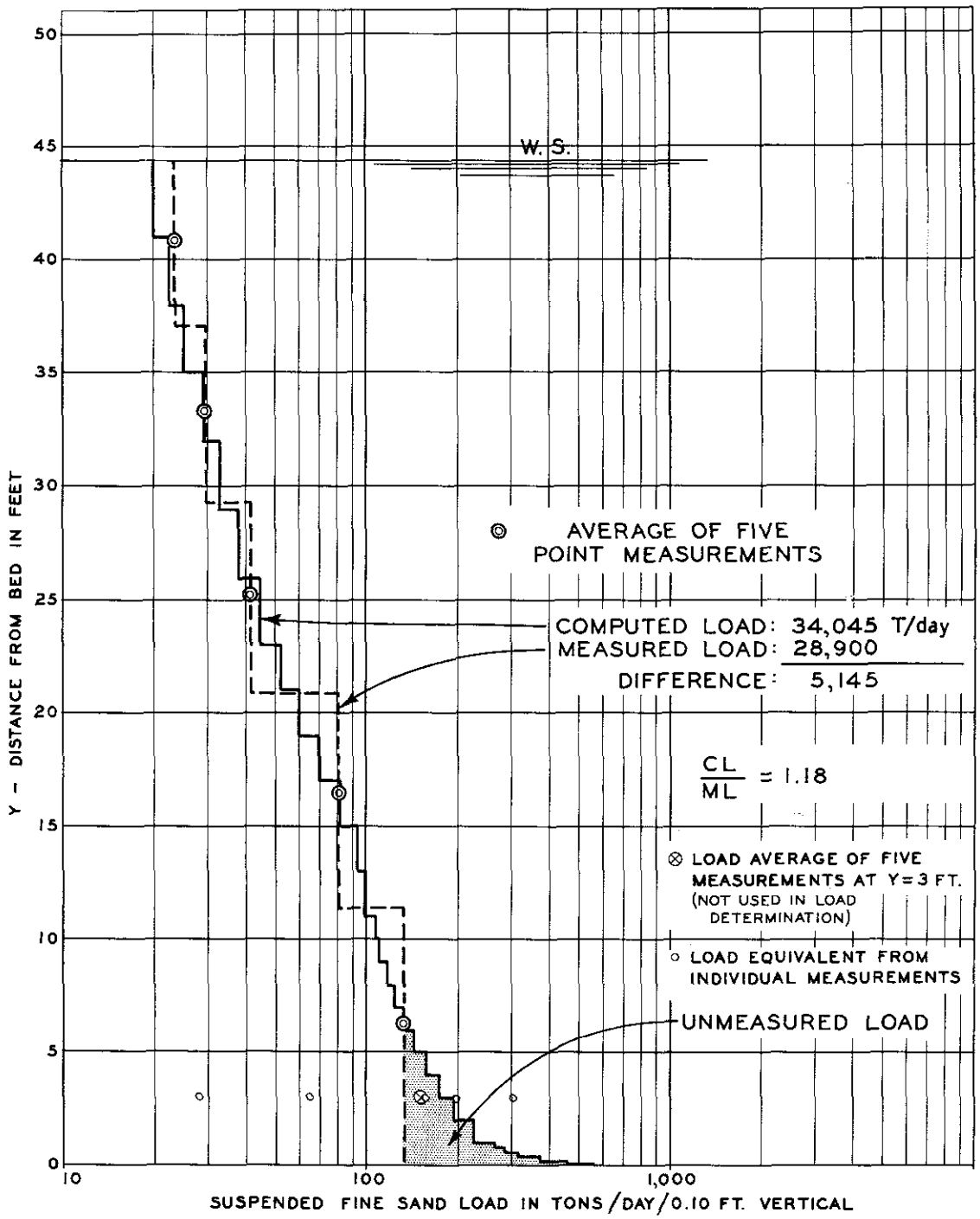
FIGURE 8. — Computed, measured, and unmeasured suspended fine sand load at Simmesport, Atchafalaya River, April 21, 1961.



PERTINENT DATA : JUNE 23, 1961

Q : 284,000 CFS
WIDTH : 1333 FT
$\bar{u}$: 4.77 FPS
d : 44.7 FT
S : 3.74×10^{-5}
D_{65} : 7.35×10^{-4} FT
WATER TEMP. : 78 °F
FINE SAND PORTION OF BED MATERIAL : 68.3 %

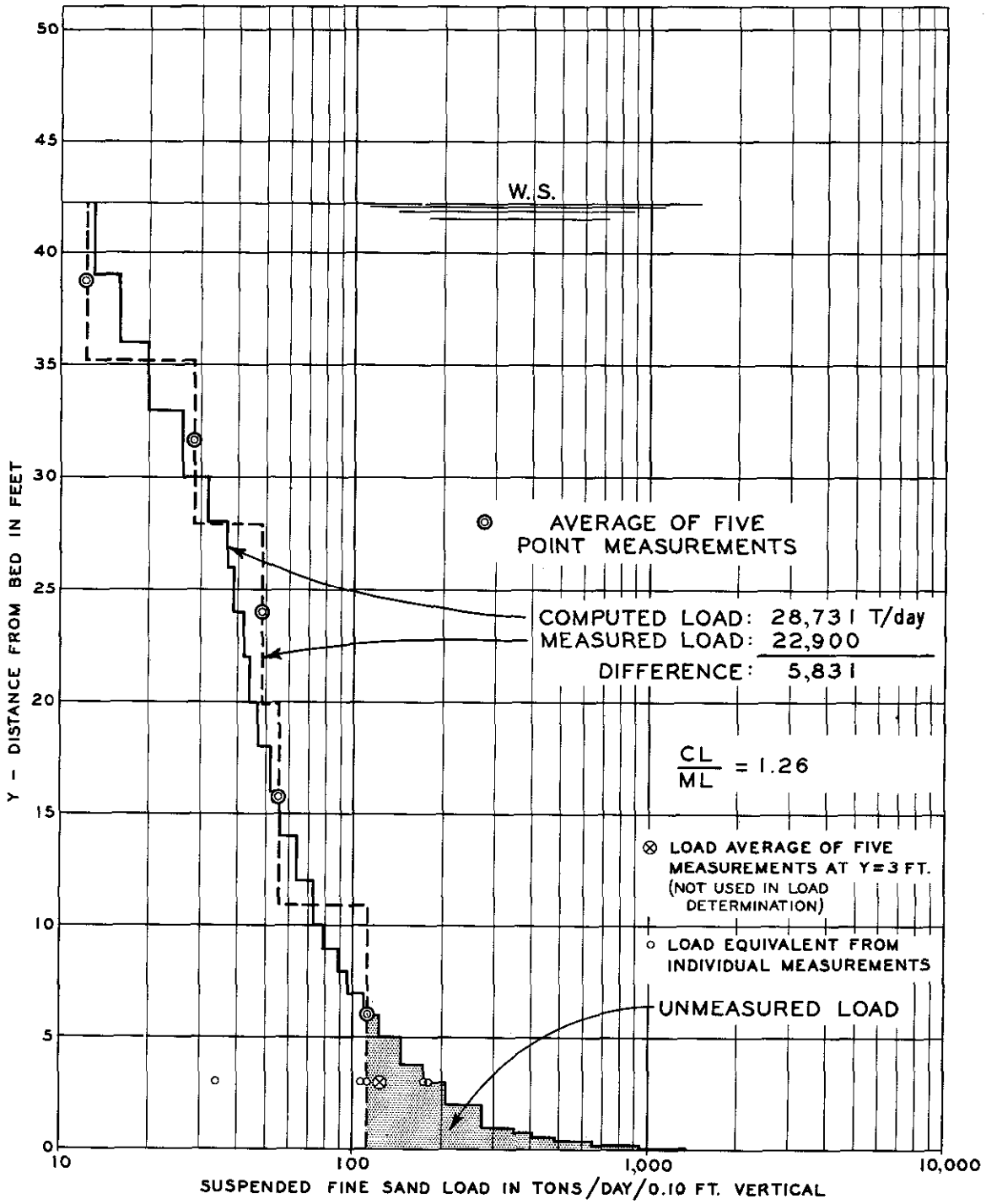
FIGURE 9.—Computed, measured, and unmeasured suspended fine sand load at Simmesport, Atchafalaya River, June 23, 1961.



PERTINENT DATA : NOV. 30, 1961

Q	: 254,000 CFS	S	: 3.74×10^{-5}
WIDTH	: 1312 FT	D_{65}	: 7.60×10^{-4} FT
$\bar{u}$	: 4.36 FPS	WATER TEMP.	: 52° F
d	: 44.4 FT	FINE SAND PORTION OF BED MATERIAL	: — %

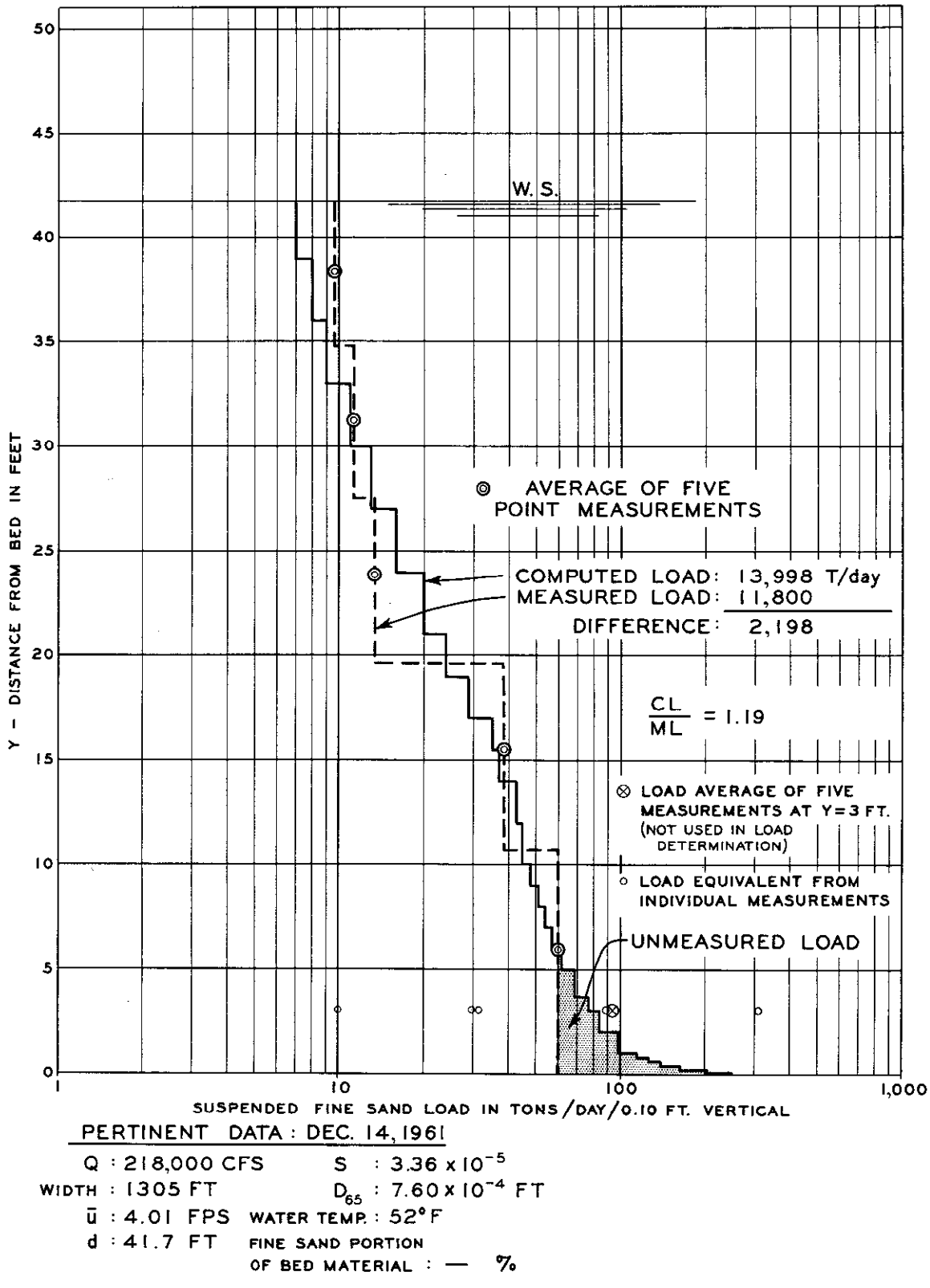
FIGURE 10. — Computed, measured, and unmeasured suspended fine sand load at Simmesport, Atchafalaya River, November 30, 1961.



PERTINENT DATA : JULY 14, 1960

Q	: 233,000 CFS	S	: 3.42×10^{-5}
WIDTH	: 1290 FT	D_{65}	: 7.57×10^{-4} FT
$\bar{u}$	: 4.28 FPS	WATER TEMP.	: 84 °F
d	: 42.2 FT	FINE SAND PORTION OF BED MATERIAL	: 40.5 %

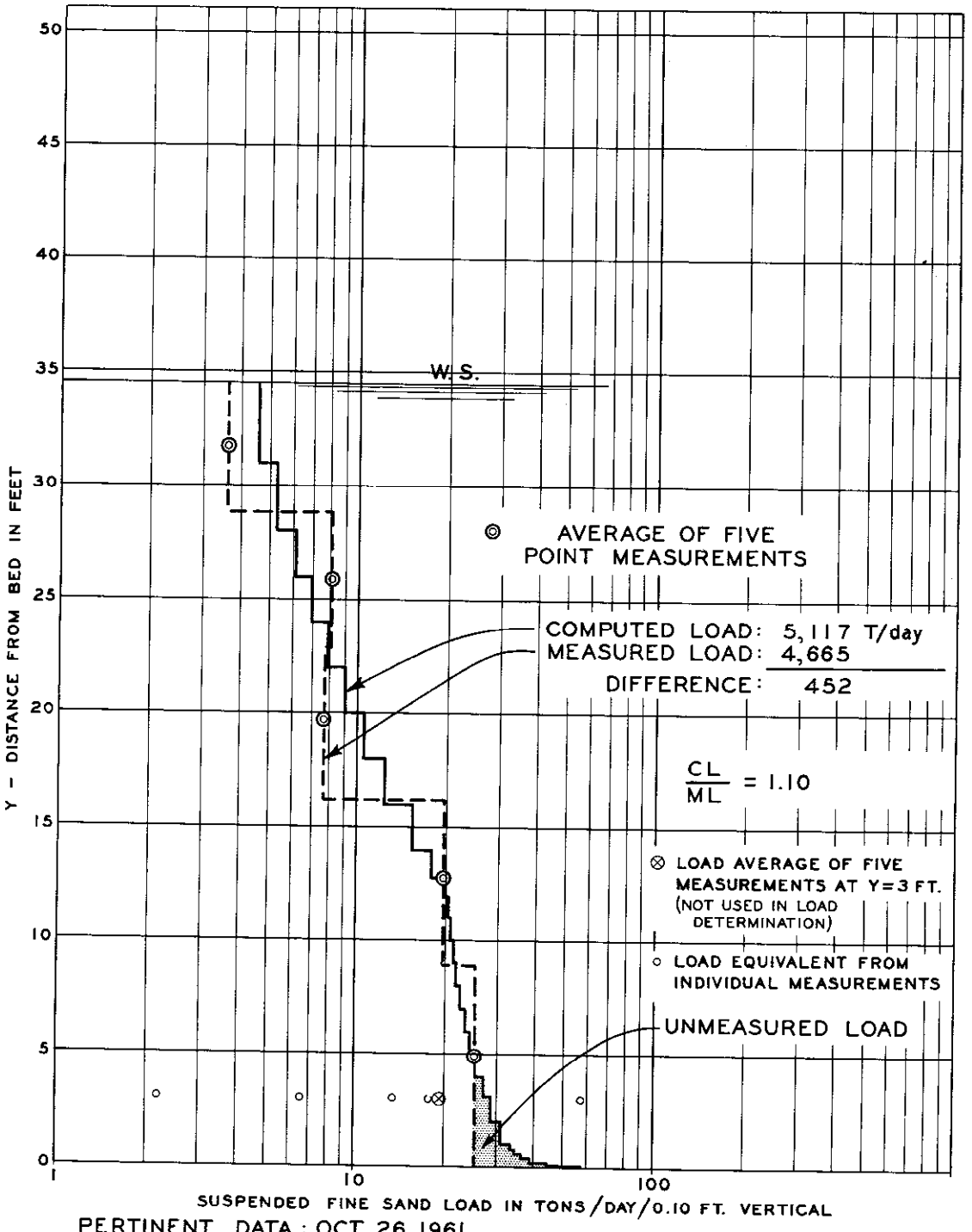
FIGURE 11.—Computed, measured, and unmeasured suspended fine sand load at Simmesport, Atchafalaya River, July 14, 1960.



PERTINENT DATA: DEC. 14, 1961

Q	: 218,000 CFS	S	: 3.36×10^{-5}
WIDTH	: 1305 FT	D_{65}	: 7.60×10^{-4} FT
$\bar{u}$	: 4.01 FPS	WATER TEMP.	: 52°F
d	: 41.7 FT	FINE SAND PORTION OF BED MATERIAL	: — %

FIGURE 12. — Computed, measured, and unmeasured suspended fine sand load at Simmesport, Atchafalaya River, December 14, 1961.



PERTINENT DATA : OCT. 26, 1961

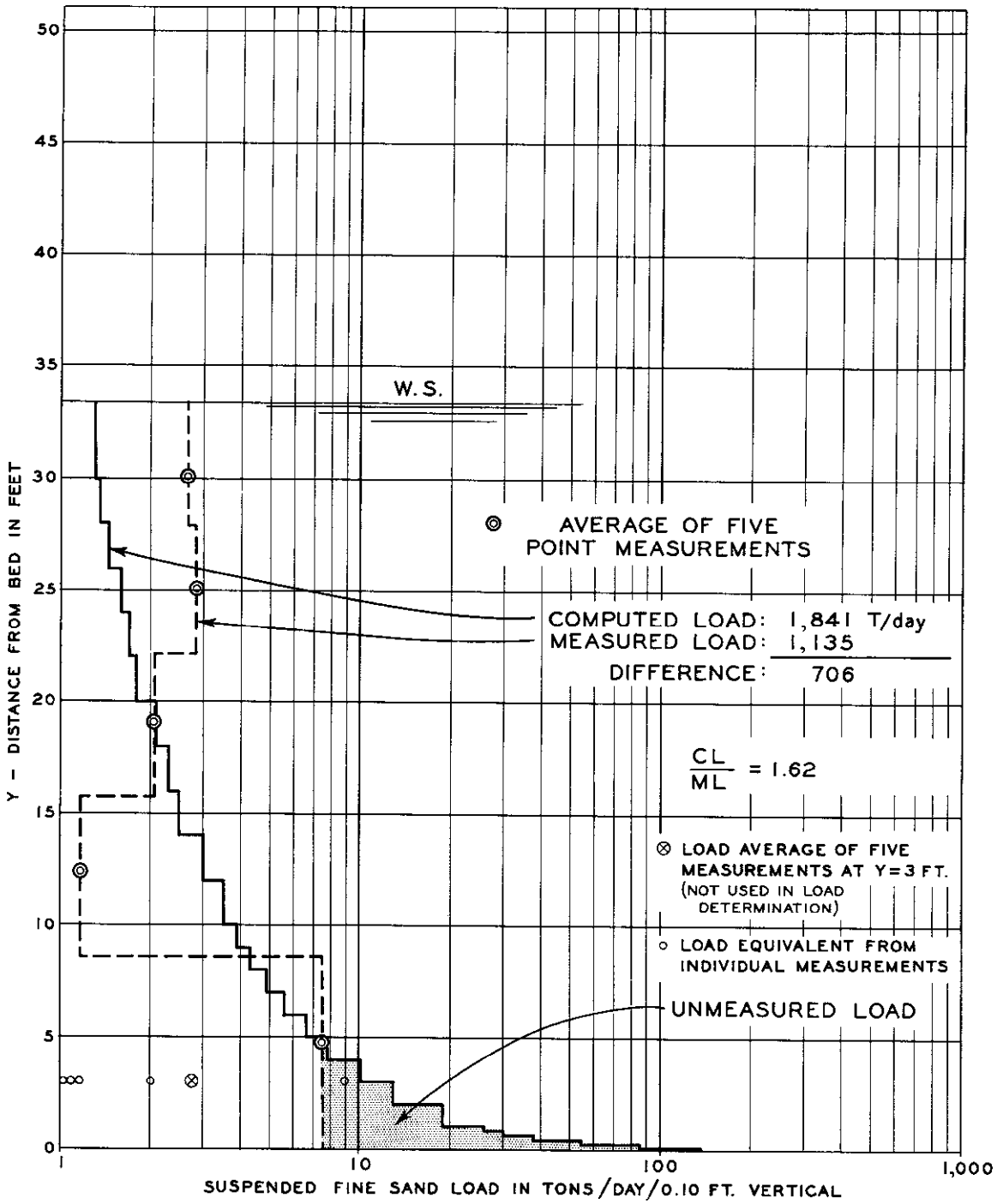
Q : 120,000 CFS S : 2.91×10^{-5}

WIDTH : 1224 FT D_{65} : 1.00×10^{-3} FT

$\bar{u}$: 2.84 FPS WATER TEMP. : 66°F

d : 34.5 FT FINE SAND PORTION OF BED MATERIAL : 20.8 %

FIGURE 13. — Computed, measured, and unmeasured suspended fine sand load at Simmesport, Atchafalaya River, October 26, 1961.



PERTINENT DATA : AUG. 11, 1960

Q	: 80,800 CFS	S	: 1.96×10^{-5}
WIDTH	: 1076 FT	D_{65}	: 7.96×10^{-4} FT
$\bar{u}$	: 2.25 FPS	WATER TEMP.	: 87°F
d	: 33.4 FT	FINE SAND PORTION OF BED MATERIAL	: 54.9 %

FIGURE 14. — Computed, measured, and unmeasured suspended fine sand load at Simmesport, Atchafalaya River, August 11, 1960.

CALCULATION OF SUSPENDED LOAD DISCHARGE FROM VELOCITY AND CONCENTRATION PARAMETERS

[Paper No. 29]

By NORMAN H. BROOKS, *professor of Civil Engineering, W. M. Keck Laboratory of Hydraulics and Water Resources, California Institute of Technology;* and *visiting professor (1962-1963), Hydrodynamics Laboratory, Massachusetts Institute of Technology*

Abstract

A method is presented for the integration of suspended load discharge quickly and easily from measured or predicted distributions of velocity and sediment concentration. The procedure is based on the assumptions that (1) the velocity distribution is logarithmic, and (2) the concentration follows the suspended load equation. Each of these two functions is represented by two parameters, essentially an intercept and a slope. A further parameter is the lower limit of integration.

In applying the formulas and graphs, the investigator has complete flexibility of choice of all five parameters, because the integration procedure is not interlocked with any other assumptions.

The effect of the lower limit of integration is discussed, and a logical procedure is recommended for its selection. Unmeasured suspended load may also be easily estimated.

Introduction

The integration of the suspended load discharge of a stream is basically a mathematical problem. In the technical literature (*3,4,5,6,7*), these simple mathematical operations have often been clouded with various assumptions and assumed constants, which are not pertinent to the integration but rather are part of a more comprehensive analysis (such as the so-called "Modified Einstein Procedure" for estimating total sediment transport from measured suspended sediment samples). Since the integration depends only on the concentration and velocity distributions over the depth, it is most convenient to express the result just in parameters defining these distributions. It is the purpose of this paper to describe only this integration procedure, and thus to allow the reader flexibility to use any measurements or assumptions he desires to define these profiles.

Analysis[1]

The total suspended load discharge per unit width, q_s, may be expressed for a two-dimensional channel as the integral:

$$q_s = \int_{y_o}^{d} cu\, dy \qquad (1)$$

where

$c = c(y)$ = concentration of suspended sediment (either in volume or weight per unit volume);

$u = u(y)$ = stream velocity (=downstream velocity of sediment particles);

y = distance above the bed;

y_o = lower limit of integration; and

d = total depth.

It is necessary to use a small but finite lower limit y_o instead of 0 because the sediment very close to the bed is transported as bedload, for which the usual equations for $c(y)$ and $u(y)$ do not apply, and the sediment grains no longer move at the same speed as the water.

The velocity distribution for two-dimensional flow may be expressed by the usual logarithmic velocity defect law in the form

$$u = u_{max} + \frac{u_*}{k} \ln \frac{y}{d} \qquad (2)$$

where

$u_{max} = u(d)$ = velocity at the surface ($y=d$),

$u_* = \sqrt{\tau_o/\rho}$ = shear velocity, and

k = von Karman's constant.

Integrating over the depth to determine the average velocity, $\bar{u}$,

$$\bar{u} = \frac{1}{d}\int_0^d \left(u_{max} + \frac{u_*}{k} \ln \frac{y}{d}\right) dy = u_{max} - \frac{u_*}{k} \qquad (3)$$

If equation 2 is rewritten with $\bar{u}$ instead of u_{max},

$$u = \bar{u} + \frac{u_*}{k}\left(1 + \ln \frac{y}{d}\right), \qquad (4)$$

or if logarithms to base 10 are used,

$$u = \bar{u} + \frac{u_*}{k} + \frac{2.30u_*}{k} \log_{10} \frac{y}{d} \qquad (5)$$

It may be noted that u_* is related to $\bar{u}$ and the friction factor (f) by the Darcy-Weisbach

[1] Most of the analyses and figure 2 were included in the author's Ph.D. dissertation. The helpful guidance and counsel of Vito A. Vanoni, his thesis adviser, is most gratefully acknowledged.

formula as applied to open channels,

$$u_*=\bar{u}\sqrt{f/8} \qquad (6)$$

If the velocity distribution is plotted on semilogarithmic graph paper, it is a straight line for which $\bar{u}$ is the intercept at $y=d/e=0.37d$, and $2.30u_*/k$ is the slope (units of velocity per cycle of y) as shown in figure 1,*a*. Any logarithmic velocity distribution may thus be completely described by these two parameters ($\bar{u}$ and u_*/k).

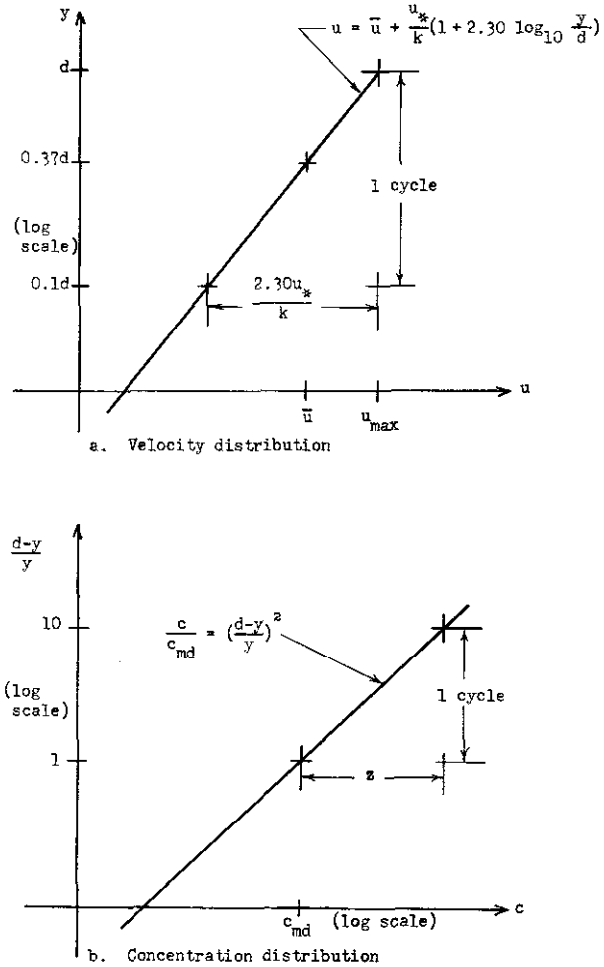


FIGURE 1. — Definition of intercepts and slopes: *a*, Velocity distribution; *b*, concentration distribution.

The concentration distribution, $c(y)$, for each sediment size fraction can usually be represented adequately by the suspended load equation (*10,11*)[2]

$$\frac{c}{c_a}=\left(\frac{a}{d-a}\,\frac{d-y}{y}\right)^z, \qquad (7)$$

where

$c_a=c(a)$ = concentration at reference level $y=a$, and

z = exponent.

Take the reference level, $a=d/2$, and equation 7 simplifies to

$$c=c_{md}\left(\frac{d-y}{y}\right)^z, \qquad (8)$$

where

$c_{md}=c(d/2)$ = middepth concentration.

If c is plotted versus $(d-y)/y$ on logarithmic graph paper, equation 8 is a straight line for which c_{md} is the intercept at $(d-y)/y=1$, and z is the slope of the straight line in cycles of c per cycle of $(d-y)/y$, as shown in figure 1,*b*. These two parameters alone are sufficient to define the distribution.

The exponent z in equation 7 or 8 has been shown theoretically by Ismail (*8*) (after nearly similar expressions given earlier by others) to be

$$z=\frac{w}{\beta k u_*}, \qquad (9)$$

where

w = settling velocity of the grains, and

$\beta=\epsilon_s/\epsilon_m$ = ratio of eddy diffusion coefficient for sediment to the eddy viscosity (≈ 1).

For a limited group of experiments with flat beds of 0.10 and 0.16 mm. sand, the writer[3] has found the value of β to be very close to 1.0 (0.93 to 1.08); however, in other laboratory investigations Vanoni (*11*) reports some larger deviations of observed exponents from the theoretical, based on $\beta=1$. In the field the failure to obtain good correlation between theoretical and measured z-values, such as reported by Colby and Hembree (*4*) and Hubbell and Matejka (*7*), is probably due to difficulties of accurate determination of the various factors required to compute the z-value by equation 9.

If equations 4 and 8 are substituted into equation 1, the suspended load discharge is

$$q_s=c_{md}\int_{y_o}^{d}\left(\frac{d-y}{y}\right)^z\left[\bar{u}+\frac{u_*}{k}\left(1+\ln\frac{y}{d}\right)\right]dy \qquad (10)$$

It is convenient to simplify equation 10 by the

[2] U.S. CORPS OF ENGINEERS. SEDIMENT TRANSPORTATION CHARACTERISTICS STUDY OF MISSOURI RIVER AT OMAHA, NEBRASKA. 1951. Unpublished reports.

[3] BROOKS, N. H. LABORATORY STUDIES OF THE MECHANICS OF STREAMS FLOWING OVER A MOVABLE BED OF FINE SAND. Ph.D. dissertation. On file, library of California Institute of Technology, Pasadena. 1954. (See pp. 188–194.)

change of variable $\eta = y/d$, giving:

$$q_s = c_{md}\,\bar{u}d\int_{\eta_o}^{1}\left(\frac{1-\eta}{\eta}\right)^z\left[1+\frac{u_*}{k\bar{u}}(1+\ln\eta)\right]d\eta, \quad (11)$$

where $\eta_o = y_o/d$ is now the lower limit. By writing $\bar{u}d = q$, the discharge per unit width, and separating into two integrals, equation 11 can be rewritten as:

$$\frac{q_s}{q\,c_{md}} = \left(1+\frac{u_*}{k\bar{u}}\right)\int_{\eta_o}^{1}\left(\frac{1-\eta}{\eta}\right)^z d\eta + \frac{u_*}{k\bar{u}}\int_{\eta_o}^{1}\left(\frac{1-\eta}{\eta}\right)^z \ln\eta\, d\eta \quad (12)$$

Einstein (*6*) discussed integration of suspended load, but his formulation was not as versatile as equation 12 above. Nevertheless the same integrals are involved and Einstein published tables of computed numerical values of them, which he has designated as

$$J_1(z, \eta_o) = \int_{\eta_o}^{1}\left(\frac{1-\eta}{\eta}\right)^z d\eta, \quad (13)$$

and

$$J_2(z, \eta_o) = -\int_{\eta_o}^{1}\left(\frac{1-\eta}{\eta}\right)^z \ln\eta\, d\eta \quad (14)$$

Convenient graphs giving the values of the integrals, J_1 and J_2, have also been published (*4,7*). Equation 12 may now be conveniently rewritten in terms of a transport function, T, as

$$\frac{q_s}{q\,c_{md}} = T\left(k\frac{\bar{u}}{u_*}, z, \eta_o\right), \quad (15a)$$

where T is defined by

$$T\left(k\frac{\bar{u}}{u_*}, z, \eta_o\right) = J_1(z, \eta_o) + \left[J_1(z, \eta_o) - J_2(z, \eta_o)\right]\frac{u_*}{k\bar{u}} \quad (15b)$$

The left-hand member of equation 15a can be expressed in other ways. If the suspended load discharge concentration, q_s/q, is denoted by $\bar{c}$, then

$$\frac{q_s}{q\,c_{md}} = \frac{\bar{c}}{c_{md}} \quad (16)$$

Furthermore, if it is preferred to use the intercept c_a instead of c_{md} as the reference concentration, then equation 16 becomes

$$\frac{q_s}{q\,c_{md}} = \frac{\bar{c}}{c_a}\left(\frac{d-a}{a}\right)^z \quad (17)$$

The right-hand member of equation 15b contains only the three parameters, $k(\bar{u}/u_*)$, z, and η_o, as indicated by functional notation of T. The first two may easily be derived from measured data as explained above (see figure 1,*b*). For instance,

$$k\frac{\bar{u}}{u_*} = \frac{\bar{u}}{u_*/k} = \frac{2.30\cdot(\text{intercept at } y=0.37d)}{\text{slope of velocity profile graph}} \quad (18)$$

Note that it is not necessary to know u_* and k separately if a velocity profile is available. As an alternative, one may use equation 6 and obtain

$$k\frac{\bar{u}}{u_*} = k\sqrt{8/f} \quad (19)$$

The exponent z is measured as the slope of the concentration graph, or computed (less reliably) from equation 9. The only further information needed is a value for η_o, the lower limit of integration. The selection of η_o is the subject of the following section.

Lane and Kalinske (*9*) have also given a solution for suspended load integration, but it is based on their exponential concentration distribution formula, derived by assuming a constant eddy diffusion coefficient over the depth. The writer [4] has also generalized their analysis in a manner similar to the above development, but it is not included here because the suspended load equation (equation 8) is preferred.

Choice of Lower Limit of Integration

The integrals J_1 and J_2 are convergent as $\eta_o \to 0$ only for $z<1$. Therefore, for values of z near 1 or larger than 1, the integrals change significantly with the choice of η_o. Not only is the divergence a problem, but physically the profile equations (equations 4 and 8) may not be accurate anyway for very small values of η. Thus, the limiting factor in the validity of the integration procedure is probably the accuracy of the equations representing velocity and concentration very close to the bed. Up to the present time no good way has been found to measure these profiles in the dense layer very close to the bed.

Although there are a number of possible ways to choose the lower limit $\eta_o = y_o/d$, only three will be discussed here. Since the particles can scarcely be suspended closer to the bed than two particle diameters ($2D$), Einstein (*6*) has suggested using $\eta_o = 2D/d$. For cases where the streambed is flat, this is probably as reasonable as any assumption. On the other hand for dune-covered beds for which the individual grain size is far less than the equivalent Nikuradse sand

[4] See footnote 3.

roughness for the surface, the extrapolated velocity profile will become negative near the lower limit suggested by Einstein. Thus, if $\eta_o = 2D/d$ is used, the suspended load integral may actually include some negative load contributions (cu) very close to the bed.

Another possibility for the choice of η_o is the level for which $u=0$, according to the logarithmic velocity profile. In this case from equation 4,

$$0=\bar{u}+\frac{u_*}{k}(1+\ln \eta_o).$$

Solving for η_o,

$$\eta_o = e^{-k\frac{\bar{u}}{u_*}-1}$$

Admittedly, the profile equation is not intended to be applicable down to $u=0$, but stopping the integration there certainly prevents possible unintentional negative contributions to the integral using $\eta_o=2D/d$. In fact, for the η_o giving $u=0$, the value of the transport integral $\int_{\eta_o}^{1} cu\, d\eta$ is a maximum; all other choices of η_o will mathematically result in smaller values of this integral as determined by equation 15.

A third possibility for η_o is to let it be the level at which the concentration, by extrapolation of the fitted suspended load equation, becomes equal to the concentration of the bed or of a size fraction in the bed, c_b. For this case, equation 8 gives

$$c_b = c_{md}\left(\frac{1-\eta_o}{\eta_o}\right)^z$$

and since $\eta_o << 1$,

$$\eta_o = \left(\frac{c_{md}}{c_b}\right)^{\frac{1}{z}}$$

It would certainly be physically unreasonable to use a smaller lower limit than this, as is sometimes the case when $\eta_o=2D/d$ or $u(\eta_o)=0$ is used, for then the integration would include values of $c > c_b$.

The three possibilities outlined above may now be summarized as follows:

(a) $\eta_o = \frac{2D}{d}$ (Einstein bed layer) (20)

(b) $\eta_o = e^{-k\frac{\bar{u}}{u_*}-1}$ $u(\eta_o)=0$ (21)

(c) $\eta_o = \left(\frac{c_{md}}{c_b}\right)^{\frac{1}{z}}$ $c(\eta_o)=c_b$ (22)

Perhaps the most reasonable choice of the three is the *largest* η_o, because it has been shown that it is physically unreasonable to go lower than any one of the three. Further experimental investigation is needed to verify whether this is a satisfactory approach, at least for approximate solutions.

Because of its simplicity, choice (b) warrants special attention. As equation 15 stands, $q_s/q\ c_{md}$ is a function of the three parameters $k\frac{\bar{u}}{u_*}$, z, and η_o; but since η_o is a function of $k\frac{\bar{u}}{u_*}$ by equation 21, η_o may be eliminated as an explicit parameter. The dimensionless transport given by equation 15 depends then only on z and $k\frac{\bar{u}}{u_*}$ and may be represented on a single chart.

Figure 2 was constructed by the writer[5] on this basis from the values of the integrals given by Einstein (*6*).

For a given z in figure 2, the ratio $q_s/q\ c_{md}$ increases when $k\frac{\bar{u}}{u_*}$ (or $k\sqrt{8/f}$) increases. This relation is partly the result of changing the shape of the velocity profile and partly the effect of reducing the lower limit η_o. The latter is especially important for the larger z values, for which the concentration profile is very steep and most of the suspended load moves near the bed; in this case, a small change in the η_o value can change the value of the q_s integral a large amount. In fact, since the value of η_o based on $u(\eta_o)=0$ may not be the best value anyway, considerable error can be expected when $q_s/q\ c_{md}$ gets very large on the chart. Consequently, the curves have not been extended for $q_s/q\ c_{md} > 50$, or for $z > 2$.

It may be observed from figure 2 that for $z < 0.3$, $q_s/q\ c_{md}$ may be considered equal to 1.0 for all practical purposes. The slight drop below 1.0 for the curves for the smaller values of $k\frac{\bar{u}}{u_*}$ at small z is real and may be explained as follows. Although concentrations are larger toward the bed, velocities are greater toward the surface. Usually the nonuniformity of concentration is much more pronounced than for the velocity, resulting in a weighted average concentration $\bar{c}$ greater than the middepth concentration c_{md}. However, the situation is reversed for *small* z (relatively uniform c) and small $k\frac{\bar{u}}{u_*}$ (high roughness, steep velocity profile), and the average $\bar{c}$ does actually become less than c_{md}.

Application of Results

Example of Use of Figure 2

The use of figure 2 may be illustrated by the

[5] See footnote 3.

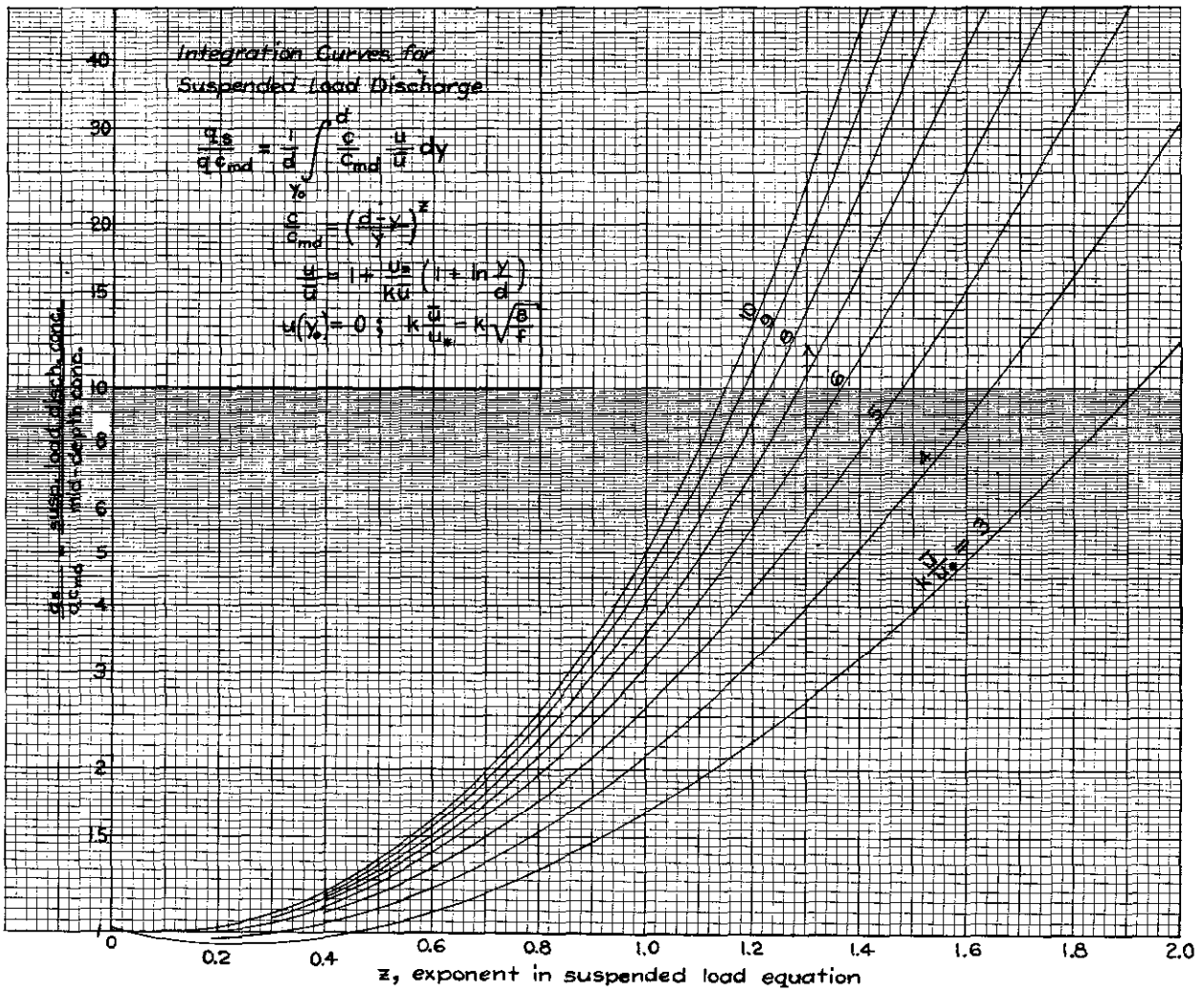


FIGURE 2. — Integration curves for suspended load discharge by equations 15 and 21.

following example. Suppose that from a measured velocity profile for a wide stream 3 ft. deep it is found that $\bar{u}=u(0.37d)=3.5$ ft./sec. The slope of the velocity profile=1.25 ft./sec. per cycle$=\frac{2.3u_*}{k}$; therefore $\frac{u_*}{k}=0.54$. In addition, when c is plotted on logarithmic paper vs. $(d-y)/y$, the slope of the line of best fit is $z=1.1$, and the line intersects $(d-y)/y=1$ at $c_{md}=0.05$ g./l.=50 p.p.m. Therefore,

$$k\frac{\bar{u}}{u_*}=\frac{3.5}{0.54}=6.5,$$

$$z=1.1$$

By figure 2,

$$\frac{q_s}{q\,c_{md}}=4.5,$$

and hence,

$$\frac{q_s}{q}=4.5\,c_{md}=4.5\,(50)=225 \text{ p.p.m.}$$

Since the discharge per unit width is

$$q=\bar{u}\,d=(3.5)\ (3)=10.5 \text{ c.f.s./ft.},$$

the suspended load discharge per unit width is

$$q_s=\frac{225}{10^6}\times 10.5\times 62.4=0.15 \text{ lb./ft.-sec.}$$

Laboratory Experiments

The writer has applied figure 2 to laboratory experiments in a 10.5-inch wide flume, and compared the results with measured total sediment loads. Two different bed sands were used, with mean sieve diameters of 0.145 mm. and 0.088 mm., respectively. Only flat bed cases were used because no velocity and concentration profiles were measured for the dune bed cases.

A detailed account of the experiments, including velocity and concentration profiles, is given

in the writer's Ph. D. thesis,[6] and a briefer description including tabulation of the basic data (but not the profiles) is available in reference (*2*).

A rigorous comparison of experimental results with the calculated suspended load discharge is not possible, because the measured sediment discharge concentration, $\bar{C}$, includes also the bedload and because the centerline profiles of concentration and velocity on which the integration is based are not truly representative of the whole cross section. Nevertheless, with the high transportation rates associated with the runs for which profiles were obtained, the bedload is known to be small. Furthermore, it has been observed that the concentration does not vary substantially going across the channel at any given level. Since most of the load is carried close to the bed anyway, the distortion of the velocity distribution by the walls at the upper levels does not introduce a very large error. Consequently, the order of magnitude of the measured concentration, $\bar{C}$, should be the same as the calculated ratio of suspended load discharge to water discharge, q_s/q.

When any of the integration procedures are used, it is important to have the suspended load equation fit the observed concentrations well only where the concentrations are large. If the curve deviates moderate amounts when the concentrations are relatively small, as was often the case in the plotted concentration profiles, the error introduced in the total transportation rate is still small.

The basic experimental data required for the calculation of the integrals are listed in table 1. As indicated, the parameter $k\frac{\bar{u}}{u_*}$ was determined from the intercept and slope of velocity distribution (on the centerline of the flume). The concentration distribution yields c_{md} (intercept) and z (slope). The mean discharge concentration was measured in a vertical part of the return flow circuit at a point where all the load was suspended.

The three values of η_o, which are listed in table 1, were calculated by equations 20 to 22 and represent various possibilities for the lower limit of integration. The values of the dimensionless ratio, $q_s/q\ c_{md}$, listed in table 1, were calculated from $k\frac{\bar{u}}{u_*}$, z, and η_o by equation 15 and appropriate charts in references (*4*), (*6*), or (*7*), and may be compared with the measured ratio C/c_{md}. (For column 2, the values of $q_s/q\ c_{md}$ were obtained directly from figure 2, except for run 7.)

From an inspection of the table, it is clear that the agreement between the values is not perfect, but qualitatively quite good. When the

[6] See footnote 3.

TABLE 1.—*Integration for suspended sediment discharge for flume experiments* [1]

BASIC DATA

Run No.	Mean sieve diameter of bed (D)	Mean depth (d)	Velocity distribution: Mean = u (0.37d) ($\bar{u}$)	Velocity distribution: Slope $\left(2.30\frac{u_*}{k}\right)$	Velocity distribution: $k\frac{\bar{u}}{u_*}$	Concentration dist.: Mid-depth conc. (c_{md})	Concentration dist.: Slope (z)	Measured mean discharge concentration ($\bar{C}$)
	Mm.	*Feet*	*Ft./sec.*	*Ft./sec.*		*G./l.*		*G./l.*
3	0.145	0.243	2.32	1.07	4.98	0.20	1.36	1.95
4	.145	.236	2.43	1.05	5.32	.54	1.20	2.45
6	.145	.195	2.35	.99	5.45	.29	1.49	2.45
7	.145	.243	2.38	.90	6.07	.092	1.78	2.15
21	.088	.236	2.37	1.08	5.03	3.0	.76	4.85
29	.088	.280	2.36	1.07	5.05	1.9	.83	3.45

RESULTS OF INTEGRATION

Run No.	z	$k\frac{\bar{u}}{u_*}$	Values of lower limit $\eta_o=\frac{y_o}{d}$: (1) $\eta_o=\frac{2D}{d}$	(2) $u(\eta_o)=0$	(3) $c(\eta_o)=c_b$ [2]	Values of the integral, (q_s/qc_{md}), calculated for η_o as listed: (1)	(2) [3]	(3)	$\bar{C}/c_{md}$ (measured total load)
3	1.36	4.98	0.0039	0.00255	0.00145	6.6	6.8	6.6	9.7
4	1.20	5.32	.0040	.0018	.0014	4.45	4.7	4.7	4.55
6	1.49	5.45	.0049	.0016	.00325	10.5	13	11.5	8.45
7	1.78	6.07	.0039	.00084	.0044	40	65	39	23.5
21	0.76	5.03	.0025	.0024	.00029	1.62	1.65	1.57	1.62
29	0.83	5.05	.00205	.00235	.00033	1.83	1.85	1.75	1.82

[1] See footnote 3.
[2] c_b=1,450 gr./l.
[3] Values read directly from figure 2, except for run 7.

value of the exponent z is small (less than 1) and hence $q_s/q\ c_{md}$ is small, the agreement of all the values is excellent; for the theoretical calculation it appears that any one of these choices of the lower limit would be satisfactory.

However, for the higher z values (greater than 1.3 or 1.4), the calculated $q_s/q\ c_{md}$ is significantly larger than the experimentally determined concentration ratio, $\bar{C}/c_{md}$. The discrepancy results from the fact that when z is large, a major part of the load travels very close to the bed where the extrapolated concentration and velocity profiles may not be accurate. None of the choices of the lower limit gives a good value of $q_s/q\ c_{md}$ when this ratio is large.

A further example of the application of figure 2 has been included as table 2 for the concentration profiles obtained for individual sieve fractions for run 29. The exponent z ranges from 0.53 to 1.27, and the values obtained from figure 2 compare favorably with the measured ratio $\bar{C}/c_{md}$ for each sieve fraction.

TABLE 2.—*Integration of suspended sediment for individual sieve fractions for run 29, when* $k\frac{\bar{u}}{u_*}=5.05$

Item	Tyler sieve fractions				Overall (data from table 1)
	150	170	200	250	
z, exponent	1.27	1.03	0.93	0.76	0.83
$q_s/q\ c_{md}$, by fig. 2	5.3	2.82	2.26	1.65	1.85
$\bar{C}/c_{md}$, measured	3.92	2.66	1.92	1.43	1.82

For all the present runs where the sand bed was smooth, the level $\eta_o=2D/d$ (table 1) is higher than the level for which $u(\eta_o)=0$, and thus it might be reasoned that the former would be a better choice. However, when the bed becomes covered with dunes, the friction factor increases and the situation is reversed. For example, assume the typical values $f_b=0.08$ and $k=0.30$; by equation 19,

$$k\frac{\bar{u}}{u_*}=k\sqrt{\frac{8}{f_b}}=3.0,$$

and by equation 21,

$$\eta_o=e^{-3.0\ -1}=0.018$$

Now η_o is considerably more than $2D/d$ for any of the runs, and the use of $\eta_o=2D/d$ in a case like this would include negative values of the integrand in the load integral. Since Barton and Lin (*1*) have shown in the laboratory that the suspended load equation can be applied reasonably well to flows over a dune-covered bed, then the use of η_o for which $u(\eta_o)=0$ (and the chart in fig. 2) is recommended in place of the lower limit $\eta_o=2D/d$ for such cases.

One of the advantages of the diagram in figure 2 is the avoidance of some of the computational error accumulated in using the charts for J_1 and J_2, and equation 15. First, in reading the charts, which are on logarithmic graph paper with 3-inch cycles in reference (*4*) or (*7*), interpolation is usually required, making it difficult to obtain better than 2-place accuracy. Then making the calculation according to equation 15 usually involves taking the difference between numbers of the same order of magnitude, increasing the error further.

Estimation of Unmeasured Suspended Load

In sampling natural streams for determination of the suspended load discharge, a depth integrating sampler is frequently used. Since it samples the flow in proportion to the velocity at each level as it slowly traverses the depth, the concentration of sediment in the integrated sample is the weighted average concentration expressed as

$$c(y_1)=\frac{\int_{y_1}^{d} cu\,dy}{\int_{y_1}^{d} u\,dy}, \tag{23}$$

where the value of $\bar{c}$ obtained is a function of y_1, the lower limit of the traverse. The numerator is given by equation 10 with y_1 replacing y_o, while the denominator can be considered the partial discharge $q(y_1)$ which flows above $y=y_1$. From equation 4, and changing to the dimensionless elevation η, the partial discharge is

$$q(y_1)=\int_{y_1}^{d} u\,dy=\bar{u}\,d\int_{\eta_1}^{1}\left[1+\frac{u_*}{k\bar{u}}(1+\ln\eta)\right]d\eta \tag{24}$$

Integrating,

$$q(\eta_1)=\bar{u}d(1-\eta_1-\frac{u_*}{k\bar{u}}\eta_1\ln\eta_1) \tag{25}$$

Equation 23 may be written as

$$\bar{c}(\eta_1)=\frac{q_s(\eta_1)}{q(\eta_1)}=\frac{q_s(\eta_1)/q}{q(\eta_1)/q} \tag{26}$$

and substitution of equations 15 and 25 yields

$$\frac{\bar{c}(\eta_1)}{c_{md}}=\frac{T(k\frac{\bar{u}}{u_*},z,\eta_1)}{1-\eta_1-\frac{u_*}{k\bar{u}}\eta_1\ln\eta_1} \tag{27}$$

For the total suspended load transport with lower limit η_o, equation 15 is

$$\frac{\bar{c}(\eta_o)}{c_{md}}=T(k\frac{\bar{u}}{u_*},z,\eta_o) \tag{28}$$

Dividing equation 28 by equation 27 gives

$$\frac{\bar{c}(\eta_o)}{\bar{c}(\eta_1)} = (1-\eta_1-\frac{u_*}{k\bar{u}}\eta_1 \ln \eta_1) \frac{T(k\frac{\bar{u}}{u_*},z,\eta_o)}{T(k\frac{\bar{u}}{u_*},z,\eta_1)} \quad (29)$$

If the parameters $k\frac{\bar{u}}{u_*}$, z, η_o and η_1 are known or can be estimated, then the ratio of the desired concentration $\bar{c}(\eta_o)$ (for the *total* suspended load) to measured concentration $\bar{c}(\eta_1)$ can be readily determined by equation 29, where T is defined in equation 15b.

The unmeasured suspended load transport is usually defined as

$$\text{Unmeasured } q_s = q[\bar{c}(\eta_o)-\bar{c}(\eta_1)] = q\bar{c}(\eta_1)[\frac{\bar{c}(\eta_o)}{\bar{c}(\eta_1)}-1] \quad (30)$$

where q is the full discharge and the ratio is given in equation 29. Since the quantity $qc(\eta_1)$ is called the measured suspended load transport, the term in brackets in equation 30 is the ratio of unmeasured to measured suspended load discharge.

The previous example may be extended to illustrate the use of equation 29. Suppose that z is estimated to be 1.1 and that a depth integrated sample taken down to $y=0.10d$ has a mean concentration of 0.12 g./l. The mid-depth concentration c_{md} is now presumed unknown. Evaluation of equation 29 gives

$$\frac{\bar{c}(\eta_o)}{\bar{c}(\eta_1)} = (1-0.10-\frac{2.30}{6.5}0.10\log_{10}0.10) = \frac{T(6.5,1.1,\eta_o)}{T(6.5,1.1,0.10)}.$$

If figure 2 (with η_o thus implied to be $u(\eta_o)=0$, or $\eta_o=0.00055$) is used,

$$T(6.5,1.1,\eta_o)=4.65.$$

With the use of the charts for J_1 and J_2 in (*4*) or (*7*) and the definition of T in equation 15b,

$$T(6.5,1.1,0.10)=J_1(1.1,0.10)+\frac{1}{6.5}[J_1(1.1,0.10)-J_2(1.1,0.10)] = 1.56+\frac{1}{6.5}(1.56-2.27) = 1.45$$

Then

$$\frac{\bar{c}(\eta_o)}{\bar{c}(\eta_1)}=(0.935)\frac{4.65}{1.45}=3.00,$$

and finally

$$\bar{c}(\eta_o)=\frac{q_s}{q}=3.00(0.12)=0.36 \text{ g./l.}$$

In this case the unmeasured suspended load discharge would be 2.00 times the measured load discharge. Above the level $y=0.10d$, the water discharge is 93.5 percent of the whole, and the suspended sediment discharge is $\frac{1.45}{4.65}=31$ percent.

Equation 25 may also be used to explain the anomology in figure 2 where the curve for $k\frac{\bar{u}}{u_*}=3$ rises slightly above the ordinate 1.00 as $z\to 0$. For $z=0$, $c=$constant, and equation 11 reduces to equation 24, that is

$$\frac{q_s}{\bar{u}d\,c_{md}}=\int_{\eta_o}^{1}[1+\frac{u_*}{k\bar{u}}(1+\ln\eta)]d\eta=\frac{q(\eta_o)}{\bar{u}d} = 1-\eta_o-\frac{u_*}{k\bar{u}}\eta_o\ln\eta_o \quad (31)$$

For $k\frac{\bar{u}}{u_*}=3$, equation 21 gives

$$\eta_o=e^{-3-1}=0.0183$$

and by equation 31,

$$\frac{q_s}{\bar{u}d\,c_{md}}=1.006,$$

or 0.6 percent in error. This discrepancy is due to the fact that the average velocity is based on integration from zero (equation 3) rather than the finite lower limit η_o. Since the error is so small and becomes even smaller for larger values of $k\frac{\bar{u}}{u_*}$, no adjustment was made in figure 2.

Summary

Equation 15 is recommended for integration of the suspended load transport. Numerical calculations may be made with the aid of published graphs of J_1 and J_2 (*4,7*) for any choice of η_o, the lower limit of integration. The selection of the lower limit might reasonably be selected as the largest of the three choices given by equations 20, 21, and 22. Use of equation 21 for η_o is mathematically the simplest, because the complete integration equation (equation 15) can then be represented in one chart (fig. 2). A numerical example and flume data illustrate the application of the equations and figure 2.

From the integration equation the suspended load transport measured by a depth integrating sampler can be corrected for the suspended load in the unsampled zone below the lowest point reached by the sampler during its traverse.

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EFFECT OF TEMPERATURE ON RIVERBED CONFIGURATION: ITS POSSIBLE STAGE-DISCHARGE IMPLICATIONS

[Paper No. 31]

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On the basis of field and other studies, it is concluded that the bed configuration of an alluvial river varies in accordance with water temperature; that is, crossings and bends are lowered and amplitudes of dunes or sand waves are reduced in some relationship with reduced water temperature (fig. 1).[1] These several effects substantially reduce resistance to flow, which, in turn, should produce the end result of a lowered water surface; that is, with colder water a given flow should move past any given point at a lower stage.

Collection and Analysis of Basic Data

The fathometer was the instrument used to collect the basic data; that is, continuous thalweg profiles. Vertical and longitudinal controls were good. Longitudinal control was provided by taking fixes on navigation lights and controlling boat speed by tachometer. Lateral control was somewhat less satisfactory. The longitudinal scale for the first runs was 200 feet to the inch and 300 feet to the inch for the later runs. Either scale shows the riverbed in great detail.

Seven thalweg profile runs, totaling nearly 3,000 miles, were made from New Orleans to Old River (200 miles) and return at the nominal stage of 37 feet Red River Landing gage (fig. 2).[2] River temperatures ranged from 42° to 83° F., the latter being for the August 1958[3] run, which is the basis for comparison with all the other runs made at lower temperatures. (table 1). The temperature extremes encountered in this field work are only moderately less than the all-time records at the Eagle Street water intake of the New Orleans Sewerage and Water Board.[4] The temperatures at the intake were found to agree closely with those observed by the survey boat at various locations between New Orleans and Old River during the several thalweg profile runs. As nearly as is known, the only variable involved in these seven thalweg profile runs was temperature.

Practical Implications

Some practical implications of these conclusions are suggested by examining two unresolved hydraulic anomalies: First, there is the fact that the 1950 flood, the principal criterion for determining the "project flood flowline" for the lower Mississippi, was out of step with some other historic floods. The fact that this flood crest passed New Orleans at a temperature of 50° F. suggests the possibility of temperature as the factor that might account for this disagreement with other historic floods, the crests

[1] The chief factor in the variation in wave size in figure 1 is volume of flow, and any effect of temperature is completely obscured. Figure 1 serves here simply to show nature of waves.

[2] Another series of thalweg profile runs at a nominal stage of 12.5 feet Red River Landing gage has been completed. The analysis of these data was not completed at the time of this paper, however, and will be presented in a subsequent paper.

[3] The August 1958 rise was notable in that it was the highest stage at Red River Landing in August in 30 years. It thus provided a much needed basis (82°-83° F.) for comparison with lower temperature flows at the same stage.

[4] Maximum temperature of record was 90° F., Aug. 17, 1937; minimum temperature was 36°, Jan. 30, 1948.

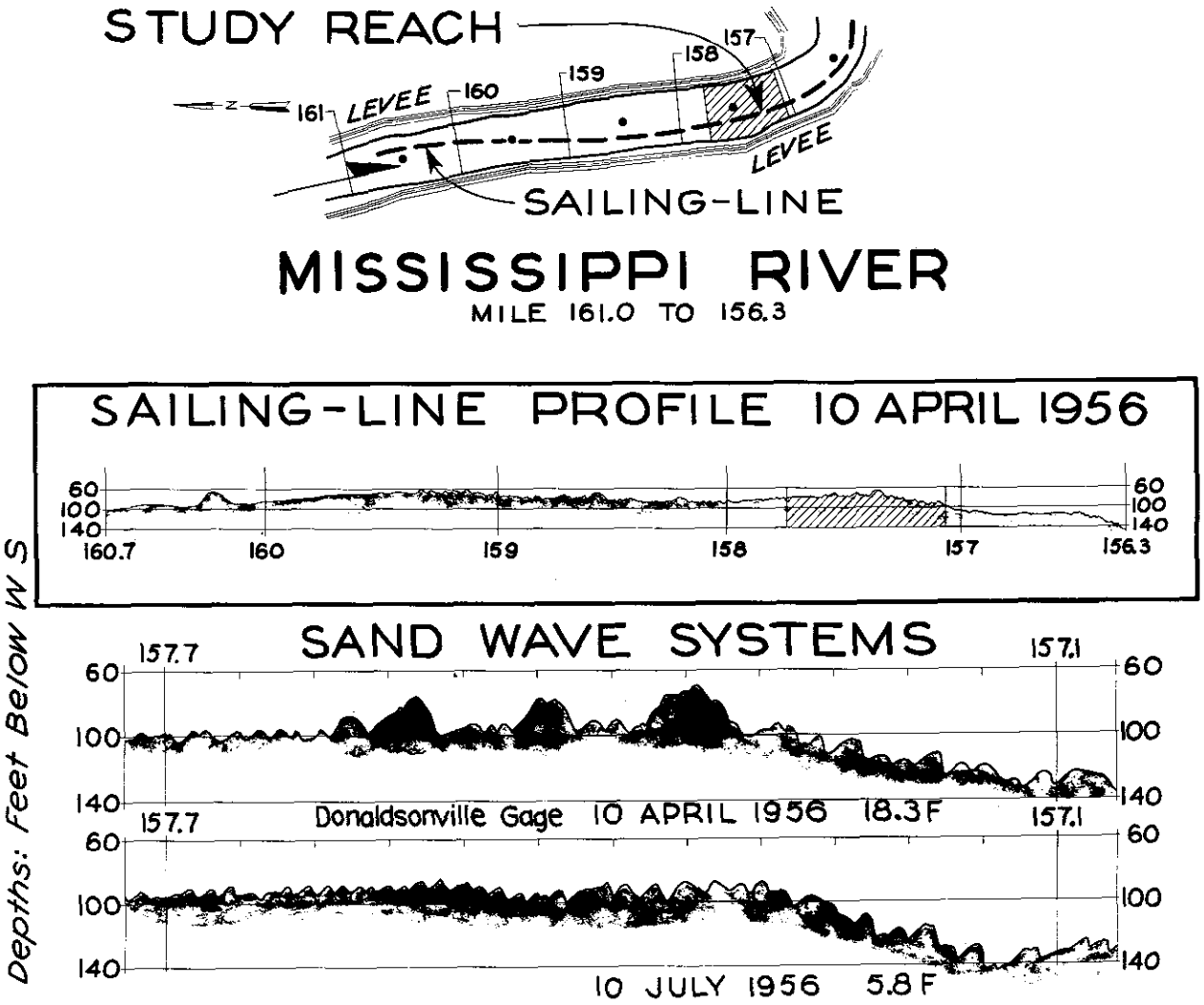


FIGURE 1. — Segments of sailing-line (thalweg) profiles at 18.3 and 5.8 feet, respectively, Donaldsonville gage, showing sand-wave systems.

of which arrived some weeks later than the 1950 crest and with somewhat higher water temperatures. Similarly, water temperature may also account for the unexplained behavior of the Mississippi at St. Louis, Mo., where it is reported that summer floods are substantially less in volume of flow as compared with spring floods reaching the same stage; that is, the river capacity seems to be greater with the colder water.

Scope of Study

This paper reports the results of field investigations concerning changes in riverbed configuration and also presents certain official published discharge records, arranged in such a manner as to provide additional support for the thesis of this paper. The full hydraulic implications of this empirical study are left for fuller exposition as a result of some future broad scale investigation and study of this aspect of river flow.

The riverbed statistics (tables 2 to 8), which provide the principal basis for this paper and its conclusions, were derived from measurements and studies made on the thalweg profiles resulting from the seven field runs over the period August 1958 to February 1962. Although temperature is suggested as being the only known variable, the riverbed statistics do not seem to conform, or fall into order, to the degree that might be expected, if temperature were the sole influence. There is a possibility that some element is involved, such as a "lag" in riverbed configuration adjustment, necessitated as a result of flow or temperature changes. In a model, time could be allowed for such adjustments, but, on the natural river, the field obser-

TABLE 1.—*Mississippi River water temperature (° F.), 1958–62*

Days of month	June 1958	July 1958	Aug. 1958	Aug. 1958 [1]	Dec. 1958	Jan. 1959	Feb. 1959	Feb. 1959 [1]	Feb. 1960	Mar. 1960	Apr. 1960	Apr. 1960 [1]	Jan. 1961	Feb. 1961	Mar. 1961	Mar. 1961 [1]	Apr. 1961	May 1961	May 1961 [1]	June 1961	June 1961 [1]	Dec. 1961	Jan. 1962	Feb. 1962	Feb. 1962 [1]
1	.....	80	84	.....	54	.....	.....	.....	49	.....	.....	.....	43	44	53	.....	56	61	.....	70	70	53	45	40	.....
2	77	80	.....	.....	54	44	45	.....	48	42	.....	.....	43	43	51	.....	58	62	.....	70	70	53	44	41	.....
3	77	80	.....	.....	53	.....	45	.....	48	42	.....	.....	43	43	51	.....	58	63	.....	70	70	53	44	42	.....
4	77	.....	84	83	53	.....	45	.....	48	42	50	.....	43	40	53	.....	58	63	.....	70	70	52	44	42	.....
5	78	.....	84	82	53	43	45	.....	49	.....	51	.....	48	41	52	.....	56	64	.....	71	71	51	44	43	.....
6	78	.....	84	82	.....	43	45	.....	.....	.....	53	.....	48	44	54	.....	56	65	.....	71	72	51	44	42	.....
7	.....	80	84	82	.....	43	.....	.....	.....	42	54	.....	48	41	54	.....	56	65	.....	72	73	50	44	43	.....
8	.....	80	84	83	53	43	.....	.....	48	42	54	.....	46	41	54	.....	56	65	66	73	.....	50	43	44	.....
9	80	81	.....	.....	53	43	46	43	48	43	.....	.....	46	40	53	.....	57	64	65	73	.....	51	43	44	.....
10	81	81	.....	.....	52	.....	.....	42	48	43	.....	.....	47	42	54	.....	56	64	64	74	.....	50	42	44	.....
11	81	81	84	82	52	.....	46	42	48	.....	55	.....	43	42	54	.....	60	65	65	74	.....	51	42	44	.....
12	81	.....	84	83	51	43	45	43	49	.....	55	.....	43	42	53	.....	60	65	65	75	.....	51	42	45	.....
13	82	.....	84	82	.....	44	46	.....	.....	42	54	.....	43	42	53	.....	58	66	.....	75	.....	52	40	45	.....
14	.....	82	84	82	.....	44	.....	.....	.....	42	54	.....	44	42	54	.....	56	66	.....	75	.....	51	40	45	.....
15	.....	82	84	83	48	44	.....	.....	48	42	.....	.....	43	44	54	.....	56	65	.....	75	...	52	40	45	.....
16	83	82	.....	.....	47	44	46	43	48	42	.....	.....	43	42	54	.....	56	66	.....	75	.....	52	40	46	47
17	83	82	.....	.....	46	.....	46	42	48	42	.....	.....	43	42	54	.....	58	66	.....	76	.....	51	40	46	46
18	84	83	84	83	46	.....	46	.....	49	.....	56	.....	43	43	58	.....	58	66	.....	76	.....	51	39	46	47
19	84	.....	83	.....	45	43	45	.....	49	.....	.....	.....	44	45	58	.....	58	66	.....	77	.....	50	39	47	47
20	83	.....	83	.....	.....	43	45	.....	.....	43	58	.....	42	46	60	56	58	68	.....	77	.....	50	39	47	.....
21	.....	84	83	.....	.....	.....	.....	.....	.....	44	58	56	43	50	60	56	57	68	.....	76	.....	50	39	47	.....
22	.....	84	83	.....	45	43	.....	.....	44	44	59	57	44	48	58	56	57	68	.....	76	.....	50	39	47	.....
23	82	84	.....	.....	45	44	46	.....	44	45	.....	59	43	53	58	58	58	68	.....	76	.....	48	39	47	.....
24	82	84	.....	.....	44	.....	46	.....	43	45	.....	60	43	58	55	57	58	68	.....	76	.....	48	39	48	.....
25	82	.....	84	.....	.....	.....	46	.....	44	.....	60	60	42	58	54	55	58	69	.....	76	.....	46	40	48	.....
26	83	.....	84	.....	.....	44	46	.....	44	.....	62	61	43	56	54	.....	59	70	.....	76	.....	46	40	48	.....
27	84	.....	83	.....	.....	44	46	.....	.....	47	62	.....	42	56	55	.....	60	70	.....	76	.....	46	39	50	.....
28	.....	84	83	.....	.....	45	.....	.....	.....	48	64	.....	42	53	56	.....	60	70	.....	76	.....	45	39	50	.....
29	.....	84	83	.....	45	45	.....	.....	44	48	64	.....	42	.....	56	.....	61	70	.....	76	.....	45	40	.....	.....
30	84	85	.....	.....	45	45	.....	.....	.....	48	.....	.....	43	.....	56	.....	61	70	.....	76	.....	45	40	.....	.....
31	.....	85	.....	.....	.....	.....	.....	.....	.....	48	.....	.....	43	.....	.....	.....	.....	70	.....	.....	.....	46	40	.....	.....

[1] Temperatures observed in river from survey boat. Other temperatures recorded by Eagle Street Plant, Sewerage and Water Board, New Orleans, La.

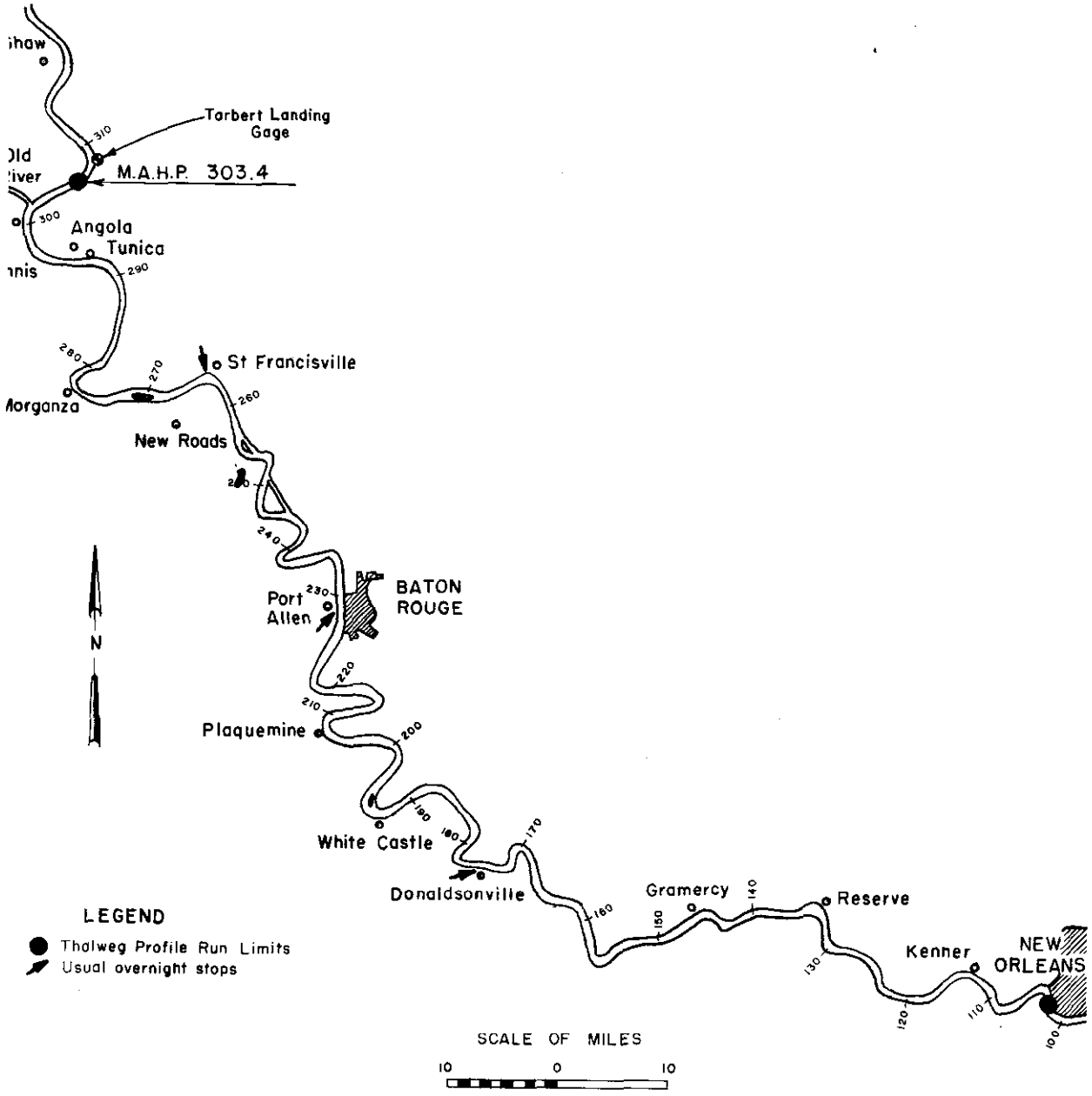


FIGURE 2. — Reach of river on which seven thalweg profile runs were made, 1958-62.

vations must be made when and as feasible. Some of the riverbed data were obtained by using thalweg profile microfilms in a "viewer"; a much more convenient technique than using the fathometer rolls or reproductions of same.

The resulting riverbed data are tabulated in two ways:

(1) Tables 2 to 7: a detailed tabulation of comparative measurements and physical characteristics of the crossings at the times of the several thalweg surveys. Study of the stage and discharge hydrographs (figs. 3, 4, 5, 6, and 7) is suggested as a preliminary review to a study of the tables.

The thalweg run in February 1959 was made at a stage substantially (6 or more feet) lower than other runs, and the data are accordingly not strictly comparable. The lowerings of the crossings and the reductions in sand wave amplitudes in tables 2 and 8 would have been substantially greater if the February 1959 run had been made at the same nominal stage (37

TABLE 2.—*Effect of water temperature on riverbed configuration, Mississippi River, mile 110 to mile 298 A.H.P., August 4 to 18, 1958, (82° to 83° F.) versus February 9 to 18, 1959 (42° to 43° F.)* [1]

Crossing location mile, A.H.P.,[2] date, and direction	High point M.L.W.[3]	Average difference in elevation	Wave amplitude and length	Average difference in amplitude	Condition of crossing slopes		Description of top of crossing: nature and location of wave formations and of smooth stretches
					Approach slope	Leaving slope	
	Feet	*Feet*	*Feet*	*Feet*			
110.0, Aug. 1958, upstream.	−63	.....	Amplitude 6, length 100.	.....	Rough. Waves 4 to 6 ft.	Upper end rough. Waves 3 to 5 ft. lower part smooth.	Top rough. Waves 3 to 6 ft.
110.0, Aug. 1958, downstream.	−60	.....	Amplitude 13, length 240.	.....	Rough. Waves 6 to 13 ft.	Almost smooth throughout.	Top rough. Waves 6 to 13 ft.
110.0, Feb. 1959, upstream.	−68	.....	([4])	.....	No definite waves. Smooth undulations.	Upper end smooth with undulations. 5 ft. waves at lower end.	Top smooth with general undulations.
110.0, Feb. 1959, downstream.	−66	−5.5	Amplitude 8, length 190.	−5.5	Smooth with some general undulations.	Upper end waves 4 to 6 ft. Smooth to lower end.	Top smooth with general undulations.
116.0, Aug. 1958, upstream.	−41.0	.....	Amplitude 7, length 125.	.....	Smooth to near upper end. Then waves 2 to 3 ft.	Upper end waves 3 to 5 ft.	Top rough. Waves 4 to 7 ft.
116.0, Aug. 1958, downstream.	−39.5	.....	Amplitude 8, length 165.	.....	Lower end undulating. Upper end rough. Waves 4 to 6 ft.	Rough waves 3 to 6 ft.	Top rough. Waves 5 to 8 ft.
116.0, Feb. 1959, upstream.	−44.0	.....	Amplitude 2, length 35.	.....	Lower end smooth. Upper end waves 1 to 2 ft.	Smooth with some very gentle undulations.	Top smooth. Some gentle undulations.
116.0, Feb. 1959, downstream.	−45.0	−4.25	Amplitude 4, length 70.	−4.5	Lower end waves 2 ft. Upper end waves 4 ft.	Smooth	Top smooth and regular.
140.5, Aug. 1958, upstream.	−54.5	.....	Amplitude 5, length 90.	.....	Rough. Waves 3 to 5 ft.	Upper end rough. Waves 2 to 4 ft. Lower end smooth.	Top rough. Waves 3 to 5 ft.
140.5, Aug. 1958, downstream.	−57.0	.....	Amplitude 6, length 170.	.....	Upstream end smooth. Downstream end waves 2 to 5 ft.	Upstream end rough with 2 to 3 ft. waves. Downstream end smooth.	Top rough. Waves 3 to 6 ft.
140.5, Feb. 1959, upstream.	−59.0	.....	([5])	.....	Smooth.	Smooth.	Top smooth.
140.5, Feb. 1959, downstream.	−58.0	−2.75	([5])	−5.5	Smooth. Slightly rough upstream end.	Smooth.	Top smooth.
154.0, Aug. 1958, upstream.	−34.0	.....	Amplitude 6, length 130.	.....	Slightly rough. Waves 1 to 2 ft.	Rough. Waves 2 to 4 ft.	Top rough. Waves 2 to 6 ft.
154.0, Aug. 1958, downstream.	−34.5	.....	Amplitude 7, length 125.	.....	Rough. Waves 3 to 7 ft.	Rough. Waves 4 to 6 ft.	Top rough. Waves 3 to 6 ft.
154.0, Feb. 1959, upstream.	−37.5	.....	Amplitude 3, length 75.	.....	Smooth nearly to top. Then waves 1 to 2 ft.	Smooth	Upstream half with waves 1 to 3 ft. Downstream half smooth.
154.0, Feb. 1959, downstream.	−34.0	−1.50	Amplitude 4, length 100.	−3.0	Rough. Waves 3 to 4 ft.	Smooth.	Rough except for upstream quarter which is smooth. Waves 2 to 3 ft.
163.0, Aug. 1958, upstream.	−44.5	.....	Amplitude 8, length 150.	.....	Smooth upstream half. Waves 2 to 8 ft.	Smooth downstream half. Waves 3 to 5 ft.	Top rough. Waves 3 to 5 ft.
163.0, Aug. 1958, downstream.	−48.0	.....	Amplitude 9, length 110.	.....	Rough. Waves 4 to 9 ft.	Smooth downstream half. Waves 3 to 5 ft.	Top rough. Waves 3 to 6 ft.
163.0, Feb. 1959, upstream.	−45.5	.....	Amplitude 4, length 125.	.....	Smooth.	Rough except at middle. Waves 3 ft.	Top smooth upstream half. Rough downstream half. Waves 2 to 4 ft.

TABLE 2.—*Effect of water temperature on riverbed configuration, Mississippi River, mile 110 to mile 298 A.H.P., August 4 to 18, 1958, (82° to 83° F.) versus February 9 to 18, 1959 (42° to 43° F.)*[1]*—Continued*

Crossing location mile, A.H.P.,[2] date and direction	High point M.L.W.[3]	Average difference in elevation	Wave amplitude and length	Average difference in amplitude	Condition of crossing slopes		Description of top of crossing: nature and location of wave formations and of smooth stretches
					Approach slope	Leaving slope	
	Feet	*Feet*	*Feet*	*Feet*			
163.0, Feb. 1959, downstream.	−51.5	−2.0	Amplitude 2, length 60.	−5.5	Smooth except downstream third. Waves 2 ft.	Smooth except 4 ft. waves upstream toe.	Top rough. Waves 2 to 3 ft.
167.0, Aug. 1958, upstream.	−54.5	.....	Amplitude 11, length 200.	.....	Smooth. Small waves 2 to 3 ft. at top.	Rough. Waves 6 to 11 ft.	Top rough. Waves 6 to 10 ft.
167.0, Aug. 1958, downstream.	−55.0	.....	Amplitude 12, length 270.	.....	Rough. Small waves 2 to 3 ft. on remnants large waves 4 to 6 ft.	Rough. Waves 5 to 9 ft.	Top rough. Waves 6 to 12 ft.
167.0, Feb. 1959. upstream.	−55.0	.....	Amplitude 8, length 150.	.....	Rough 5 to 7 ft. Smoothing up at bottom.	Smooth.	Top rough. Waves 5 to 8 ft. Downstream end smooth.
167.0, Feb. 1959, downstream.	−53.5	+ .5	Amplitude 10, length 250.	−2.5	Rough. Waves 3 to 6 ft.	Smooth.	Top rough. Waves 5 to 10 ft.
183.0, Aug. 1958, upstream.	−44.0	.....	Amplitude 14, length 210.	.....	Rough. Waves 2 to 3 ft.	Smo h.	Rough. Waves 6 to 10 ft.
183.0, Aug. 1958, downstream.	−41.0	.....	Amplitude 14, length 375.	.....	Rough. Waves 3 to 6 ft.	Rough. Waves 3 to 6 ft.	Rough. Waves 6 to 14 ft.
183.0, Feb. 1959, upstream.	−46.0	.....	Amplitude 14, length 180.	.....	Rough. Waves 6 to 9 ft.	Smooth. Very steep.	Smooth.
183.0, Feb. 1959, downstream.	−41.5	−1.25	Amplitude 11, length 250.	−1.5	Rough. Waves 3 to 10 ft.	Rough. Waves 6 to 11 ft.	Smooth except up and downstream ends with waves 3 to 6 ft.
190.5, Aug. 1958, upstream.	−34.5	.....	Amplitude 10, length 160.	.....	Smooth upstream half. Waves downstream 2 to 6 ft.	Rough. Waves 5 to 8 ft.	Top rough. Waves 6 to 10 ft.
190.5, Aug. 1958, downstream.	−36.0	.....	Amplitude 13, length 180.	.....	Smooth at bottom. Waves 2 to 6 ft.	Rough. Waves 3 to 6 ft.	Top rough. Waves 4 to 13 ft.
190.5, Feb. 1959, upstream.	−38.0	.....	Amplitude 7, length 160.	.....	Smooth at bottom. Waves 3 to 7 ft. at top.	Smooth.	Top smooth except short stretch upstream with waves 4 to 5 ft.
190.5, Feb. 1959, downstream.	−37.5	−2.50	Amplitude 7, length 120.	−4.5	Smooth at bottom. Waves 3 to 5 ft. at top.	Smooth.	Top smooth except short stretch upstream with waves 3 to 7 ft.
197.5, Aug. 1958, upstream.	−37.5	.....	Amplitude 9, length 160.	.....	Generally smooth with waves 1 to 2 ft.	Rough. Waves 2 to 5 ft.	Top rough. Waves 5 to 9 ft.
197.5, Aug. 1958, downstream.	−36.0	.....	Amplitude 10, length 190.	.....	Smooth at bottom. Rough with waves.	Rough. Waves.	Top rough. Waves 6 to 10 ft.
197.5, Feb. 1959, upstream.	−39.5	.....	Amplitude 8, length 175.	.....	Rough. Waves 3 to 8 ft.	Smooth.	Top smooth except small part upstream end waves 5 to 8 ft.
197.5, Feb. 1959, downstream.	−37.5	−1.75	Amplitude 7, length 175.	−2.0	Rough. Waves 1 to 2 ft.	Smooth.	Top mostly smooth. Upstream end waves 2 to 7 ft.
204.0, Aug. 1958, upstream.	−33.0	.....	Amplitude 12, length 170.	.....	Rough. Waves 3 to 12 ft.	Smooth at bottom. Waves 2 to 5 ft.	Top rough. Waves 6 to 12 ft.
204.0, Aug. 1958, downstream.	−34.5	.....	Amplitude 10, length 190.	.....	Rough. Waves 3 to 9 ft.	Smooth.	Top rough. Waves 5 to 10 ft.
204.0, Feb. 1959, upstream.	−40.0	.....	Amplitude 8, length 130.	.....	Rough. Waves 3 to 9 ft.	Smooth except few 2 to 3 ft. waves at bottom.	Top smooth, except few waves 2 to 5 ft. at upstream end.

204.0, Feb. 1959, downstream.	−37.5	−5.0	Amplitude 7, length 140.	−3.5	Bottom smooth. Waves 2 to 7 ft. to top.	Smooth.	Top smooth, except upstream end, waves 2 to 7 ft.
212.0, Aug. 1958, upstream.	−38.0	.....	Amplitude 12, length 200.	.....	Regular at bottom, waves 1 to 2 ft. 3 to 6 ft. waves at top.	Rough. Waves 3 to 8 ft.	Top rough. Waves 5 to 12 ft.
212.0, Aug. 1958, downstream.	−36.5	.....	Amplitude 17, length 250.	.....	Regular at bottom, waves 1 to 2 ft. 3 to 7 ft. waves at top.	Rough. Waves 3 to 8 ft.	Top rough. Waves 7 to 12 ft.
212.0, Feb. 1959, upstream.	−41.5	.....	Amplitude 9, length 105.	.....	Rough. Waves 5 to 9 ft.	Smooth.	Top smooth, except waves 2 to 7 ft. at upstream end.
212.0, Feb. 1959, downstream.	−41.5	−4.2	Amplitude 9, length 140.	−5.5	Rough. Waves 4 to 9 ft.	Smooth.	Top smooth, except upstream third with waves 2 to 7 ft.
223.5, Aug. 1958, upstream.	−31.0	.....	Amplitude 13, length 450.	.....	Rough. Waves 3 to 10 ft.	Rough. Waves 2 to 9 ft.	Rough. Waves 8 to 13 ft.
223.5, Aug. 1958, downstream.	−32.0	.....	Amplitude 15, length 475.	.....	Rough. Waves 4 to 12 ft.	Rough. Waves 3 to 9 ft.	Rough. Waves 6 to 15 ft.
223.5, Feb. 1959, upstream.	−33.5	.....	Amplitude 13, length 500.	.....	Smooth with undulations at bottom.	Rough. Waves 3 to 7 ft.	Rough. Waves 5 to 13 ft.
223.5, Feb. 1959, downstream.	−33.5	−2.0	Amplitude 13, length 500.	−1.0	Rough. Waves 5 to 11 ft.	Rough. Waves 4 to 11 ft.	Rough. Waves 5 to 13 ft.
236.0, Aug. 1958, upstream.	−24.0	.....	Amplitude 20, length 350.	.....	Smooth with waves 1 to 2 ft.	Rough. Waves 3 to 8 ft.	Rough. Waves 9 to 17 ft.
236.0, Aug. 1958, downstream.	−26.5	.....	Amplitude 10, length 100.	.....	Rough. Waves 4 to 7 ft.	Rough. Waves 5 to 7 ft.	Rough. Waves 5 to 10 ft.
236.0, Feb. 1959, upstream.	−33.0	.....	Amplitude 17, length 330.	.....	Smooth.	Smooth with undulations.	Rough. Waves 7 to 17 ft. Beginning to smooth out.
236.0, Feb. 1959, downstream.	−28.5	−5.50	Amplitude 9, length 100.	−2.0	Rough. Waves 3 to 6 ft.	Lower part smooth. Waves 5 to 9 ft.	Rough. Waves 2 to 6 ft. Beginning to smooth out.
241.0, Aug. 1958, upstream.	−23.0	.....	Amplitude 8, length 130.	.....	Slightly rough with waves 1 to 3 ft.	Upper part smooth. Waves 3 to 5 ft.	Rough. Waves 3 to 8 ft.
241.0, Aug. 1958, downstream.	−20.0	.....	Amplitude 15, length 200.	.....	Rough. Waves 4 to 8 ft.	Smooth except upper part with waves 3 to 6 ft.	Rough. Waves 5 to 12 ft.
241.0, Feb. 1959, upstream.	−25.0	.....	Amplitude 12, length 300.	.....	Smooth.	Smooth.	Smooth except downstream quarter with waves 5 to 12 ft.
241.0, Feb. 1959, downstream.	−24.0	−3.0	Amplitude 12, length 350.	− .5	Regular with waves 2 to 3 ft.	Lower part smooth. Waves 5 to 10 ft.	Rough. Waves 4 to 12 ft.
251.0, Aug. 1958, upstream.	−12.5	.....	Amplitude 14, length 275.	.....	Regular with waves 2 to 6 ft.	Rough. Waves 3 to 7 ft.	Rough. Waves 5 to 14 ft.
251.0, Aug. 1958, downstream.	−16.5	.....	Amplitude 14, length 250.	.....	Regular with waves 2 to 6 ft.	Rough. Waves 5 to 14 ft.	Rough. Waves 4 to 14 ft.
251.0, Feb. 1959, upstream.	−19.5	.....	Amplitude 11, length 175.	.....	Lower part smooth. Upper part waves 3 to 6 ft.	Rough. Waves 4 to 11 ft.	Downstream quarter is smooth. Remainder waves 4 to 8 ft.
251.0, Feb. 1959, downstream.	−20.0	−5.25	Amplitude 8, length 75.	−4.5	Lower part smooth. Upper part waves 3 to 8 ft.	Rough. Waves 3 to ? ft.	Rough. Waves 3 to 7 ft.
258.5, Aug. 1958, upstream.	−20.0	.....	Amplitude 16, length 500.	.....	Regular with small waves 2 to 3 ft.	Rough. Waves 4 to 9 ft.	Rough. Waves 4 to 16 ft.
258.5, Aug. 1958, downstream.	−18.0	.....	Amplitude 15, length 500.	.....	Regular with small waves 2 to 5 ft.	Rough. Waves 4 to 10 ft.	Rough. Waves 4 to 16 ft.

See footnotes at end of table.

TABLE 2.—*Effect of water temperature on riverbed configuration, Mississippi River, mile 110 to mile 298 A.H.P., August 4 to 18, 1958, (82° to 83° F.) versus February 9 to 18, 1959 (42° to 43° F.)*[1]*—Continued*

Crossing location mile, A.H.P.,[2] date, and direction	High point M.L.W.[3]	Average difference in elevation	Wave amplitude and length	Average difference in amplitude	Condition of crossing slopes		Description of top of crossing: nature and location of wave formations and of smooth stretches
					Approach slope	Leaving slope	
	Feet	*Feet*	*Feet*	*Feet*			
258.5, Feb. 1959, upstream.	−21.0	.....	Amplitude 12, length 350.	.....	Regular with waves 2 to 5 ft.	Rough. Waves 4 to 10 ft.	Rough. Waves 3 to 12 ft.
258.5, Feb. 1959, downstream.	−19.5	−1.25	Amplitude 11, length 80.	−4.0	Regular with small waves 2 to 3 ft.	Rough. Waves.	Rough. Waves 6 to 11 ft.
269.5, Aug. 1958, upstream.	−30.5	.....	Amplitude 12, length 250.	.....	Rough. Waves 5 to 12 ft.	Top rough with 3 to 10 ft. waves. Bottom smooth.	Rough. Waves 4 to 12 ft.
269.5, Aug. 1958, downstream.	−39.0	.....	Amplitude 15, length 225.	.....	Rough. Waves 3 to 6 ft.	Top rough with 3 to 6 ft. waves.	Rough. Waves 4 to 15 ft.
269.5, Feb. 1959, upstream.	−33.5	.....	Amplitude 11, length 225.	.....	Rough.	Smooth at bottom. Top with waves 5 to 11 ft.	Upstream end smooth. Downstream with waves 3 to 8 ft.
269.5, Feb. 1959, downstream.	−37.5	−0.75	Amplitude 11, length 275.	−2.5	Rough. Waves 3 to 9 ft.	Smooth with undulations.	Rough. Waves 3 to 11 ft.
273.0, Aug. 1958, upstream.	− 9.0	.....	Amplitude 16, length 1000.	.....	Regular with 2 to 5 ft. long undulations.	Smooth with long undulations.	Rough upper end with very long waves 5 to 16 ft. Lower end smooth.
273.0, Aug. 1958, downstream.	− 7.0	.....	Amplitude 21, length 1250.	.....	Smooth at bottom. Top with 2 to 6 ft. waves.	Waves 3 to 5 ft. and long undulations.	Rough. Very long waves 7 to 17 ft.
273.0, Feb. 1959, upstream.	−16.5	.....	Amplitude 13, length 350.	.....	Rough Waves 3 to 5 ft.	Rough. Waves 2 to 3 ft.	Rough. Waves 3 to 13 ft.
273.0, Feb. 1959, downstream.	−18.5	−9.5	Amplitude 8, length 195.	−8.0	Rough Waves 3 to 6 ft.	Rough. Waves 3 to 6 ft.	Smooth.
279.0, Aug. 1958, upstream.	−32.5	.....	Amplitude 22, length 350.	.....	Rough. Waves 3 to 7 ft.	Rough. Waves 3 to 6 ft.	Rough. Waves 4 to 15 ft.
279.0, Aug. 1958, downstream.	−29.0	.....	Amplitude 23, length 350.	.....	Rough. Waves 3 to 5 ft.	Smooth.	Rough. Waves 4 to 14 ft.
279.0, Feb. 1959, upstream.	−27.5	.....	Amplitude 10.[6]	.....	Smooth.	Rough. Undulations 3 to 6 ft.	Smooth.
279.0, Feb. 1959, downstream.	−32.0	+1.0	Amplitude 6, length 175.	−14.5	Rough at bottom third. Remainder smooth.	Smooth and steep.	Smooth.
286.5, Aug. 1958, upstream.	− 6.0	.....	Amplitude 25, length 425.	.....	Long undulations 3 to 6 ft. with small waves 1 to 2 ft. superimposed.	Long undulations 3 to 5 ft.	Rough. Very large waves 6 to 19 ft.
286.5, Aug. 1958, downstream.	− 7.0	.....	Amplitude 25, length 350.	.....	Long undulations 3 to 6 ft. with small 1 to 2 ft. waves superimposed.	Long undulations 3 to 5 ft.	Rough. Very large waves 5 to 25 ft.
286.5, Feb. 1959, upstream.	−23.5	.....	Amplitude 10, length 160.	.....	Smooth.	Rough. Waves 3 to 10 ft.	Rough downstream two-thirds waves 3 to 8 ft. Upstream third smooth.
286.5, Feb. 1959, downstream.	−24.0	−17.25	Amplitude 8, length 125.	−16.0	Smooth.	Rough. Waves 3 to 8 ft.	Rough. Small waves 1 to 3 ft.
298.0, Aug. 1958, upstream.	− 0.5	.....	Amplitude 10, length 450.	.....	Long undulations 1 to 2 ft. waves superimposed.	Rough. Waves 3 to 9 ft.	Rough. Waves 4 to 10 ft.
298.0, Aug. 1958, downstream.	− 2.5	.....	Amplitude 11, length 250.	.....	Long waves 3 to 6 ft.	Smooth and steep.	Rough. Waves 3 to 11 ft.

298.0, Feb. 1959, upstream.	− 7.5	.....	Amplitude 12, length 375.	.....	Rough. Waves 3 to 6 ft.	Waves 2 to 3 ft. Steep.	Rough. Waves 3 to 10 ft.
298.0, Feb. 1959, downstream.	− 7.5	−6.5	Amplitude 6, length 250.	−1.5	Rough. Waves 3 to 6 ft.	Waves 1 to 2 ft. Steep.	Rough. Waves 2 to 6 ft. Upstream third beginning to smooth out.

[1]Because of the lower stage for the Feb. 9–18, 1959, run (see hydrographs), the differences in heights of crossings and wave amplitudes are only partially ascribable to temperature.
[2]Above head of passes.
[3]Mean low water.
[4]No definite waves. Smooth undulations.
[5]Smooth.
[6]No waves.

TABLE 3.—*Effect of water temperature on riverbed configuration, Mississippi River, mile 110 to mile 298 A.H.P.,[1] August 4 to 18, 1958, (82° to 83° F.) versus April 21 to 27, 1960 (56° to 61° F.)*

Crossing location mile, A.H.P.,[1] date, and direction	High point M.L.W.[2]	Average difference in elevation	Wave amplitude and length	Average difference in amplitude	Condition of crossing slopes		Description of top of crossing: nature and location of wave formations and of smooth stretches
					Approach slope	Leaving slope	
	Feet	*Feet*	*Feet*	*Feet*			
110.0, Aug. 1958, upstream.	−63.0	.....	Amplitude 6, length 100.	.....	Rough. Waves 4 to 6 ft.	Upstream end rough. Waves 3 to 5 ft. Downstream part smooth.	Top rough. Waves 3 to 6 ft.
110.0, Aug. 1958, downstream.	−60.0	.....	Amplitude 13, length 240.	.....	Rough. Waves 6 to 13 ft.	Almost smooth throughout.	Top rough. Waves 6 to 13 ft.
110.0, Apr. 1960, upstream.	−65.0	.....	Amplitude 6, length 190.	.....	Rough. Waves 2 to 5 ft.	Rough. Waves 4 to 9 ft.	Top rough. Waves 3 to 6 ft.
110.0, Apr. 1960, downstream.	−64.0	−3.0	Amplitude 9, length 250.	−2.00	Smooth. Waves 2 to 4 ft.	Rough. Waves 3 to 10 ft. Steep drop at downstream end.	Top rough. Waves 4 to 9 ft.
116.0, Aug. 1958, upstream.	−41.0	.....	Amplitude 7, length 125.	.....	Smooth to near upstream end. Then waves 2 to 3 ft.	Upstream end, waves 3 to 5 ft.	Top rough. Waves 4 to 7 ft.
116.0, Aug. 1958, downstream.	−39.5	.....	Amplitude 8, length 165.	.....	Downstream end undulating. Upstream end rough. Waves 4 to 6 ft.	Rough. Waves 3 to 6 ft.	Top rough. Waves 5 to 8 ft.
116.0, Apr. 1960, upstream.	−42.5	.....	Amplitude 8, length 150.	.....	Smooth with slight ripples.	Smooth and steep.	Top rough. Waves 2 to 8 ft.
116.0, Apr. 1960, downstream.	−40.0	−1.00	Amplitude 6, length 100.	− .50	Rough. Waves 2 to 6 ft.	Rough. Waves 4 to 10 ft.	Top rough. Waves 2 to 6 ft.
140.5, Aug. 1958, upstream.	−54.5	.....	Amplitude 5, length 90.	.....	Rough. Waves 3 to 5 ft.	Upstream end rough. Waves 2 to 4 ft. Downstream end smooth.	Top rough. Waves 3 to 5 ft.
140.5, Aug. 1958, downstream.	−57.0	.....	Amplitude 6, length 170.	.....	Upstream end smooth. Downstream end, waves 2 to 5 ft.	Upstream end rough. Waves 2 to 3 ft. Downstream end smooth.	Top rough. Waves 3 to 6 ft.
140.5, Apr. 1960, upstream.	−52.5	.....	Amplitude 9, length 140.	.....	Smooth at bottom. Rough at top. Waves 2 to 5 ft.	Rough. Waves 2 to 13 ft.	Top rough. Waves 3 to 9 ft.
140.5, Apr. 1960, downstream.	−51.0	+4.0	Amplitude 11, length 250.	+4.50	Rough. Waves 1 to 6 ft.	Rough. Waves 4 to 8 ft.	Rop rough. Waves 2 to 11 ft.

See footnotes at end of table.

TABLE 3.—*Effect of water temperature on riverbed configuration, Mississippi River, mile 110 to mile 298 A.H.P.,[1] August 4 to 18, 1958, (82° to 83° F.) versus April 21 to 27, 1960 (56° to 61° F.)—Continued*

Crossing location mile, A.H.P.,[1] date, and direction	High point M.L.W.[2]	Average difference in elevation	Wave amplitude and length	Average difference in amplitude	Condition of crossing slopes		Description of top of crossing: nature and location of wave formations and of smooth stretches
					Approach slope	Leaving slope	
	Feet	*Feet*	*Feet*	*Feet*			
154.0, Aug. 1958, upstream.	−34.0	.....	Amplitude 6, length 130.	.....	Slightly rough. Waves 1 to 2 ft.	Rough. Waves 2 to 4 ft.	Top rough. Waves 2 to 6 ft.
154.0, Aug. 1958, downstream.	−34.5	.....	Amplitude 7, length 125.	.....	Rough. Waves 3 to 7 ft.	Rough. Waves 4 to 6 ft.	Top rough. Waves 3 to 6 ft.
154.0, Apr. 1960, upstream.	−38.0	.....	Amplitude 8, length 180.	.....	Smooth. Small waves 1 to 2 ft.	Smooth, steep downstream drop.	Extreme upstream end rough, waves 1 to 8 ft. Middle portion and downstream end smooth with waves 1 to 2 ft.
154.0, Apr. 1960, downstream.	−34.5	−2.00	Amplitude 9, length 200.	+2.00	Rough. Waves 3 to 5 ft.	Rough. Waves 3 to 9 ft.	Top rough. Waves 4 to 9 ft.
163.0, Aug. 1958, upstream.	−44.5	.....	Amplitude 8, length 150.	.....	Smooth downstream half. Waves 2 to 8 ft.	Smooth downstream half. Waves 3 to 5 ft.	Top rough. Waves 3 to 5 ft.
163.0, Aug. 1958, downstream.	−48.0	.....	Amplitude 9, length 110.	.....	Rough. Waves 4 to 9 ft.	Smooth downstream half. Waves 3 to 5 ft.	Top rough. Waves 3 to 6 ft.
163.0, Apr. 1960, upstream.	−47.0	.....	Amplitude 6, length 130.	.....	Rough. Waves 2 to 5 ft.	Rough. Waves 2 to 10 ft. Steep slope.	Top rough. Waves 3 to 6 ft.
163.0, Apr. 1960, downstream.	−46.0	−0.25	Amplitude 9, length 180.	−1.00	Rough. Waves 4 to 17 ft.	Smooth, steep slope with slight ripples 1 ft.	Top rough. Waves 4 to 9 ft.
167.0, Aug. 1958, upstream.	−54.5	.....	Amplitude 11, length 200.	.....	Smooth. Small waves 2 to 3 ft. at top.	Rough. Waves 6 to 11 ft.	Top rough. Waves 6 to 10 ft.
167.0, Aug. 1958, downstream.	−55.0	.....	Amplitude 12, length 270.	.....	Rough. Small waves 2 to 3 ft. on remnants of large waves 4 to 6 ft.	Rough. Waves 5 to 9 ft.	Top rough. Waves 6 to 12 ft.
167.0, Apr. 1960, upstream.	−60.5	.....	Amplitude 11, length 270.	.....	Rough. Small waves 2 to 4 ft.	Rough. Waves 4 to 12 ft.	Top rough. Waves 2 to 11 ft.
167.0, Apr. 1960, downstream.	−57.0	−4.00	Amplitude 9, length 300.	−1.50	Smooth with ripples 1 to 3 ft.	Rough. Waves 4 to 12 ft.	Top rough. Waves 4 to 9 ft.
183.0, Aug. 1958, upstream.	−44.0	.....	Amplitude 14, length 210.	.....	Rough. Waves 2 to 3 ft.	Smooth.	Rough. Waves 6 to 10 ft.
183.0, Aug. 1958, downstream.	−41.0	.....	Amplitude 14, length 375.	.....	Rough. Waves 3 to 6 ft.	Rough. Waves 3 to 6 ft.	Rough. Waves 6 to 14 ft.
183.0, Apr. 1960, upstream.	−47.5	.....	Amplitude 19, length 550.	.....	Rough. Waves 4 to 17 ft.	Rough. Waves 4 to 22 ft.	Rough. Waves 4 to 19 ft.
183.0, Apr. 1960, downstream.	−45.0	−3.75	Amplitude 11, length 550.	+1.00	Rough. Waves 4 to 15 ft.	Rough. Waves 3 to 17 ft.	Top rough. Waves 3 to 11 ft.
190.5, Aug. 1958, upstream.	−34.5	.....	Amplitude 10, length 160.	.....	Smooth upstream half. Waves downstream 2 to 6 ft.	Rough. Waves 5 to 8 ft.	Top rough. Waves 6 to 10 ft.
190.5, Aug. 1958, downstream.	−36.0	.....	Amplitude 13, length 180.	.....	Smooth at bottom. Waves 2 to 6 ft.	Rough. Waves 3 to 6 ft.	Top rough. Waves 4 to 13 ft.
190.5, Apr. 1960, upstream.	−45.0	.....	Amplitude 10, length 400.	.....	Rough. Waves 3 to 8 ft.	Rough. Waves 3 to 9 ft.	Top rough. Waves 4 to 10 ft.
190.5, Apr. 1960, downstream.	−40.0	−7.50	Amplitude 14, length 500.	− .50	Rough. Waves 2 to 7 ft.	Rough. Waves 3 to 14 ft.	Top rough. Waves 3 to 14 ft.
197.5, Aug. 1958, upstream.	−37.5	.....	Amplitude 9, length 160.	.....	Generally smooth with waves 1 to 2 ft.	Rough. Waves 2 to 5 ft.	Top rough. Waves 5 to 9 ft.
197.5, Aug. 1958, downstream.	−36.0	.....	Amplitude 10, length 190.	.....	Smooth at bottom. Rough with waves.	Rough. Waves.	Top rough. Waves 6 to 10 ft.

197.5, Apr. 1960, upstream.	−38.0	.....	Amplitude 6, length 300.	.....	Smooth at bottom. Rough with waves 2 to 5 ft. at top.	Rough. Waves 2 to 4 ft.	Top rough. Waves 1 to 6 ft.
197.5, Apr. 1960, downstream.	−35.5	−0.00	Amplitude 7, length 400.	−3.00	Smooth at bottom with 1 ft. ripples. Rough at top.	Rough. Waves 3 to 11 ft.	Top rough with waves 3 to 7 ft.
204.0, Aug. 1958, upstream.	−33.0	.....	Amplitude 12, length 170.	.....	Rough. Waves 3 to 12 ft.	Smooth at bottom. Waves 2 to 5 ft.	Top rough. Waves 6 to 12 ft.
204.0, Aug. 1958, downstream.	−34.5	.....	Amplitude 10, length 190.	.....	Rough. Waves 3 to 9 ft.	Smooth.	Top rough. Waves 5 to 10 ft.
204.0, Apr. 1960, upstream.	−39.5	.....	Amplitude 12, length 300.	.....	Rough. Waves 3 to 15 ft.	Smooth and steep. Long undulations.	Top rough. Waves 4 to 12 ft.
204.0, Apr. 1960, downstream.	−38.0	−5.00	Amplitude 14, length 350.	+2.00	Rough. Waves 2 to 6 ft.	Smooth, steep slope with 1 ft. ripples.	Top rough. Waves 3 to 14 ft.
212.0, Aug. 1958, upstream.	−38.0	.....	Amplitude 12, length 200.	.....	Regular at bottom. Waves 1 to 2 ft. Waves 3 to 6-ft. at top.	Rough. Waves 3 to 8 ft.	Top rough. Waves 5 to 12 ft.
212.0, Aug. 1958, downstream.	−36.5	.....	Amplitude 17, length 250.	.....	Regular at bottom. Waves 1 to 2 ft. Waves 3 to 7 ft. at top.	Rough. Waves 3 to 8 ft.	Top rough. Waves 7 to 12 ft.
212.0, Apr. 1960, upstream.	−48.0	.....	Amplitude 8, length 200.	.....	Rough. Waves 2 to 4 ft.	Rough. Waves 3 to 10 ft.	Top rough. Waves 2 to 8 ft.
212.0, Apr. 1960, downstream.	−48.0	−10.75	Amplitude 11, length 250	−5.0	Smooth.	Smooth at bottom. Rough at top.	Top rough. Waves 2 to 11 ft.
223.5, Aug. 1958, upstream.	−31.0	.....	Amplitude 13, length 450.	.....	Rough. Waves 3 to 10 ft.	Rough. Waves 2 to 9 ft.	Rough. Waves 8 to 13 ft.
223.5, Aug. 1958, downstream	−32.0	.....	Amplitude 15, length 475.	.....	Rough. Waves 4 to 12 ft.	Rough. Waves 3 to 9 ft.	Rough. Waves 6 to 15 ft.
223.5, Apr. 1960, upstream.	−26.0	.....	Amplitude 10, length 370.	.....	Smooth 1 ft. ripples.	Rough. Waves 3 to 10 ft.	Rough. Waves 3 to 10 ft.
223.5, Apr. 1960, downstream.	−32.0	+2.50	Amplitude 7, length 150.	−5.50	Rough. Waves 3 to 6 ft.	Rough. Waves 2 to 10 ft.	Rough. Waves 3 to 7 ft.
236.0, Aug. 1958, upstream.	−24.0	.....	Amplitude 20, length 350.	.....	Smooth. Waves 1 to 2 ft.	Rough. Waves 3 to 8 ft.	Rough. Waves 9 to 17 ft.
236.0, Aug. 1958, downstream.	−26.5	.....	Amplitude 10, length 100.	.....	Rough. Waves 4 to 7 ft.	Rough. Waves 5 to 7 ft.	Rough. Waves 5 to 10 ft.
236.0, Apr. 1960, upstream.	−34.0	.....	Amplitude 21, length 1,070.	.....	Upstream waves 3 to 14 ft. Downstream waves 1 to 5 ft.	Rough waves upstream 3 to 18 ft. Downstream smooth, steep drop.	Very long undulations 5 to 21 ft.
236.0, Apr. 1960, downstream.	−34.0	−8.75	Amplitude 24, length 1,280.	+8.00	Rough waves 3 to 19 ft.	Rough waves, steep drop.	Very long undulations, 5 to 24 ft.
241.0, Aug. 1958, upstream.	−23.0	.....	Amplitude 8, length 130.	.....	Slightly rough with waves 1 to 3 ft.	Upstream part smooth. Waves 3 to 5 ft.	Rough. Waves 3 to 8 ft.
241.0, Aug. 1958, downstream.	−20.0	.....	Amplitude 15, length 200.	.....	Rough. Waves 4 to 8 ft.	Smooth except upstream part, with waves 3 to 6 ft.	Rough. Waves 5 to 12 ft.
241.0, Apr. 1960, upstream.	−27.0	.....	Amplitude[3] length	.....	Rough. Waves 3 to 6 ft.	Smooth upstream. Sudden drop, then rough waves 3 to 8 ft.	Smooth with slight ripples 1 ft.
241.0, Apr. 1960, downstream.	−25.0	−4.50	Amplitude[3] length	−12.00	Rough. Waves 2 to 7 ft.	Smooth, sudden drop.	Smooth with slight ripples 1 ft.

See footnotes at end of table.

TABLE 3.—*Effect of water temperature on riverbed configuration, Mississippi River, mile 110 to mile 298 A.H.P.,[1] August 4 to 18, 1958, (82° to 83° F.) versus April 21 to 27, 1960 (56° to 61° F.)—Continued*

Crossing location mile, A.H.P.,[1] date, and direction	High point M.L.W.[2]	Average difference in elevation	Wave amplitude and length	Average difference in amplitude	Condition of crossing slopes		Description of top of crossing: nature and location of wave formations and of smooth stretches
					Approach slope	Leaving slope	
	Feet	*Feet*	*Feet*	*Feet*			
251.0, Aug. 1958, upstream.	−12.5	.....	Amplitude 14, length 275.	.....	Regular with waves 2 to 6 ft.	Rough. Waves 3 to 7 ft.	Rough. Waves 5 to 14 ft.
251.0, Aug. 1958, downstream.	−16.5	.....	Amplitude 14, length 250.	.....	Regular with waves 2 to 6 ft.	Rough. Waves 5 to 14 ft.	Rough. Waves 4 to 14 ft.
251.0, Apr. 1960, upstream.	−20.5	.....	Amplitude 12, length 300.	.....	Rough. Waves 2 to 12 ft.	Rough. Waves 2 to 7 ft.	Rough. Waves 4 to 12 ft.
251.0, Apr. 1960, downstream.	−20.0	−5.75	Amplitude 12, length 250.	−2.00	Rough. Waves 3 to 8 ft.	Rough. Waves 2 to 6 ft.	Rough. Waves 4 to 12ft.
258.5, Aug. 1958, upstream.	−20.0	.....	Amplitude 16, length 500.	.....	Regular with small waves 2 to 3 ft.	Rough. Waves 4 to 9 ft.	Rough. Waves 4 to 16 ft.
258.5, Aug. 1958, downstream.	−18.0	.....	Amplitude 15, length 500.	.....	Regular with small waves 2 to 5 ft.	Rough. Waves 4 to 10 ft.	Rough. Waves 4 to 16 ft.
258.5, Apr. 1960, upstream.	−29.0	.....	Amplitude[3] length	.....	Rough. Waves 2 to 7 ft.	Smooth with small waves.	Smooth.
258.5, Apr. 1960, downstream.	−27.0	−9.00	Amplitude[3] length	−15.50	Rough with waves 2 to 6 ft.	Rough. Waves 2 to 5 ft.	Very smooth.
269.5, Aug. 1958, upstream.	−30.5	.....	Amplitude 12, length 250.	.....	Rough. Waves 5 to 12 ft.	Top rough. Waves 3 to 10 ft. Bottom smooth.	Rough. Waves 4 to 12 ft.
269.5, Aug. 1958, downstream.	−39.0	.....	Amplitude 15, length 225.	.....	Rough. Waves 3 to 6 ft.	Top rough, with 3 to 6 ft. waves.	Rough. Waves 4 to 15 ft.
269.5, Apr. 1960, upstream	−40.0	.....	Amplitude 5, length 75.	.....	Rough. Waves 2 to 4 ft. Steep slope.	Rough. Waves 3 to 6 ft.	Small waves 1 to 5 ft.
269.5, Apr. 1960, downstream.	−43.5	−7.00	Amplitude 9, length 250.	−6.50	Rough. Waves 2 to 6 ft.	Top rough with 1 to 6 ft. waves. Bottom smooth.	Rough. Waves 3 to 9 ft.
273.0, Aug. 1958, upstream.	− 9.0	.....	Amplitude 16, length 1,000.	.....	Regular with 2 to 5 ft. long undulations.	Smooth, with long undulations.	Rough upstream end with very long waves 5 to 16 ft. Lower end smooth.
273.0, Aug. 1958, downstream.	− 7.0	.....	Amplitude 21, length 1,250.	.....	Smooth at bottom. Top with 2 to 6 ft. waves.	Waves 3 to 5 ft. and long undulations.	Rough. Very long waves 7 to 17 ft.
273.0, Apr. 1960, upstream.	−15.0	.....	Amplitude[3] length	.....	Smooth upward slope.	Smooth steep slope.	Very smooth at top.
273.0, Apr. 1960, downstream.	−12.5	−5.75	Amplitude[3] length[4]	−18.50	Smooth with ripples 2 ft.	Smooth with long undulations.	Very smooth.
279.0, Aug. 1958, upstream.	−32.5	.....	Amplitude 22, length 350.	.....	Rough. Waves 3 to 7 ft.	Rough. Waves 3 to 6 ft.	Rough. Waves 4 to 15 ft.
279.0, Aug. 1958, downstream.	−29.0	.....	Amplitude 23, length 350.	.....	Rough. Waves 3 to 5 ft.	Smooth.	Rough. Waves 4 to 14 ft.
279.0, Apr. 1960, upstream.	−26.5	.....	Amplitude 8, length 400.	.....	Rough. Waves 3 to 5 ft.	Smooth steep slope.	Rough. Waves 4 to 8 ft.
279.0, Apr. 1960, downstream.	−22.0	+6.50	Amplitude 8, length 540.	−14.50	Rough. Waves 3 to 6 ft.	Smooth steep slope.	Rough. Waves 4 to 8 ft.
286.5, Aug. 1958, upstream.	− 6.0	.....	Amplitude 25, length 425.	.....	Long undulations 3 to 6 ft. with small 1 to 2 ft. waves superimposed.	Long undulations 3 to 5 ft.	Rough. Very large waves 6 to 19 ft.

286.5, Aug. 1958, downstream.	− 7.0	.....	Amplitude 25, length 350.	.....	Long undulations 3 to 6 ft. with small 1 to 2 ft. waves superimposed.	Long undulations 3 to 5 ft.	Rough. Very large waves 5 to 25 ft.
286.5, Apr. 1960, upstream.	−10.0	.....	Amplitude 15, length 2,000	.....	Rough. Waves 2 to 13 ft.	Smooth.	Very long, smooth waves, 7 to 15 ft.
286.5, Apr. 1960, downstream.	− 8.0	−2.50	Amplitude 25, length 2,400.	−5.00	Rough. Waves 3 to 10 ft.	Smooth with slight ripples.	Very smooth surface between 4 long waves, 13 to 25 ft. in amplitude and 1,200 to 2,400 ft. in length.
298.0, Aug. 1958, upstream.	+ 0.5	.....	Amplitude 10, length 450.	.....	Long undulations 1 to 2 ft. waves superimposed.	Rough. Waves 3 to 9 ft.	Rough. Waves 4 to 10 ft.
298.0, Aug. 1958, downstream.	− 2.5	.....	Amplitude 11, length 250.	.....	Long waves 3 to 6 ft.	Smooth and steep.	Rough. Waves 3 to 11 ft.
298.0, Apr. 1960, upstream.	− 8.5	.....	Amplitude 10, length 350.	.....	Smooth, ripples 1 ft.	Smooth and steep.	Smooth with waves 1 to 2 ft.
298.0, Apr. 1960, downstream.	− 9.0	−7.75	Amplitude 9, length 550.	−1.00	Smooth, ripples 1 ft.	Ripples and steep.	One to four foot waves at upstream end. Rough, with waves 1 to 9 ft. on downstream end.
286.5, upstream.	.....	.....	Amplitude 26, length 1,600.	THESE WERE THE LARGEST WAVES FOUND ANYWHERE.			
286.5, downstream.	.....	.....	Amplitude 26, length 2,200.	SIMILAR WAVES FOUND AT THIS LOCATION IN 1958.			

[1] Above head of passes.
[2] Mean low water.
[3] Smooth.
[4] No waves.

TABLE 4.—*Effect of water temperature on riverbed configuration, Mississippi River, mile 110 to mile 298 A.H.P., August 4 to 18, 1958, (82° to 83° F.) versus March 20 to 25, 1961 (55° to 58° F.)*

Crossing location mile A.H.P.,[1] date, and direction	High point M.L.W.[2]	Average difference in elevation	Wave amplitude and length	Average difference in amplitude	Condition of crossing slopes		Description of top of crossing: nature and location of wave formations and of smooth stretches
					Approach slope	Leaving slope	
	Feet	*Feet*	*Feet*	*Feet*			
110.0, Aug. 1958, upstream.	−63.0	.....	Amplitude 6, length 100.	.....	Rough. Waves 4 to 6 ft.	Upper end rough. Waves 3 to 5 ft. Lower part smooth.	Top rough. Waves 3 to 6 ft.
110.0, Aug. 1958, downstream.	−60.0	.....	Amplitude 13, length 240.	.....	Rough. Waves 4 to 13 ft.	Almost smooth throughout	Top rough. Waves 6 to 13 ft.
110.0, Mar. 1961, upstream.	−70.0	.....	([3])	.....	Generally smooth.	Generally smooth.	Large irregularities— amplitude less than 5 ft.
110.0, Mar. 1961, downstream.	−71.0	−9.00	([3])	.....	Ripply.	Generally rough. Steep drop.	Large irregularities—amplitude less than 5 ft.
116.0, Aug. 1958, upstream.	−41.0	.....	Amplitude 7, length 125.	.....	Smooth to near upper end. Then waves 2 to 3 ft.	Upper end waves 3 to 5 ft.	Top rough. Waves 4 to 7 ft.
116.0, Aug. 1958, downstream.	−39.5	.....	Amplitude 8, length 165.	.....	Lower end undulating. Upper end rough waves 4 to 6 ft.	Rough waves 3 to 6 ft.	Top rough. Waves 5 to 8 ft.
116.0, Mar. 1961, upstream.	−49.0	.....	Amplitude 8, length 100.	.....	Waves 2 to 5 ft. Rough.	Rough. Waves 3 to 8 ft.	Rough. Waves 3 to 7 ft.

See footnotes at end of table.

TABLE 4.—*Effect of water temperature on riverbed configuration, Mississippi River, mile 110 to mile 298 A.H.P., August 4 to 18, 1958, (82° to 83° F.) versus March 20 to 25, 1961 (55° to 58° F.)—Continued*

Crossing location mile A.H.P.,[1] date, and direction	High point M.L.W.[2]	Average difference in elevation	Wave amplitude and length	Average difference in amplitude	Condition of crossing slopes		Description of top of crossing: nature and location of wave formations and of smooth stretches
					Approach slope	Leaving slope	
	Feet	*Feet*	*Feet*	*Feet*			
116.0, Mar. 1961, downstream.	−49.0	−8.75	Amplitude 8, length 200.	+0.50	Rough. Waves 3 to 6 ft.	Rough. Small waves 1 to 3 ft.	Rough. Waves 2 to 5 ft.
140.5, Aug. 1958, upstream.	−54.5	.....	Amplitude 5, length 90.	.....	Rough. Waves 3 to 5 ft.	Upper end rough. Waves 2 to 4 ft. Lower end smooth.	Top rough. Waves 3 to 5 ft.
140.5, Aug. 1958, downstream.	−57.0	.....	Amplitude 6, length 170.	.....	Upstream end smooth. Downstream end waves 2 to 5 ft.	Upstream end rough with 2 to 3 ft. waves. Downstream end smooth.	Top rough. Waves 3 to 6 ft.
140.5, Mar. 1961, upstream.	−52.0	.....	Amplitude 9, length 150.	.....	Rough. Waves 4 to 9 ft.	Rough. Waves 4 to 8 ft.	Rough. Waves 3 to 8 ft.
140.5, Mar. 1961, downstream.	−54.0	+2.75	Amplitude 11, length 500.	+4.50	Rough. Waves 2 to 10 ft.	Rough. Waves 5 to 10 ft.	Rough. Waves 5 to 10 ft.
154.0, Aug. 1958, upstream.	−34.0	.....	Amplitude 6, length 130.	.....	Slightly rough. Waves 1 to 2 ft.	Rough. Waves 2 to 4 ft.	Top rough. Waves 2 to 6 ft.
154.0, Aug. 1958, downstream.	−34.5	.....	Amplitude 7, length 125.	.....	Rough. Waves 3 to 7 ft.	Rough. Waves 4 to 6 ft.	Top rough. Waves 3 to 6 ft.
154.0, Mar. 1961, upstream.	−39.0	.....	Amplitude 8, length 100.	.....	Rough. Waves 9 to 100 ft.	Rough. Waves 10 to 150 ft.	Downstream part has a few waves. Top is smooth. Upstream end has a few waves.
154.0, Mar. 1961, downstream.	−36.0	−3.25	Amplitude 22, length 225.	+8.50	Rough. Waves 10 to 20 ft. upstream end.	Rough. Waves 3 to 10 ft.	Rough. Small waves 2 to 4 ft.
163.0, Aug. 1958, upstream.	−44.5	.....	Amplitude 8, length 150.	.....	Smooth upstream half. Waves 2 to 8 ft.	Smooth downstream half. Waves 3 to 5 ft.	Top rough. Waves 3 to 5 ft.
163.0, Aug. 1958, downstream.	−48.0	.....	Amplitude 9, length 110.	.....	Rough. Waves 4 to 9 ft.	Smooth downstream half. Waves 3 to 5 ft.	Top rough. Waves 3 to 6 ft.
163.0, Mar. 1961, upstream.	−47.0	.....	Amplitude 12, length 100.	.....	Rough. Waves 5 to 12 ft.	Gentle rounded waves.	Rough waves 7 x 125 ft. Waves 4 to 8 ft.
163.0, Mar. 1961, downstream.	−53.0	−3.75	Amplitude 15, length 300.	+5.00	Rough. Waves 10 to 15 ft.	Steep and rough. Waves 1 to 5 ft.	Rough. Waves 10 to 15 ft.
167.0, Aug. 1958, upstream.	−54.5	.....	Amplitude 11, length 200.	.....	Smooth. Small waves 2 to 3 ft. at top.	Rough. Waves 6 to 11 ft.	Top rough. Waves 6 to 10 ft.
167.0, Aug. 1958, downstream.	−55.0	.....	Amplitude 12, length 270.	.....	Rough. Small waves 2 to 3 ft. on remnants large waves 4 to 6 ft.	Rough. Waves 5 to 9 ft.	Top rough. Waves 6 to 12 ft.
167.0, Mar. 1961, upstream.	−58.0	.....	Amplitude 7, length 170.	.....	Rough. Waves 3 to 7 ft. Waves 100 ft. long.	Rough. Waves long and flat.	Generally smooth. Very slight undulations—less than 1 ft. amplitude.
167.0, Mar. 1961, downstream.	−47.0	+2.25	Amplitude 15, length 500.	− .50	Generally smooth.	Smooth. Gets rougher at lower end. Waves 5 to 9 ft.	Rough. Large waves.
183.0, Aug. 1958, upstream.	−44.0	.....	Amplitude 14, length 210.	.....	Rough. Waves 2 to 3 ft.	Smooth.	Rough. Waves 6 to 10 ft.
183.0, Aug. 1958, downstream.	−41.0	.....	Amplitude 14, length 375.	.....	Rough. Waves 3 to 6 ft.	Rough. Waves 3 to 6 ft.	Rough. Waves 6 to 14 ft.

183.0, Mar. 1961, upstream.	−46.0	.....	Amplitude 12, length 125.	.....	Generally smooth. Long and flat. Very few waves. Waves 4 to 7 ft.	Rough. Sloping and erratic. Waves 15 x 200 ft.	Generally smooth. A little rough on each edge.
283.0, Mar. 1961, downstream.	−41.0	−1.00	Amplitude 14, length 220.	−1.00	Rough. Upstream end gets smooth. Waves 1 to 6 ft.	Rough. Waves 5 to 14 ft.	Smooth.
190.5, Aug. 1958, upstream.	−34.5	.....	Amplitude 10, length 160.	.....	Smooth upstream half. Waves downstream 2 to 6 ft.	Rough. Waves 5 to 8 ft.	Top rough. Waves 6 to 10 ft.
190.5, Aug. 1958, downstream.	−36.0	.....	Amplitude 13, length 180.	.....	Smooth at bottom. Waves 2 to 6 ft.	Rough. Waves 3 to 6 ft.	Top rough. Waves 4 to 13 ft.
190.5, Mar. 1961, upstream.	−44.0	.....	Amplitude 15, length 400.	.....	Rough. Small waves 2 to 5 ft.	Rough. Waves 6 to 15 ft.	Rough. Large waves 8 x 300 ft.
190.5, Mar. 1961, downstream.	−41.0	−7.25	Amplitude 18, length 400.	+5.00	Rough. Waves 5 to 10 ft.	Rough. Waves 2 to 8 ft.	Rough. Large waves.
197.5, Aug. 1958, upstream.	−37.5	.....	Amplitude 9, length 160.	.....	Generally smooth with waves 1 to 2 ft.	Rough. Waves 2 to 5 ft.	Top rough. Waves 5 to 9 ft.
197.5, Aug. 1958, downstream.	−36.0	.....	Amplitude 10, length 190.	.....	Smooth at bottom. Rough with waves.	Rough. Waves.	Top rough. Waves 6 to 10 ft.
197.5, Mar. 1961, upstream.	−43.0	.....	Amplitude 8, length 300.	.....	Rough. Well defined undulations—2 to 8 ft. Mostly about 4 ft.	Rough. Waves 1 to 7 ft.	Short. Generally smooth top. About 1,000 ft. long.
197.5, Mar. 1961, downstream.	−40.0	−4.75	Amplitude 10, length 250.	−0.50	Rough. 2 to 10 ft. waves.	Rough. Waves 5 to 10 ft.	Rough. Large waves.
204.0, Aug. 1958, upstream.	−33.0	.....	Amplitude 12, length 170.	.....	Rough. Waves 3 to 12 ft.	Smooth at bottom. Waves 2 to 5 ft.	Top rough. Waves 6 to 12 ft.
204.0, Aug. 1958, downstream.	−34.5	.....	Amplitude 10, length 190.	.....	Rough. Waves 3 to 9 ft.	Smooth.	Top rough. Waves 5 to 10 ft.
204.0, Mar. 1961, upstream.	−44.0	.....	Amplitude 4, length 200.	.....	Rough. No well developed wave system.	Rough with terrace effect.	Fairly smooth—no waves.
204.0, Mar. 1961, downstream.	−50.0	−13.25	Amplitude 11, length 300.	−3.50	Smooth. Gentle undulations.	Rough. Steep drop.	Smooth—gentle undulations.
212.0, Aug. 1958, upstream.	−38.0	.....	Amplitude 12, length 200.	.....	Regular at bottom, waves 1 to 2 ft. 3 to 6 ft. waves at top.	Rough. Waves 3 to 8 ft.	Top rough. Waves 5 to 12 ft.
212.0, Aug. 1958, downstream.	−36.5	.....	Amplitude 17, length 250.	.....	Regular at bottom, waves 1 to 2 ft. 3 to 7 ft. waves at top.	Rough. Waves 3 to 8 ft.	Top rough. Waves 7 to 12 ft.
212.0, Mar. 1961, upstream.	−38.0	.....	Amplitude 10, length 200.	.....	Rough. Waves 5 to 10 ft.	Rough. Not well defined.	Short top with steep approaches in the form of large waves 1,600 ft. long.
212.0, Mar. 1961, downstream.	−46.0	−4.75	Amplitude 8, length 200.	−5.50	Rough. Waves 2 to 8 ft.	Rough. Waves 2 to 5 ft.	Smooth. Gentle undulations.
223.5, Aug. 1958, upstream.	−31.0	.....	Amplitude 13, length 450.	.....	Rough. Waves 3 to 10 ft.	Rough. Waves 2 to 9 ft.	Rough. Waves 8 to 13 ft.
223.5, Aug. 1958, downstream.	−32.0	.....	Amplitude 15, length 475.	.....	Rough. Waves 4 to 12 ft.	Rough. Waves 3 to 9 ft.	Rough. Waves 6 to 15 ft.
223.5, Mar. 1961. upstream.	−53.0	.....	Amplitude 15, length 600.	.....	Rough. Waves 2 to 10 ft.	Rough. Big waves over 500 wide x 15 ft. high.	Flat. Smooth top.
223.5, Mar. 1961, downstream.	−51.0	−20.50	Amplitude 14, length 350.	+ .50	Rough. Waves 2 to 5 ft.	Rough. Big waves.	Flat. Smooth.
236.0, Aug. 1958, upstream.	−24.0	.....	Amplitude 20, length 350.	.....	Smooth with waves 1 to 2 ft.	Rough. Waves 3 to 8 ft.	Rough. Waves 9 to 17 ft.

See footnotes at end of table.

TABLE 4.—*Effect of water temperature on riverbed configuration, Mississippi River, mile 110 to mile 298 A.H.P., August 4 to 18, 1958, (82° to 83° F.) versus March 20 to 25, 1961 (55° to 58° F.)—Continued*

Crossing location mile, A.H.P.,[1] date, and direction	High point M.L.W.[2]	Average difference in elevation	Wave amplitude and length	Average difference in amplitude	Condition of crossing slopes		Description of top of crossing: nature and location of wave formations and of smooth stretches
					Approach slope	Leaving slope	
	Feet	*Feet*	*Feet*	*Feet*			
236.0, Aug. 1958, downstream.	−26.5	.....	Amplitude 10, length 100.	.....	Rough. Waves 4 to 7 ft.	Rough. Waves 5 to 7 ft.	Rough. Waves 5 to 10 ft.
236.0, Mar. 1961, upstream.	−46.0	.....	Amplitude 5, length 150.	.....	Rough. Varying to smooth and ripply. Waves 5 x 150 ft.	Smooth and ripply. No wave form. Steep drop.	Smooth with slight ripples 1 ft. or less.
236.0, Mar. 1961, downstream.	−47.0	−21.25	Amplitude 4, length 150.	−10.5	Rough. 2 to 4 ft. waves.	Smooth. Steel drop.	Smooth. Gentle undulations.
241.0, Aug. 1958, upstream.	−23.0	.....	Amplitude 8, length 130.	.....	Slightly rough with waves 1 to 3 ft.	Upper part smooth. Waves 3 to 5 ft.	Rough Waves 3 to 8 ft.
241.0, Aug. 1958, downstream.	−20.0	.....	Amplitude 15, length 200.	.....	Rough. Waves 4 to 8 ft.	Smooth except upper part with waves 3 to 6 ft.	Rough. Waves 5 to 12 ft.
241.0, Mar. 1961, upstream.	−35.0	.....	Amplitude 10, length 125.	.....	Very rough. Well developed waves 5 to 10 ft.	Slightly rough. Steep drop .	Short and less rough than approach. No wave form.
241.0, Mar. 1961, downstream.	−37.0	−14.50	Amplitude 10, length 150.	−1.50	Rough. Waves 10 ft.	Smooth. Steep drop. Gentle undulations.	Smooth on top.
251.0, Aug. 1958, upstream.	−12.5	.....	Amplitude 14, length 275.	.....	Regular with waves 2 to 6 ft.	Rough. Waves 3 to 7 ft.	Rough. Waves 5 to 14 ft.
251.0, Aug. 1958, downstream.	−16.5	.....	Amplitude 14, length 250.	.....	Regular with waves 2 to 6 ft.	Rough. Waves 5 to 14 ft.	Rough. Waves 4 to 14 ft.
251.0, Mar. 1961, upstream.	−30.5	.....	Amplitude 15, length 125.	.....	Very rough. Waves 2 to 8 ft.	Rough. Waves 15 x 125 ft.	Short and smooth. Rough on both approaches. Downstream waves 3 to 9 ft.
251.0, Mar. 1961, downstream.	−32.0	−16.75	Amplitude 13, length 300.	± .00	Rough. Small waves upstream part. Larger waves downstream part.	Rough. Large waves 12 x 300 ft.	Rough. Large waves.
258.5, Aug. 1958, upstream.	−20.0	.....	Amplitude 16, length 500.	.....	Regular with small waves 2 to 3 ft.	Rough. Waves 4 to 9 ft.	Rough. Waves 4 to 16 ft.
258.5, Aug. 1958, downstream.	−18.0	.....	Amplitude 15, length 500.	.....	Regular with small waves 2 to 5 ft.	Rough. Waves 4 to 10 ft.	Rough. Waves 4 to 16 ft.
258.5, Mar. 1961, upstream.	−28.0	.....	Amplitude 18, length 400	.....	Very long, varying from flat to very rough. Waves 5 to 15 ft.	Fairly smooth. Undulations. Rough with steep drop at end.	Very large waves 17 x 700 ft.
258.5, Mar. 1961, downstream.	−25.5	−7.75	Amplitude 16, length 300.	+1.50	Very long. Rough. Waves very large.	Fairly smooth undulations. Steep drop at end.	Rough. Large waves 15 x 300 ft.
269.5, Aug. 1958, upstream.	−30.5	.....	Amplitude 12, length 250.	.....	Rough. Waves 5 to 12 ft.	Top rough with 3 to 10 ft. waves. Bottom smooth.	Rough. Waves 4 to 12 ft.
269.5, Aug. 1958, downstream.	−39.0	.....	Amplitude 15, length 225.	.....	Rough. Waves 3 to 6 ft.	Top rough with 3 to 6 ft. waves.	Rough. Waves 4 to 15 ft.
269.5, Mar. 1961, upstream.	−49.5	.....	Amplitude 15, length 300.	.....	Rough. Small waves 2 to 5 ft.	Rough. Waves 5 to 15 ft.	Rough. Waves 5 to 7 ft.
269.5, Mar. 1961, downstream.	−46.0	−13.00	Amplitude 9, length 275.	−1.50	Rough. Small waves 1 to 3 ft.	Smooth. Generally undulating.	Rough. Waves 3 to 9 ft.

273.0, Aug. 1958, upstream.	− 9.0	.....	Amplitude 16, length 1,000.	.....	Regular with 2 to 5 ft. long undulations.	Smooth with long undulations.	Rough upper end with very long waves 5 to 16 ft. Lower end smooth.
273.0, Aug. 1958, downstream.	− 7.0	.....	Amplitude 21, length 1,250.	.....	Smooth at bottom. Top with 2 to 6 ft. waves.	Waves 3 to 5 ft. and long undulations.	Rough. Very long waves 7 to 17 ft.
273.0, Mar. 1961, upstream.	−26.5	.....	Amplitude 7, length 200.	.....	Rough. Waves 3 to 8 ft.	Smooth.	Smooth.
273.0, Mar. 1961, downstream.	−29.0	−19.75	Amplitude 15, length 300.	−7.50	Rough. Waves 3 to 8 ft. Upper part generally smooth.	Rough. Waves 5 to 15 ft.	Smooth.
279.0, Aug. 1958, upstream.	−32.5	.....	Amplitude 22, length 350.	.....	Rough. Waves 3 to 7 ft.	Rough. Waves 3 to 6 ft.	Rough. Waves 4 to 15 ft.
279.0, Aug. 1958, downstream.	−29.0	.....	Amplitude 23, length 350.	.....	Rough. Waves 3 to 5 ft.	Smooth.	Rough. Waves 4 to 14 ft.
279.0, Mar. 1961, upstream.	−39.0	.....	Amplitude 15, length 400.	.....	Rough. Waves 9 ft.	Smooth undulations.	Rough. Large waves 15 x 400 ft.
279.0, Mar. 1961, downstream.	−35.0	−6.25	Amplitude 15, length 550.	−7.50	Rough. Waves 3 to 8 ft.	Smooth upper part. Lower part rough. Waves 2 to 4 ft.	Rough. Smooth upper part. Lower part rough.
286.5, Aug. 1958, upstream.	− 6.0	.....	Amplitude 25, length 425.	.....	Long undulations 3 to 6 ft. with small waves 1 to 2 ft. superimposed.	Long undulations 3 to 5 ft.	Rough. Very large waves 6 to 19 ft.
286.5, Aug. 1958, downstream.	− 7.0	.....	Amplitude 25, length 350.	.....	Long undulations 3 to 6 ft. with small waves 1 to 2 ft. superimposed.	Long undulations 3 to 5 ft.	Rough. Very large waves 5 to 25 ft.
286.5, Mar. 1961, upstream.	−31.0	.....	Amplitude 6, length 100.	.....	Rough. Small waves 2 to 4 ft.	Waves at lower end, 3 to 6 ft.	Top generally smooth.
286.5, Mar. 1961, downstream.	−30.0	−24.0	Amplitude 8, length 200.	−18.00	Rough. Large waves 4 to 8 ft.	Smooth at upstream end. Very rough downstream end. Waves 3 to 6 ft.	Upstream part rough. Downstream part smooth.
298.0, Aug. 1958, upstream.	− .5	.....	Amplitude 10, length 450.	.....	Long undulations 1 to 2 ft. with waves superimposed.	Rough. Waves 3 to 9 ft.	Rough. Waves 4 to 10 ft.
298.0, Aug. 1958, downstream.	− 2.5	.....	Amplitude 11, length 250.	.....	Long waves 3 to 6 ft.	Smooth and steep.	Rough. Waves 3 to 11 ft.
298.0, Mar. 1961, upstream.	−18.0	.....	Amplitude 8, length 125.	.....	Rough. Small waves 1 to 3 ft.	Rough. Steep. Larger waves at bottom. Waves 1 to 4 ft.	Rough. Waves 3 to 8 ft.
298.0, Mar. 1961, downstream.	−25.0	−20.50	Amplitude 11, length 150.	−1.00	Rough. Waves 3 to 11 ft.	Rough. Small waves at upper end. General undulations.	Rough. Waves 2 to 6 ft.

[1] Above head of passes.
[2] Mean low water.
[3] No wave system.

TABLE 5.—*Effect of water temperature on riverbed configuration, Mississippi River, mile 110 to mile 298 A.H.P., August 4 to 18, 1958, (82° to 83° F.) versus May 8 to 12, 1961 (64° to 66° F.)*

Crossing location mile, A.H.P.,[1] date, and direction	High point M.L.W.[2]	Average difference in elevation	Wave amplitude and length	Average difference in amplitude	Condition of crossing slopes		Description of top of crossing: nature and location of wave formations and of smooth stretches
					Approach slope	Leaving slope	
	Feet	*Feet*	*Feet*	*Feet*			
110.0, Aug. 1958, upstream.	−63.0	.....	Amplitude 6, length 100.	.....	Rough. Waves 4 to 6 ft.	Upper end rough. Waves 3 to 5 ft. Lower part smooth.	Top rough. Waves 3 to 6 ft.
110.0, Aug. 1958, downstream.	−60.0	.....	Amplitude 13, length 240.	.....	Rough. Waves 6 to 13 ft.	Almost smooth throughout.	Top rough. Waves 6 to 13 ft.
110.0, May 1961, upstream.	−70.0	.....	Amplitude 15, length 300.	.....	Rough. Waves 2 to 4 ft.	Rough. Waves 10 to 15 ft.	Rough. Waves 2 to 15 ft.
110.0, May 1961, downstream.	−72.0	−9.50	Amplitude 7, length 100.	+1.50	Rough. Waves 1 to 3 ft.	Rough. Waves 4 to 7 ft.	Smooth—ripply.
116.0, Aug. 1958, upstream.	−41.0	.....	Amplitude 7, length 125.	.....	Smooth to near upper end. Then waves 2 to 3 ft.	Upper end waves 3 to 5 ft.	Top rough. Waves 4 to 7 ft.
116.0, Aug. 1958, downstream	−39.5	.....	Amplitude 8, length 165.	.....	Lower end undulating. Upper end rough Waves 4 to 6 ft.	Rough. Waves 3 to 6 ft.	Top rough. Waves 5 to 8 ft.
116.0, May 1961, upstream.	−36.0	.....	Amplitude 12, length 300.	.....	Rough. Waves 1 to 8 ft.	Upper end smooth. Lower end rough.	Rough. Waves 3 to 6 ft.
116.0, May 1961, downstream.	−36.0	+4.25	Amplitude 15, length 300.	+6.00	Ripples to rough. Waves 3 to 10 ft.	Ripples.	Rough. Waves 3 to 15 ft.
140.5, Aug. 1958, upstream.	−54.5	.....	Amplitude 5, length 90.	.....	Rough. Waves 3 to 5 ft.	Upper end rough. Waves 2 to 4 ft. Lower end smooth.	Top rough. Waves 3 to 5 ft.
140.5, Aug. 1958, downstream.	−57.0	.....	Amplitude 6, length 170.	.....	Upstream end smooth. Downstream end waves 2 to 5 ft.	Upstream end rough with 2 to 3 ft. waves. Downstream end smooth.	Top rough. Waves 3 to 6 ft.
140.5, May 1961, upstream.	−48.0	.....	Amplitude 17, length 500.	.....	Rough. Waves 2 to 8 ft.	Rough. Waves 1 to 6 ft.	Rough. Waves 5 to 15 ft.
140.5, May 1961, downstream.	−49.0	+7.25	Amplitude 10, length 300.	+8.00	Rough. Waves 2 to 10 ft.	Rough. Waves 2 to 10 ft.	Rough. Waves 2 to 10 ft.
154.0, Aug. 1958, upstream.	−34.0	.....	Amplitude 6, length 130.	.....	Slightly rough. Waves 1 to 2 ft.	Rough. Waves 2 to 4 ft.	Top rough. Waves 2 to 6 ft.
154.0, Aug. 1958, downstream	−34.5	.....	Amplitude 7, length 125.	.....	Rough. Waves 3 to 7 ft.	Rough. Waves 4 to 6 ft.	Top rough. Waves 3 to 6 ft.
154.0, May 1961, upstream.	−27.0	.....	Amplitude 12, length 250.	.....	Rough. Ripply.	Rough at lower end. Waves 2 to 4 ft. Short and steep.	Rough. Waves 2 to 12 ft.
154.0, May 1961, downstream.	−25.0	+8.25	Amplitude 13, length 1,300.	+6.00	Ripply.	Rough. Waves 2 to 5 ft.	Rough. Waves 2 to 10 ft.
163.0, Aug. 1958, upstream.	−44.5	.....	Amplitude 8, length 150.	.....	Smooth upstream half. Waves 2 to 8 ft.	Smooth downstream half. Waves 3 to 5 ft.	Top rough. Waves 3 to 5 ft.
163.0, Aug. 1958, downstream.	−48.0	.....	Amplitude 9, length 110.	.....	Rough. Waves 4 to 9 ft.	Smooth downstream half. Waves 3 to 5 ft.	Rop rough. Waves 3 to 6 ft.
163.0, May 1961, upstream.	−42.0	.....	Amplitude 8, length 250.	.....	Rough. Waves 3 to 5 ft.	Rough. Irregular. Short and steep.	Rough. Waves 3 to 15 ft.
163.0, May 1961, downstream	−43.0	+3.75	Amplitude 8, length 300.	−0.50	Rough. Waves 3 to 6 ft.	Smooth. Steep.	Rough. Waves 3 to 10 ft.

167.0, Aug. 1958, upstream.	−54.5	.....	Amplitude 11, length 200.	.....	Smooth. Small waves 2 to 3 ft. at top.	Rough. Waves 6 to 11 ft.	Top rough. Waves 6 to 10 ft.
167.0, Aug. 1958, downstream.	−55.0	.....	Amplitude 12, length 270.	.....	Rough. Small waves 2 to 3 ft. on remnants large waves 4 to 6 ft.	Rough. Waves 5 to 9 ft.	Top rough. Waves 6 to 12 ft.
167.0, May 1961, upstream.	−46.0	.....	Amplitude 12, length 500.	.....	Rough. Small waves 2 to 3 ft.	Rough. Very large waves 12 ft.	Rough. Waves 2 to 8 ft.
167.0, May 1961, downstream.	−48.0	+7.75	Amplitude 20, length 500.	+4.50	Rough. Large waves 5 to 15 ft.	Ripply and steep.	Rough. Waves 7 to 20 ft.
183.0, Aug. 1958, upstream.	−44.0	.....	Amplitude 14, length 210.	.....	Rough. Waves 2 to 3 ft.	Smooth.	Rough. Waves 6 to 10 ft.
183.0, Aug. 1958, downstream.	−41.0	.....	Amplitude 14, length 375.	.....	Rough. Waves 3 to 6 ft.	Rough. Waves 3 to 6 ft.	Rough. Waves 6 to 14 ft.
183.0, May 1961, upstream.	−44.0	.....	Amplitude 15, length 650.	.....	Long—rough.	Smooth. Gentle undulations.	Gentle undulations.
183.0, May 1961, downstream.	−47.5	−3.25	Amplitude 25, length 750.	+6.00	Rough. Large waves.	Rough. Waves 2 to 4 ft.	Smooth.
190.5, Aug. 1958, upstream.	−34.5	.....	Amplitude 10, length 160.	.....	Smooth upstream half. Waves downstream 2 to 6 ft.	Rough. Waves 5 to 8 ft.	Top rough. Waves 6 to 10 ft.
190.5, Aug. 1958, downstream.	−36.0	.....	Amplitude 13, length 180.	.....	Smooth at bottom. Waves 2 to 6 ft.	Rough. Waves 3 to 6 ft.	Top rough. Waves 4 to 13 ft.
190.5, May 1961, upstream.	−32.0	.....	Amplitude 9, length 300.	.....	Long, gently undulating.	Rough. Waves 2 to 12 ft.	Rough. Waves 2 to 8 ft.
190.5, May 1961, downstream.	−32.0	+3.25	Amplitude 18, length 500.	+2.00	Rough. Waves 2 to 10 ft.	Long. Smooth and undulating.	Rough. Large waves.
197.5, Aug. 1958, upstream.	−37.5	.....	Amplitude 9, length 160.	.....	Generally smooth with waves 1 to 2 ft.	Rough. Waves 2 to 5 ft.	Top rough. Waves 5 to 9 ft.
197.5, Aug. 1958, downstream.	−36.0	.....	Amplitude 10, length 190.	.....	Smooth at bottom. Rough with waves.	Rough. Waves.	Top rough. Waves 6 to 10 ft.
197.5, May 1961, upstream.	−30.0	.....	Amplitude 20, length 1,100.	.....	Rough. Waves 2 to 10 ft.	Irregular, steep, no order to surface.	Top rough. Waves 1 to 13 ft.
197.5, May 1961, downstream.	−29.0	+7.25	Amplitude 20, length 750.	+10.5	Rough. Waves 2 to 15 ft.	Rough. Waves 2 to 4 ft.	Rough. Large waves 5 to 15 ft.
204.0, Aug. 1958, upstream.	−33.0	.....	Amplitude 12, length 170.	.....	Rough. Waves 3 to 12 ft.	Smooth at bottom. Waves 2 to 5 ft.	Top rough. Waves 6 to 12 ft.
204.0, Aug. 1958, downstream.	−34.5	.....	Amplitude 10, length 190.	.....	Rough. Waves 3 to 9 ft.	Smooth.	Top rough. Waves 5 to 10 ft.
204.0, May 1961, upstream.	−31.0	.....	Amplitude 14, length 800.	.....	Rough. Ripples at start. Waves 7 to 12 ft. near top.	Steep. Undulations.	Rough. Waves 5 to 12 ft.
204.0, May 1961, downstream.	−33.0	+1.75	Amplitude 14, length 500.	+3.00	Rough. Waves 5 to 15 ft.	Ripply.	Rough. Waves 5 to 15 ft.
212.0, Aug. 1958, upstream.	−38.0	.....	Amplitude 12, length 200.	.....	Regular at bottom. Waves 1 to 2 ft. 3 to 6 ft. waves at top.	Rough. Waves 3 to 8 ft.	Top rough. Waves 5 to 12 ft.
212.0, Aug. 1958, downstream.	−36.5	.....	Amplitude 17, length 250.	.....	Regular at bottom. Waves 1 to 2 ft. 3 to 7 ft. waves at top.	Rough. Waves 3 to 8 ft.	Top rough. Waves 7 to 12 ft.
212.0, May 1961, upstream.	−29.0	.....	Amplitude 23, length 1,100.	.....	Rough. Waves 4 to 15 ft.	Rough. Waves 1 to 11 ft.	Few large smooth waves 6 to 12 ft.
212.0, May 1961, downstream.	−27.0	+9.25	Amplitude 24, length 1,000	+9.00	Rough. Waves 2 to 15 ft.	Rough. Waves 1 to 10 ft.	Rough. Large waves 10 to 20 ft.

See footnotes at end of table.

TABLE 5.—*Effect of water temperature on riverbed configuration, Mississippi River, mile 110 to mile 298 A.H.P., August 4 to 18, 1958, (82° to 83° F.) versus May 8 to 12, 1961 (64° to 66° F.)—Continued*

Crossing location mile, A.H.P.,[1] date, and direction	High point M.L.W.[2]	Average difference in elevation	Wave amplitude and length	Average difference in amplitude	Condition of crossing slopes		Description of top of crossing: nature and location of wave formations and of smooth stretches
					Approach slope	Leaving slope	
	Feet	*Feet*	*Feet*	*Feet*			
223.5, Aug. 1958, upstream.	−31.0	.....	Amplitude 13, length 450.	.....	Rough. Waves 3 to 10 ft.	Rough. Waves 2 to 9 ft.	Rough. Waves 8 to 13 ft.
223.5, Aug. 1958, downstream.	−32.0	.....	Amplitude 15, length 475.	.....	Rough. Waves 4 to 12 ft.	Rough. Waves 3 to 9 ft.	Rough. Waves 6 to 15 ft.
223.5, May 1961, upstream.	−26.0	.....	Amplitude 12, length 900.	.....	Rough. Large waves 5 to 15 ft.	Undulating. Smooth.	Rough. Large waves.
223.5, May 1961, downstream.	−36.5	+ .25	Amplitude 18, length 300.	+1.00	Rough. Large waves 2 to 15 ft.	Ripply.	Smooth. Large waves.
236.0, Aug. 1958, upstream.	−24.0	.....	Amplitude 20, length 350.	.....	Smooth with waves 1 to 2 ft.	Rough. Waves 3 to 8 ft.	Rough. Waves 9 to 17 ft.
236.0, Aug. 1958, downstream.	−26.5	.....	Amplitude 10, length 100.	.....	Rough. Waves 4 to 7 ft.	Rough. Waves 5 to 7 ft.	Rough. Waves 5 to 10 ft.
236.0, May 1961, upstream.	−28.0	.....	Amplitude 14, length 350.	.....	Rough. Waves 1 to 10 ft.	Undulating. Steep drop.	Rough. Waves 7 to 14 ft.
236.0, May 1961, downstream.	−32.0	−4.75	Amplitude 10, length 400.	−3.00	Rough. Waves 2 to 10 ft.	Rough. Steep. Large waves.	Rough. Large waves 4 to 12 ft.
241.0, Aug. 1958, upstream.	−23.0	.....	Amplitude 8, length 130.	.....	Slightly rough with waves 1 to 3 ft.	Upper part smooth. Waves 3 to 5 ft.	Rough. Waves 3 to 8 ft.
241.0, Aug. 1958, downstream.	−20.0	.....	Amplitude 15, length 200.	.....	Rough. Waves 4 to 8 ft.	Smooth except upper part with waves 3 to 6 ft.	Rough. Waves 5 to 12 ft.
241.0, May 1961, upstream.	−36.0	.....	Amplitude 8, length 100.	.....	Rough. Ripply.	Rough. Steep descent.	Rough. Waves 2 to 8 ft.
241.0, May 1961, downstream.	−34.0	−13.50	Amplitude 10, length 250.	−2.50	Ripples to rough. Waves 1 to 10 ft.	Rough. Steep descent.	Rough. Waves 3 to 10 ft.
251.0, Aug. 1958, upstream.	−12.5	.....	Amplitude 14, length 275.	.....	Regular with waves 2 to 6 ft.	Rough. Waves 3 to 7 ft.	Rough. Waves 5 to 14 ft.
251.0, Aug. 1958, downstream.	−16.5	.....	Amplitude 14, length 250.	.....	Regular with waves 2 to 6 ft.	Rough. Waves 5 to 14 ft.	Rough. Waves 4 to 14 ft.
251.0, May 1961, upstream.	−12.0	.....	Amplitude 10, length 150.	.....	Approach rough. Large waves 2 to 10 ft.	Smooth. Gentle undulations.	Rough. Frequent undulations.
251.0, May 1961, downstream.	−15.0	+1.00	Amplitude 13, length 500.	−2.50	Ripply to rough.	Undulating. Smooth.	Rough. Waves 2 to 4 ft.
258.5, Aug. 1958, upstream.	−20.0	.....	Amplitude 16, length 500.	.....	Regular with small waves 2 to 3 ft.	Rough. Waves 4 to 9 ft.	Rough. Waves 4 to 16 ft.
258.5, Aug. 1958, downstream.	−18.0	.....	Amplitude 15, length 500.	.....	Regular with small waves 2 to 5 ft.	Rough. Waves 4 to 10 ft.	Rough. Waves 4 to 16 ft.
258.5, May 1961, upstream.	−13.0	.....	Amplitude 13, length 200.	.....	Rough. Waves 2 to 8 ft.	Smooth. Undulating.	Rough. Waves 2 to 7 ft.
258.5, May 1961, downstream.	−32.0	−12.50	Amplitude 15, length 300.	−1.50	Rough. Waves 2 to 7 ft.	Smooth. Undulating.	Smooth. Undulating.
269.5, Aug. 1958, upstream.	−30.5	.....	Amplitude 12, length 250.	.....	Rough. Waves 5 to 12 ft.	Top rough with 3 to 10 ft. waves. Bottom smooth.	Rough. Waves 4 to 12 ft.
269.5, Aug. 1958, downstream.	−39.0	.....	Amplitude 15, length 225.	.....	Rough. Waves 3 to 6 ft.	Top rough with 3 to 6 ft. waves.	Rough. Waves 4 to 15 ft.

[illegible], May 1961, upstream.	−31.0	.....	Amplitude 13, length 250.	.....	Steep. Rough. Irregular.	Rough. Waves 5 to 13 ft.	Rough. Waves 2 to 5 ft.
269.5, May 1961, downstream.	−53.0	−17.25	Amplitude 18, length 200.	+2.00	Steep. Rough. Irregular.	Rough. Waves 5 to 15 ft.	Rough. Waves 2 to 5 ft.
273.0, Aug. 1958, upstream.	− 9.0	.....	Amplitude 16, length 1,000.	.....	Regular with 2 to 5 ft. long undulations.	Smooth with long undulations.	Rough upper end with very long waves 5 to 16 ft. Lower end smooth.
273.0, Aug. 1958, downstream.	− 7.0	.....	Amplitude 21, length 1,250.	.....	Smooth at bottom. Top with 2 to 6 ft. waves.	Waves 3 to 5 ft. and long undulations.	Rough. Very long waves 7 to 17 ft.
273.0, May 1961, upstream.	−18.0	.....	Amplitude 18, length 700.	.....	Ripply. Constant ascent.	Rough. Waves 2 to 5 ft.	Rough. Large waves.
273.0, May 1961, downstream.	−15.0	−8.50	Amplitude 10, length 150.	−4.50	Ripply.	Rough. Waves 2 to 10 ft.	Rough. Waves 2 to 10 ft.
279.0, Aug. 1958, upstream.	−32.5	.....	Amplitude 22, length 350.	.....	Rough. Waves 3 to 7 ft.	Rough. Waves 3 to 6 ft.	Rough. Waves 4 to 15 ft.
279.0, Aug. 1958, downstream.	−29.0	.....	Amplitude 23, length 350.	.....	Rough. Waves 3 to 5 ft.	Smooth.	Rough. Waves 4 to 14 ft.
279.0, May 1961, upstream.	−30.0	.....	Amplitude 20, length 400.	.....	Rough. Large waves.	Rough. Ripply. Steep. Waves 1 to 18 ft.	Rough. Waves 5 to 13 ft.
279.0, May 1961, downstream.	−34.5	−1.50	Amplitude 20, length 500.	−2.50	Rough. Waves 2 to 15 ft.	Smooth. Undulating.	Rough. Waves 5 to 20 ft.
286.5, Aug. 1958, upstream.	− 6.0	.....	Amplitude 25, length 425.	.....	Long undulations 3 to 6 ft. with small waves 1 to 2 ft. superimposed.	Long undulations 3 to 5 ft.	Rough. Very large waves 6 to 19 ft.
286.5, Aug. 1958, downstream.	− 7.0	.....	Amplitude 25, length 350.	.....	Long undulations 3 to 6 ft. with small waves 1 to 2 ft. superimposed.	Long undulations 3 to 5 ft.	Rough. Very large waves 5 to 25 ft.
286.5, May 1961, upstream.	−22.0	.....	Amplitude 13, length 1,000.	.....	Rough. Small waves 2 to 8 ft.	Rough. Irregular pattern.	Rough. Large waves.
286.5, May 1961, downstream.	−23.0	−16.0	Amplitude 17, length 800.	−10.00	Rough. Waves 17 x 400 ft.	Gentle. Undulating.	Rough. Large waves.
298.0, Aug. 1958, upstream.	− .5	.....	Amplitude 10, length 450.	.....	Long undulations 1 to 2 ft. with waves superimposed.	Rough. Waves 3 to 9 ft.	Rough. Waves 4 to 10 ft.
298.0, Aug. 1958, downstream.	− 2.5	.....	Amplitude 11, length 250.	.....	Long waves 3 to 6 ft.	Smooth and steep.	Rough. Waves 3 to 11 ft.
298.0, May 1961, upstream.	−17.0	.....	(3)	.....	Ripply. Gentle undulations.	Ripply. Gentle undulations.	Ripply. Gentle undulations.
298.0, May 1961, downstream.	−16.0	−15.50	(3)	.....	Ripply. Gentle undulations.	Ripply. Gentle undulations.	Ripply. Gentle undulations.

[1] Above head of passes.
[2] Mean low water.
[3] No wave pattern.

TABLE 6.—*Effect of water temperature on riverbed configuration, Mississippi River, mile 110 to mile 298 A.H.P., August 4 to 18, 1958, (82° to 83° F.) versus June 1 to 6, 1961 (70° to 73° F.)*

Crossing location mile, A.H.P.,[1] date, and direction	High point M.L.W.[2]	Average difference in elevation	Wave amplitude and length	Average difference in amplitude	Condition of crossing slopes		Description of top of crossing: nature and location of wave formations and of smooth stretches
					Approach slope	Leaving slope	
	Feet	*Feet*	*Feet*	*Feet*			
110.0, Aug. 1958, upstream.	−63.0	.....	Amplitude 6, length 100.	.....	Rough. Waves 4 to 6 ft.	Upper end rough. Waves 3 to 5 ft. Lower part smooth.	Top rough. Waves 3 to 6 ft.
110.0, Aug. 1958, downstream.	−60.0	.....	Amplitude 13, length 240.	.....	Rough. Waves 2 to 13 ft.	Almost smooth throughout.	Top rough. Waves 6 to 13 ft.
110.0, June 1961, upstream.	−63.0	.....	Amplitude 22, length 394.	.....	Rough. Waves 2 to 10 ft. Steep approach slope.	Rough. Waves 1 to 6 ft.	Rough. Waves 5 to 22 ft.
110.0, June 1961, downstream.	−67.0	−3.50	Amplitude 25, length 600.	+14.00	Rough. Waves 5 to 25 ft.	Rough. Waves 3 to 6 ft.	Smooth.
116.0, Aug. 1958, upstream.	−41.0	.....	Amplitude 7, length 125.	.....	Smooth to near upper end. Then waves 2 to 3 ft.	Upper end waves 3 to 5 ft.	Top rough. Waves 4 to 7 ft.
116.0, Aug. 1958, downstream.	−39.5	.....	Amplitude 8, length 165.	.....	Lower end undulating. Upper end rough. Waves 4 to 6 ft.	Rough. Waves 3 to 6 ft.	Top rough. Waves 5 to 8 ft.
116.0, June 1961, upstream.	−32.0	.....	Amplitude 18, length 591.	.....	Smooth at start. Waves 2 to 8 ft. at top.	Smooth.	Rough. Waves 5 to 18 ft.
116.0, June 1961, downstream.	−34.0	+7.25	Amplitude 10, length 220.	+6.50	Rough. Waves 2 to 10 ft.	Irregular waves 2 to 3 ft.	Smooth.
140.5, Aug. 1958, upstream.	−54.5	.....	Amplitude 5, length 90.	.....	Rough. Waves 3 to 5 ft.	Upper end rough. Waves 2 to 4 ft. Lower end smooth.	Top rough. Waves 3 to 5 ft.
140.5, Aug. 1958, downstream.	−57.0	.....	Amplitude 6, length 170.	.....	Upstream end smooth. Downstream end waves 2 to 5 ft.	Upstream end rough with 2 to 3 ft. waves. Downstream end smooth.	Top rough. Waves 3 to 6 ft.
140.5, June 1961, upstream.	−50.0	.....	Amplitude 15, length 400.	.....	Rough. Waves 2 to 11 ft.	Rough. Waves 5 to 11 ft.	Rough. Waves 5 to 15 ft.
140.5, June 1961, Downstream.	−52.0	+4.75	Amplitude 16, length 612.	+10.00	Rough. Waves 2 to 12 ft.	Rough. Waves 4 to 12 ft.	Rough. Waves 10 to 16 ft.
154.0, Aug. 1958, upstream.	−34.0	.....	Amplitude 6, length 130.	.....	Slightly rough. Waves 1 to 2 ft.	Rough. Waves 2 to 4 ft.	Top rough. Waves 2 to 6 ft.
154.0, Aug. 1958, downstream.	−34.5	.....	Amplitude 7, length 125.	.....	Rough. Waves 3 to 7 ft.	Rough. Waves 4 to 6 ft.	Top rough. Waves 3 to 6 ft.
154.0, June 1961, upstream.	−28.0	.....	Amplitude 12, length 294.	.....	Irregular waves 2 to 6 ft.	Irregular waves 1 to 3 ft.	Rough downstream. Waves 2 to 12 ft. Upstream ⅜ is smooth.
154.0, June 1961, downstream.	−23.0	+8.75	Amplitude 11, length 283.	+5.00	Rough. Waves 5 to 8 ft.	Smooth.	Rough. Waves 4 to 11 ft.
163.0, Aug. 1958, upstream.	−44.5	.....	Amplitude 8, length 150.	.....	Smooth upstream half. Waves 2 to 8 ft.	Smooth downstream half. Waves 3 to 5 ft.	Top rough. Waves 3 to 5 ft.
163.0, Aug. 1958, downstream.	−48.0	.....	Amplitude 9, length 110.	.....	Rough. Waves 4 to 9 ft.	Smooth downstream half. Waves 3 to 5 ft.	Top rough. Waves 3 to 6 ft.
163.0, June 1961, upstream.	−46.0	.....	Amplitude 11, length 98.	.....	Rough. Waves 2 to 9 ft.	Smooth.	Rough. Waves 3 to 11 ft.
163.0, June 1961, downstream.	−46.0	+ .25	Amplitude 13, length 303.	+3.50	Rough. Waves 4 to 13 ft.	Smooth.	Rough. Waves 6 to 10 ft.

167.0, Aug. 1958, upstream.	−54.5	.....	Amplitude 11, length 200.	.....	Smooth. Small waves 2 to 3 ft. at top.	Rough. Waves 6 to 11 ft.	Top rough. Waves 6 to 10 ft.
167.0, Aug. 1958, downstream.	−55.0	.....	Amplitude 12, length 270.	.....	Rough. Small waves 2 to 3 ft. remnants large waves 4 to 6 ft.	Rough. Waves 5 to 9 ft.	Top rough. Waves 6 to 12 ft.
167.0, June 1961, upstream.	−47.0	.....	Amplitude 28, length 392.	.....	Rough. Waves 2 to 28 ft.	Irregular. Waves 2 to 4 ft.	Rough. Waves 8 to 20 ft.
167.0, June 1961, downstream.	−48.0	+7.25	Amplitude 20, length 404.	+12.50	Rough. Waves 6 to 18 ft.	Rough. Waves 2 to 6 ft.	Rough. Waves 8 to 20 ft.
183.0, Aug. 1958, upstream.	−44.0	.....	Amplitude 14, length 210.	.....	Rough. Waves 2 to 3 ft.	Smooth.	Rough. Waves 6 to 10 ft.
183.0, Aug. 1958, downstream.	−41.0	.....	Amplitude 14, length 375.	.....	Rough. Waves 3 to 6 ft.	Rough. Waves 3 to 6 ft.	Rough. Waves 6 to 14 ft.
183.0, June 1961, upstream.	−50.0	.....	Amplitude 17, length 554.	.....	Rough. Waves 3 to 17 ft.	Smooth.	Almost smooth. Waves 3 to 6 ft.
183.0, June 1961, downstream.	−49.0	−7.00	Amplitude 24, length 378.	+6.50	Rough. Waves 7 to 24 ft.	Rough. Waves 5 to 8 ft.	Rough. Waves 9 to 24 ft.
190.5, Aug. 1958, upstream.	−34.5	.....	Amplitude 10, length 160.	.....	Smooth upstream half. Waves downstream 2 to 6 ft.	Rough. Waves 5 to 8 ft.	Top rough. Waves 6 to 10 ft.
190.5, Aug. 1958, downstream.	−36.0	.....	Amplitude 13, length 180.	.....	Smooth at bottom. Waves 2 to 6 ft.	Rough. Waves 3 to 6 ft.	Top rough. Waves 4 to 13 ft.
190.5, June 1961, upstream.	−37.0	.....	Amplitude 16, length 792.	.....	Rough. Waves 2 to 15 ft.	Smooth.	Rough. Waves 7 to 16 ft.
190.5, June 1961, downstream.	−41.0	−3.75	Amplitude 15, length 438.	+4.00	Irregular.	Irregular. Waves 2 to 3 ft.	Rough. Waves 7 to 15 ft.
197.5, Aug. 1958, upstream.	−37.5	.....	Amplitude 9, length 160.	.....	Generally smooth with waves 1 to 2 ft.	Rough. Waves 2 to 5 ft.	Top rough. Waves 5 to 9 ft.
197.5, Aug. 1958, downstream.	−36.0	.....	Amplitude 10, length 190.	.....	Smooth at bottom. Rough with waves.	Rough. Waves	Top rough. Waves 6 to 10 ft.
197.5, June 1961, upstream.	−36.0	.....	Amplitude 17, length 594.	.....	Rough. Waves 2 to 11 ft.	Irregular. Waves 2 to 5 ft.	Rough. Waves 4 to 17 ft.
197.5, June 1961, downstream.	−37.0	+ .25	Amplitude 16, length 279.	+7.00	Rough. Waves 2 to 12 ft.	Rough. Waves 2 to 6 ft.	Rough. Waves 6 to 16 ft.
204.0, Aug. 1958, upstream.	−33.0	.....	Amplitude 12, length 170.	.....	Rough. Waves 3 to 12 ft.	Smooth at bottom. Waves 2 to 5 ft.	Top rough. Waves 6 to 12 ft.
204.0, Aug. 1958, downstream.	−34.5	.....	Amplitude 10, length 190.	.....	Rough. Waves 3 to 9 ft.	Smooth.	Top rough. Waves 5 to 10 ft.
204.0, June 1961, upstream.	−42.0	.....	Amplitude 15, length 392.	.....	Rough. Waves 2 to 10 ft.	Irregular. Waves 1 to 3 ft.	Rough. Waves 5 to 15 ft.
204.0, June 1961, downstream.	−41.0	−7.75	Amplitude 16, length 286.	+4.50	Bottom rough. Upper part smooth.	Smooth and steep.	Rough. Waves 10 to 16 ft.
212.0, Aug. 1958, upstream.	−38.0	.....	Amplitude 12, length 200.	.....	Regular at bottom, waves 1 to 2 ft. 3 to 6 ft. waves at top.	Rough. Waves 3 to 8 ft.	Top rough. Waves 5 to 12 ft.
212.0, Aug. 1958, downstream.	−36.5	.....	Amplitude 17, length 250.	.....	Regular at bottom, waves 1 to 2 ft. 3 to 7 ft. waves at top.	Rough. Waves 3 to 8 ft.	Top rough. Waves 7 to 12 ft.
212.0, June 1961, upstream.	−40.0	.....	Amplitude 14, length 1,078.	.....	Rough. Waves 2 to 6 ft.	Irregular. Waves 2 to 5 ft.	Rough. Waves 8 to 14 ft.
212.0, June 1961, downstream.	−35.0	− .25	Amplitude 25, length 1,591	+5.00	Rough. Waves 2 to 6 ft.	Rough. Waves 3 to 9 ft.	Top rough. Waves 15 to 25 ft.

See footnotes at end of table.

TABLE 6.—*Effect of water temperature on riverbed configuration, Mississippi River, mile 110 to mile 298 A.H.P., August 4 to 18, 1958, (82° to 83° F.) versus June 1 to 6, 1961 (70° to 73° F.)—Continued*

Crossing location mile, A.H.P.,[1] date, and direction	High point M.L.W.[2]	Average difference in elevation	Wave amplitude and length	Average difference in amplitude	Condition of crossing slopes		Description of top of crossing: nature and location of wave formations and of smooth stretches
					Approach slope	Leaving slope	
	Feet	*Feet*	*Feet*	*Feet*			
223.5, Aug. 1958, upstream.	−31.0	.....	Amplitude 13, length 450.	.....	Rough. Waves 3 to 10 ft.	Rough. Waves 2 to 9 ft.	Rough. Waves 8 to 13 ft.
223.5, Aug. 1958, downstream.	−32.0	.....	Amplitude 15, length 475.	.....	Rough. Waves 4 to 12 ft.	Rough. Waves 3 to 9 ft.	Rough. Waves 6 to 15 ft.
223.5, June 1961, upstream.	−33.0	.....	Amplitude 13, length 497.	.....	Rough. Waves 2 to 7 ft.	Smooth.	Rough. Waves 4 to 13 ft.
223.5, June 1961, downstream.	−32.0	−1.00	Amplitude 17, length 500.	+1.00	Rough. Waves 2 to 10 ft.	Smooth at top. Lower part has waves 3 to 5 ft.	Rough. Waves 7 to 17 ft.
236.0, Aug. 1958, upstream.	−24.0	.....	Amplitude 20, length 350.	.....	Smooth with waves 1 to 2 ft.	Rough. Waves 3 to 8 ft.	Rough. Waves 9 to 17 ft.
236.0, Aug. 1958, downstream.	−26.5	.....	Amplitude 10, length 100.	.....	Rough. Waves 4 to 7 ft.	Rough. Waves 5 to 7 ft.	Rough. Waves 5 to 10 ft.
236.0, June 1961, upstream.	−32.0	.....	Amplitude 21, length 306.	.....	Rough. Waves 3 to 21 ft.	Rough. Waves 3 to 19 ft. Lower half is smooth.	Rough. Waves 3 to 14 ft.
236.0, June 1961, downstream.	−41.0	−11.25	Amplitude 18, length 1,300.	+4.50	Rough. Waves 3 to 11 ft.	Smooth.	Rough. Waves 6 to 18 ft.
241.0, Aug. 1958, upstream.	−23.0	.....	Amplitude 8, length 130.	.....	Slightly rough with waves 1 to 3 ft.	Upper part smooth. Waves 3 to 5 ft.	Rough. Waves 3 to 8 ft.
241.0, Aug. 1958, downstream.	−20.0	.....	Amplitude 15, length 200.	.....	Rough. Waves 4 to 8 ft.	Smooth except upper part with waves 3 to 6 ft.	Rough. Waves 5 to 12 ft.
241.0, June 1961, upstream.	−33.0	.....	Amplitude 13, length 229.	.....	Rough. Waves 2 to 9 ft.	Smooth.	Rough. Waves 1 to 13 ft.
241.0, June 1961, downstream.	−38.0	−14.00	Amplitude 18, length 400.	+4.00	Rough. Waves 2 to 8 ft.	Smooth and steep.	Rough. Waves 9 to 18 ft.
251.0, Aug. 1958, upstream.	−12.5	.....	Amplitude 14, length 275.	.....	Regular with waves 2 to 6 ft.	Rough. Waves 3 to 7 ft.	Rough. Waves 5 to 14 ft.
251.0, Aug. 1958, downstream.	−16.5	.....	Amplitude 14, length 250.	.....	Regular with waves 2 to 6 ft.	Rough. Waves 5 to 14 ft.	Rough. Waves 4 to 14 ft.
251.0, June 1961, upstream.	−22.0	.....	Amplitude 15, length 300.		Rough. Waves 1 to 3 ft.	Smooth.	Upstream part rough. Waves 5 to 15 ft. Downstream part smooth.
251.0, June 1961, downstream.	−22.0	−7.50	Amplitude 15, length 444.	+1.00	Rough. Waves 1 to 6 ft.	Smooth.	Rough. Waves 5 to 15 ft.
258.5, Aug. 1958, upstream.	−20.0	.....	Amplitude 16, length 500.	.....	Regular with small waves 2 to 3 ft.	Rough. Waves 4 to 9 ft.	Rough. Waves 4 to 16 ft.
258.5, Aug. 1958, downstream.	−18.0	.....	Amplitude 15, length 500.	.....	Regular with small waves 2 to 5 ft.	Rough. Waves 4 to 10 ft.	Rough. Waves 4 to 16 ft.
258.5, June 1961, upstream.	−34.0	.....	Amplitude 16, length 1,060.	.....	Rough. Waves 2 to 10 ft.	Smooth.	Rough upstream. Waves 7 to 16 ft. Downstream part smooth.
258.5, June 1961, downstream.	−32.0	−14.00	Amplitude 16, length 355.	+1.00	Rough. Waves 1 to 3 ft.	Smooth.	Rough. Waves 6 to 17 ft.
269.5, Aug. 1958, upstream.	−30.5	.....	Amplitude 12, length 250.	.....	Rough. Waves 5 to 12 ft.	Top rough with 3 to 10 ft. waves. Bottom smooth.	Rough. Waves 4 to 12 ft.

269.5, Aug. 1958, downstream.	−39.0	.....	Amplitude 15, length 225.	.....	Rough. Waves 3 to 6 ft.	Top rough with 3 to 6 ft. waves.	Rough. Waves 4 to 15 ft.
269.5, June 1961, upstream.	−61.0	.....	Amplitude 12, length 200.	.....	Irregular. Waves 2 to 4 ft.	Rough. Waves 3 to 6 ft.	Rough. Waves 2 to 5 ft.
269.5, June 1961, downstream.	−59.0	−25.25	Amplitude 9, length 222.	−3.00	Rough. Waves 2 to 5 ft.	Rough. Waves 3 to 10 ft.	Rough. Waves 2 to 6 ft.
273.0, Aug. 1958, upstream.	− 9.0	.....	Amplitude 16, length 1,000.	.....	Regular with 2 to 5 ft. long undulations.	Smooth with long undulations.	Rough upper end with very long waves 5 to 16 ft. Lower end smooth.
273.0, Aug. 1958, downstream.	− 7.0	.....	Amplitude 21, length 1,250.	.....	Smooth at bottom. Top with 2 to 6 ft. waves.	Waves 3 to 5 ft. and long undulations.	Rough. Very long waves 7 to 17 ft.
273.0, June 1961 upstream.	−28.0	.....	Amplitude 15, length 695.	.....	Rough. Waves 3 to 6 ft.	No system.	Rough. Waves 5 to 15 ft.
273.0, June 1961, downstream.	−23.0	−17.50	Amplitude 14, length 304.	−4.00	Rough. Waves 3 to 9 ft.	No system.	Rough. Waves 3 to 14 ft.
279.0, Aug. 1958, upstream.	−32.5	.....	Amplitude 22, length 350.	.....	Rough. Waves 3 to 7 ft.	Rough. Waves 3 to 6 ft.	Rough. Waves 4 to 15 ft.
279.0, Aug. 1958, downstream.	−29.0	.....	Amplitude 23, length 350.	.....	Rough. Waves 3 to 5 ft.	Smooth.	Rough. Waves 4 to 14 ft.
279.0, June 1961, upstream.	−44.0	.....	Amplitude 20, length 556.	.....	Irregular. Waves 2 to 3 ft.	Smooth.	Rough. Waves 2 to 20 ft. Some smooth stretches.
279.0, June 1961 downstream.	−50.0	−16.25	Amplitude 18, length 665.	−4.00	Rough. Waves 3 to 16 ft.	Smooth.	Upstream half is rough. Waves 6 to 18 ft. Downstream half is smooth.
286.5, Aug. 1958, upstream.	− 6.0	.....	Amplitude 25, length 425.	.....	Long undulations 3 to 6 ft. with small waves 1 to 2 ft. superimposed.	Long undulations 3 to 5 ft.	Rough. Very large waves 6 to 19 ft.
286.5, Aug. 1958, downstream.	− 7.0	.....	Amplitude 25, length 350.	.....	Long undulations 3 to 6 ft. with small waves 1 to 2 ft. superimposed.	Long undulations 3 to 5 ft.	Rough. Very large waves 5 to 25 ft.
286.5, June 1961, upstream.	−32.0	.....	Amplitude 15, length 953.	.....	Rough. Waves 3 to 6 ft.	Rough. Long waves, 5 to 15 ft.	Smooth.
286.5, June 1961, downstream.	−28.0	−23.50	Amplitude 17, length 380.	−9.00	Rough. Waves 2 to 9 ft.	Rough. Waves 2 to 9 ft.	Rough. Waves 6 to 17 ft.
298.0, Aug. 1958, upstream.	− .5	.....	Amplitude 10, length 450.	.....	Long undulations 1 to 2 ft. with waves superimposed.	Rough. Waves 3 to 9 ft.	Rough. Waves 4 to 10 ft.
298.0, Aug. 1958, downstream.	− 2.5	.....	Amplitude 11, length 250.	.....	Long waves 3 to 6 ft.	Smooth and steep.	Rough. Waves 3 to 11 ft.
298.0, June 1961, upstream.	(3)	.....	Amplitude length	DID NOT RECORD POSITION ON UPSTREAM RUN.			
298.0, June 1961, downstream.	−21.0	.....	Amplitude 6, length 548.	.....	Irregular. Waves 1 to 3 ft.	Rough. Waves 2 to 6 ft.	Rough. Waves 2 to 5 ft.

[1] Above head of passes.
[2] Mean low water.
[3] No definite waves. Smooth undulations.

TABLE 7.—*Effect of water temperature on riverbed configuration, Mississippi River, mile 110 to mile 298 A.H.P., August 4 to 18, 1958, (82° to 83° F.) versus February 16 to 19, 1962 (46° to 47° F.)*

Crossing location mile, A.H.P.,[1] date, and direction	High point M.L.W.[2]	Average difference in elevation	Wave amplitude and length	Average difference in amplitude	Condition of crossing slopes		Description of top of crossing: nature and location of wave formations and of smooth stretches
					Approach slope	Leaving slope	
	Feet	*Feet*	*Feet*	*Feet*			
110.0, Aug. 1958, upstream.	−63.0	.....	Amplitude 6, length 100.	.....	Rough. Waves 4 to 6 ft.	Upstream end rough. Waves 3 to 5 ft. Downstream part smooth.	Top rough. Waves 3 to 6 ft.
110.0, Aug. 1958, downstream.	−60.0	.....	Amplitude 13, length 240.	.....	Rough. Waves 6 to 13 ft.	Almost smooth throughout.	Top rough. Waves 6 to 13 ft.
110.0, Feb. 1962, upstream.	−78.8	.....	Amplitude 9, length 287.	.....	Smooth. General irregularities.	Smooth. General irregularities.	Smooth.
110.0, Feb. 1962, downstream.	−72.2	−14.00	Amplitude 7, length 119.	−1.50	Smooth to irregular.	Rough. Waves 2 to 6 ft.	Smooth.
116.0, Aug. 1958, upstream.	−41.0	.....	Amplitude 7, length 125.	.....	Smooth to near upstream end. Then waves 2 to 3 ft.	Upstream end, waves 3 to 5 ft.	Top rough. Waves 4 to 7 ft.
116.0, Aug. 1958, downstream.	−39.5	.....	Amplitude 8, length 165.	.....	Downstream end undulating. Upstream end rough. Waves 4 to 6 ft.	Rough. Waves 3 to 6 ft.	Top rough. Waves 5 to 8 ft.
116.0, Feb. 1962, upstream.	−48.3	.....	Amplitude 13, length 287.	.....	Rough. Waves 1 to 7 ft. high.	Generally smooth.	Rough. Waves 5 to 13 ft. high.
116.0, Feb. 1962, downstream.	−39.6	−3.75	Amplitude 14, length 297.	+6.00	Rough. Waves 2 to 14 ft.	Irregular.	Rough. Waves 6 to 14 ft.
140.5, Aug. 1958, upstream.	−54.5	.....	Amplitude 5, length 90.	.....	Rough. Waves 3 to 5 ft.	Upstream end rough. Waves 2 to 4 ft. Downstream end smooth.	Top rough. Waves 3 to 5 ft.
140.5, Aug. 1958, downstream.	−57.0	.....	Amplitude 6, length 170.	.....	Upstream end smooth. Downstream end, waves 2 to 5 ft.	Upstream end rough. Waves 2 to 3 ft. Downstream end smooth.	Top rough. Waves 3 to 6 ft.
140.5, Feb. 1962, upstream.	−49.9	.....	Amplitude 11, length 191.	.....	Rough. Waves 7 ft. high.	Rough. Waves 2 to 6 ft. high.	Rough. Waves 3 to 7 ft. high.
140.5, Feb. 1962, downstream.	−54.1	+3.75	Amplitude 15, length 293.	+7.50	Rough. Waves 3 to 15 ft. high.	Rough. Waves 2 to 8 ft.	Top rough. Waves 5 to 15 ft.
154.0, Aug. 1958, upstream.	−34.0	.....	Amplitude 6, length 130.	.....	Slightly rough. Waves 1 to 2 ft.	Rough. Waves 2 to 4 ft.	Top rough. Waves 2 to 6 ft.
154.0, Aug. 1958, downstream.	−34.5	.....	Amplitude 7, length 125.	.....	Rough. Waves 3 to 7 ft.	Rough. Waves 4 to 6 ft.	Top rough. Waves 3 to 6 ft.
154.0, Feb. 1962, upstream.	−39.2	.....	Amplitude 7, length 223.	.....	Smooth, becoming rough with waves 2 to 5 ft.	Rough and irregular. Waves 2 to 7 ft.	Rough with waves 2 to 4 ft., close together.
154.0, Feb. 1962, downstream.	−38.7	−4.75	Amplitude 13, length 147.	+3.50	Rough. Waves 2 to 5 ft.	Rough. Waves 3 to 13 ft.	Rough. Waves 2 to 6 ft.
163.0, Aug. 1958, upstream.	−44.5	.....	Amplitude 8, length 150.	.....	Smooth downstream half. Waves 2 to 8 ft.	Smooth downstream half. Waves 3 to 5 ft.	Top rough. Waves 3 to 5 ft.
163.0, Aug. 1958, downstream.	−48.0	.....	Amplitude 9, length 110.	.....	Rough. Waves 4 to 9 ft.	Smooth downstream half. Waves 3 to 5 ft.	Top rough. Waves 3 to 6 ft.
163.0, Feb. 1962, upstream.	−56.1	.....	Amplitude 11, length 239.	.....	Rough. Waves 2 to 10 ft.	Rough. Waves 2 to 6 ft.	Rough with larger waves, 5 to 11 ft.
163.0, Feb. 1962, downstream.	−41.7	−2.65	Amplitude 10, length 234.	+2.00	Rough. Waves 2 to 7 ft.	Irregular. Waves 2 to 4 ft.	Smooth and steep.

167.0, Aug. 1958, upstream.	−54.5	.....	Amplitude 11, length 200.	.....	Smooth. Small waves 2 to 3 ft. at top.	Rough. Waves 6 to 11 ft.	Top rough. Waves 6 to 10 ft.
167.0, Aug. 1958, downstream.	−55.0	.....	Amplitude 12, length 270.	.....	Rough. Small waves 2 to 3 ft. on remnants of large waves 4 to 6 ft.	Rough. Waves 5 to 9 ft.	Top rough. Waves 6 to 12 ft.
167.0, Feb. 1962, upstream.	−52.7	.....	Amplitude 20, length 447.	.....	Rough. Starting out with small waves becoming larger at top of crossing.	Rough. No small waves. Large waves 10 to 16 ft.	Rough. Waves 10 to 20 ft.
167.0, Feb. 1962, downstream.	−54.2	+1.30	Amplitude 22, length 733.	+9.50	Rough. Waves 3 to 12 ft.	Rough. Waves 3 to 10 ft.	Rough. Waves 6 to 22 ft.
183.0, Aug. 1958, upstream.	−44.0	.....	Amplitude 14, length 210.	.....	Rough. Waves 2 to 3 ft.	Smooth.	Rough. Waves 6 to 10 ft.
183.0, Aug. 1958, downstream.	−41.0	.....	Amplitude 14, length 375.	.....	Rough. Waves 3 to 6 ft.	Rough. Waves 3 to 6 ft.	Rough. Waves 6 to 14 ft.
183.0, Feb. 1962, upstream.	−45.8	.....	Amplitude 13, length 447.	.....	Fairly regular. Waves 2 to 3 ft.	Smooth.	Large waves 5 to 13 ft.
183.0, Feb. 1962, downstream.	−40.4	− .60	Amplitude 15, length 510.	+ .00	Smooth.	Smooth.	Rough. Large waves 8 to 15 ft. Lower half smooth.
190.5, Aug. 1958, upstream.	−34.5	.....	Amplitude 10, length 160.	.....	Smooth upstream half. Waves downstream 2 to 6 ft.	Rough. Waves 5 to 8 ft.	Top rough. Waves 6 to 10 ft.
190.5, Aug. 1958, downstream.	−36.0	.....	Amplitude 13, length 180.	.....	Smooth at bottom. Waves 2 to 6 ft.	Rough. Waves 3 to 6 ft.	Top rough. Waves 4 to 13 ft.
190.5, Feb. 1962, upstream.	−42.0	.....	Amplitude 12, length 255.	.....	Rough. Waves 2 to 7 ft.	Rough. Waves 3 to 11 ft.	Rough. Waves 6 to 12 ft.
190.5, Feb. 1962, downstream.	−46.6	−9.05	Amplitude 15, length 210.	+2.00	Rough. Waves 3 to 13 ft.	Rough. Waves 3 to 13 ft.	Rough. Waves 5 to 15 ft.
197.5, Aug. 1958, upstream.	−37.5	.....	Amplitude 9, length 160.	.....	Generally smooth with waves 1 to 2 ft.	Rough. Waves 2 to 5 ft.	Top rough. Waves 5 to 9 ft.
197.5, Aug. 1958, downstream.	−36.0	.....	Amplitude 10, length 190.	.....	Smooth at bottom. Rough with waves.	Rough. Waves.	Top rough. Waves 6 to 10 ft.
197.5, Feb. 1962, upstream.	−40.2	.....	Amplitude 16, length 187.	.....	Regular at bottom. Upper part waves 2 to 9 ft.	Rough .Waves 3 to 16 ft.	Waves 2 to 7 ft.
197.5, Feb. 1962, downstream.	−39.8	−3.25	Amplitude 14, length 150.	+5.50	Rough. Waves 3 to 14 ft.	Rough. Waves 3 to 12 ft.	Rough. Waves 5 to 11 ft.
204.0, Aug. 1958, upstream.	−33.0	.....	Amplitude 12, length 170.	.....	Rough. Waves 3 to 12 ft.	Smooth at bottom. Waves 2 to 5 ft.	Top rough. Waves 6 to 12 ft.
204.0, Aug. 1958, downstream.	−34.5	.....	Amplitude 10, length 190.	.....	Rough. Waves 3 to 9 ft.	Smooth.	Top rough. Waves 5 to 10 ft.
204.0, Feb. 1962, upstream.	−41.5	.....	Amplitude 14, length 249.	.....	Rough. Waves 6 to 11 ft.	Regular, smooth, no wave system.	Rough. Waves 3 to 14 ft.
204.0, Feb. 1962, downstream.	−39.2	−6.60	Amplitude 17, length 270.	+4.50	Rough. Waves 2 to 17 ft.	Generally smooth, some waves 2 to 5 ft.	Irregular. Waves 2 to 5 ft.
212.0, Aug. 1958, upstream.	−38.0	.....	Amplitude 12, length 200.	.....	Regular at bottom. Waves 1 to 2 ft. Waves 3 to 6 ft. at top.	Rough. Waves 3 to 8 ft.	Top rough. Waves 5 to 12 ft.

See footnotes at end of table.

TABLE 7.—*Effect of water temperature on riverbed configuration, Mississippi River, mile 110 to mile 293 A.H.P., August 4 to 18, 1958, (82° to 83° F.) versus February 16 to 19, 1962 (46° to 47° F.)—Continued*

Crossing location mile, A.H.P.,[1] date, and direction	High point M.L.W.[2]	Average difference in elevation	Wave amplitude and length	Average difference in amplitude	Condition of crossing slopes		Description of top of crossing: nature and location of wave formations and of smooth stretches
					Approach slope	Leaving slope	
	Feet	*Feet*	*Feet*	*Feet*			
212.0, Aug. 1958, downstream.	−36.5	.....	Amplitude 17, length 250.	.....	Regular at bottom. Waves 1 to 2 ft. Waves 3 to 7 ft. at top.	Rough. Waves 3 to 8 ft.	Top rough. Waves 7 to 12 ft.
212.0, Feb. 1962, upstream.	−40.4	.....	Amplitude 13, length 280.	.....	Irregular small waves becoming fewer and larger nearing top.	Rough. Waves 3 to 8 ft.	Top rough. Large waves 6 to 13 ft.
212.0, Feb. 1962, downstream.	−41.3	−3.60	Amplitude 14, length 450.	−1.00	Rough. Waves 2 to 12 ft.	Irregular. Waves 2 to 4 ft.	Rough. Waves 7 to 14 ft.
223.5, Aug. 1958, upstream.	−31.0	.....	Amplitude 13, length 450.	.....	Rough. Waves 3 to 10 ft.	Rough. Waves 2 to 9 ft.	Rough. Waves 8 to 13 ft.
223.5, Aug. 1958, downstream.	−32.0	.....	Amplitude 15, length 475.	.....	Rough. Waves 4 to 12 ft.	Rough. Waves 3 to 9 ft.	Rough. Waves 6 to 15 ft.
223.5, Feb. 1962, upstream.	−27.0	.....	Amplitude 16, length 498.	.....	Rough. Waves 2 to 9 ft.	Regular. No wave system.	Rough. Waves 2 to 7 ft.
223.5, Feb. 1962, downstream.	−30.8	+2.60	Amplitude 17, length 602.	+2.50	Rough. Waves 7 to 17 ft.	Rough. Waves 6 to 17 ft.	Irregular. Waves 2 to 10 ft.
236.0, Aug. 1958, upstream.	−24.0	.....	Amplitude 20, length 350.	.....	Smooth. Waves 1 to 2 ft.	Rough. Waves 3 to 8 ft.	Rough. Waves 9 to 17 ft.
236.0, Aug. 1958, downstream.	−26.5	.....	Amplitude 10, length 100.	.....	Rough. Waves 4 to 7 ft.	Rough. Waves 5 to 7 ft.	Rough. Waves 5 to 10 ft.
236.0, Feb. 1962, upstream.	−35.9	.....	Amplitude 16, length 311.	.....	Rough. Waves 3 to 10 ft.	Rough. Larger waves 6 to 10 ft.	Rough. Waves 3 to 8 ft.
236.0, Feb. 1962, downstream.	−30.6	−8.00	Amplitude 16, length 301.	+1.00	Rough. Waves 2 to 5 ft.	Smooth.	Rough. Waves 2 to 16 ft.
241.0, Aug. 1958, upstream.	−23.0	.....	Amplitude 8, length 130.	.....	Slightly rough with waves 1 to 3 ft.	Upstream part smooth. Waves 3 to 5 ft.	Rough. Waves 3 to 8 ft.
241.0, Aug. 1958, downstream.	−20.0	.....	Amplitude 15, length 200.	.....	Rough. Waves 4 to 8 ft.	Smooth except upstream part, with waves 3 to 6 ft.	Rough. Waves 5 to 12 ft.
241.0, Feb. 1962, upstream.	−30.7	.....	Amplitude 8, length 222.	.....	Irregular waves 2 to 8 ft.	Smooth.	Smooth.
241.0, Feb. 1962, downstream.	−31.0	−9.40	Amplitude 7, length 211.	−4.00	Rough. Waves 2 to 6 ft.	Smooth.	Rough. Waves 2 to 4 ft.
251.0, Aug. 1958, upstream.	−12.5	.....	Amplitude 14, length 275.	.....	Regular with waves 2 to 6 ft.	Rough. Waves 3 to 7 ft.	Rough. Waves 5 to 14 ft.
251.0, Aug. 1958, downstream.	−16.5	.....	Amplitude 14, length 250.	.....	Regular with waves 2 to 6 ft.	Rough. Waves 5 to 14 ft.	Rough. Waves 4 to 14 ft.
251.0, Feb. 1962, upstream.	.....	.....	Amplitude length	.....	WENT THROUGH PROFIT ISLAND CHUTE INSTEAD OF RIVER.		
251.0, Feb. 1962, downstream.	−28.1	.....	Amplitude 21, length 421.	.....	Rough. Waves 3 to 10 ft.	Rough. Waves 4 to 10 ft.	Rough. Waves to 21 ft.
258.5, Aug. 1958, upstream.	−20.0	.....	Amplitude 16, length 500.	.....	Regular with small waves 2 to 3 ft.	Rough. Waves 4 to 9 ft.	Rough. Waves 4 to 16 ft.
258.5, Aug. 1958, downstream.	−18.0	.....	Amplitude 15, length 500.	.....	Regular with small waves 2 to 5 ft.	Rough. Waves 4 to 10 ft.	Rough. Waves 4 to 16 ft.

258.5, Feb. 1962, upstream.	.....	.....	Amplitude length	.....	AT MOUTH OF PROFIT ISLAND CHUTE. DID NOT GET CROSSING.		
258.5, Feb. 1962, downstream.	−36.5	.....	Amplitude 16, length 421.	.....	Smooth.	Irregular.	Rough. Waves 4 to 16 ft.
269.5, Aug. 1958, upstream.	−30.5	.....	Amplitude 12, length 250.	.....	Rough. Waves 5 to 12 ft.	Top rough. Waves 3 to 10 ft. Bottom smooth.	Rough. Waves 4 to 12 ft.
269.5, Aug. 1958, downstream.	−39.0	.....	Amplitude 15, length 225.	.....	Rough. Waves 3 to 6 ft.	Top rough, with 3 to 6 ft. waves.	Rough. Waves 4 to 15 ft.
269.5, Feb. 1962, upstream.	−39.3	.....	Amplitude 18, length 634.	.....	Rough. Waves 5 to 10 ft.	Generally smooth.	No real "top", a single large wave.
269.5, Feb. 1962, downstream.	−43.5	−6.65	Amplitude 17, length 332.	+4.00	Rough. Waves 3 to 10 ft.	Smooth.	Rough. Waves 5 to 17 ft.
273.0, Aug. 1958, upstream.	− 9.0	.....	Amplitude 16, length 1,000	.....	Regular with 2 to 5 ft. long undulations.	Smooth, with long undulations.	Rough upstream end with very long waves 5 to 16 ft. Lower end smooth.
273.0, Aug. 1958, downstream.	− 7.0	.....	Amplitude 21, length 1,250.	.....	Smooth at bottom. Top with 2 to 6 ft. waves.	Waves 3 to 5 ft. and long undulations.	Rough. Very long waves 7 to 17 ft.
273.0, Feb. 1962, upstream.	− 8.3	.....	Amplitude 13, length 507.	.....	Irregular. Waves 2 to 3 ft.	Smooth at top. Irregular at bottom.	Smooth.
273.0, Feb. 1962, downstream.	−19.1	−5.70	Amplitude 12, length 277.	−6.00	Rough. Waves 6 to 12 ft.	Irregular. Waves 2 to 4 ft.	Smooth.
279.0, Aug. 1958, upstream.	−32.5	.....	Amplitude 22, length 350.	.....	Rough. Waves 3 to 7 ft.	Rough. Waves 3 to 6 ft.	Rough. Waves 4 to 15 ft.
279.0, Aug. 1958, downstream.	−29.0	.....	Amplitude 23, length 350.	.....	Rough. Waves 3 to 5 ft.	Smooth.	Rough. Waves 4 to 14 ft.
279.0, Feb. 1962, upstream.	−24.8	.....	Amplitude 23, length 1,110.	.....	Irregular. Waves 2 to 9 ft.	Smooth.	Rough. Large waves 6 to 23 ft.
279.0, Feb. 1962, downstream.	−32.0	+2.35	Amplitude 18, length 970.	−2.50	Rough. Waves 3 to 12 ft.	Irregular.	Rough. Waves 10 to 18 ft.
286.5, Aug. 1958, upstream.	− 6.0	.....	Amplitude 25, length 425.	.....	Long undulations 3 to 6 ft. with small 1 to 2 ft. waves superimposed.	Long undulations 3 to 5 ft.	Rough. Very large waves 6 to 19 ft.
286.5, Aug. 1958, downstream.	− 7.0	.....	Amplitude 25, length 350.	.....	Long undulations 3 to 6 ft. with small 1 to 2 ft. waves superimposed.	Long undulations 3 to 5 ft.	Rough. Very large waves 5 to 25 ft.
286.5, Feb. 1962, upstream.	−28.7	.....	Amplitude 6, length 349.	.....	Irregular. Waves 2 to 6 ft.	Smooth.	Very smooth.
286.5, Feb. 1962, downstream.	−24.0	19.85	Amplitude 5, length 208.	−19.50	Rough. Waves 2 to 5 ft.	Smooth.	Very smooth.
298.0, Aug. 1958, upstream.	+ 0.5	.....	Amplitude 10, length 450.	.....	Long undulations, 1 to 2 ft. waves superimposed.	Rough. Waves 3 to 9 ft.	Rough. Waves 4 to 10 ft.
298.0, Aug. 1958, downstream.	− 2.5	.....	Amplitude 11, length 250.	.....	Long waves 3 to 6 ft.	Smooth and steep.	Rough. Waves 3 to 11 ft.
298.0, Feb. 1958, upstream.	−11.0	.....	Amplitude 8, length 153.	.....	Rough. Waves 2 to 3 ft.	Rough. Waves 3 to 7 ft.	Rough. Waves 2 to 3 ft.
298.0, Feb. 1962, downstream.	−4.5	−6.75	Amplitude 3, length 69.	−5.00	Same.	Irregular.	Rough. Waves 2 to 3 ft.

[1] Above head of passes.
[2] Mean low water.

TABLE 8.—*Summary of data of effect of water temperature on riverbed configuration, Mississippi River, mile 103 to 298 A.H.P., August 4 to 18, 1958 (82° to 83° F.), versus temperatures of other thalweg runs*

Crossing mileage A.H.P.	Thalweg run date	Temp. range (°F.)	High point M.L.W.[1] (feet)	Difference in elevation versus 1958 (feet)	Wave amplitude and length (feet)	Difference in amplitude versus 1958 (feet)	Crossing mileage A.H.P.	Thalweg run dates	Temp. range (°F.)	High point M.L.W.[1]	Difference in elevation versus 1958 (feet)	Wave amplitude and length[1] (feet)	Difference in amplitude versus 1958 (feet)
110.0	Aug. 4–18, 1958..	82–83	−61.5	.....	Amplitude, 10, length, 170.	.....		May 8–12, 1961..	64–66	−32.0	+ 3.25	Amplitude, 14, length, 400.	+ 2.00
	June 1–6, 1961...	70–73	−67.0	− 3.50	Amplitude, 23, length, 497.	+14.00		Apr. 21–27, 1960.	56–61	−42.5	− 7.50	Amplitude, 12, length, 450.	+ .50
	May 8–12, 1961..	64–66	−71.0	− 9.50	Amplitude, 11, length, 200.	+ 1.50		Mar. 20–25, 1961.	55–58	−42.5	− 7.25	Amplitude, 17, length, 400.	+ 5.00
	Apr. 21–27, 1960.	56–61	−64.5	− 3.00	Amplitude, 7, length, 220.	− 2.00		Feb. 16–19, 1962.	46–47	−44.3	− 9.05	Amplitude, 13, length, 233.	+ 2.00
	Mar. 20–25, 1961.	55–58	−70.5	− 9.00	(2)	.....		Feb. 9–18, 1959..	42–43	−37.8	− 2.50	Amplitude, 7, length, 140.	− 4.50
	Feb. 16–19, 1962.	46–47	−75.0	−14.00	Amplitude, 8, length, 203.	− 1.50	197.5	Aug. 4–18, 1958..	82–83	−36.8	.....	Amplitude, 10, length, 175.	.....
	Feb. 9–18, 1959..	42–43	−67.0	− 5.50	Amplitude, 6, length, 190.	− 5.50		June 1–6, 1961...	70–73	−36.5	+ 0.25	Amplitude, 16, length, 437.	+ 7.00
116.0	Aug. 4–18, 1958..	82–83	−40.3	.....	Amplitude, 8, length, 145.	.....		May 8–12, 1961..	64–66	−29.5	+ 7.25	Amplitude, 20, length, 925.	+10.50
	June 1–6, 1961...	70–73	−33.0	+ 7.25	Amplitude, 14, length, 406.	+ 6.50		Apr. 21–27, 1960.	56–61	−36.8	+ .00	Amplitude, 7, length, 350.	− 3.00
	May 8–12, 1961..	64–66	−36.0	+ 4.25	Amplitude, 13, length, 300.	+ 6.00		Mar. 20–25, 1961.	55–58	−41.5	− 4.75	Amplitude, 9, length, 275.	− .50
	Apr. 21–27, 1960.	56–61	−41.3	− 1.00	Amplitude, 7, length, 125.	− .50		Feb. 16–19, 1962.	46–47	−40.0	− 3.25	Amplitude, 15, length, 169.	+ 5.50
	Mar. 20–25, 1961.	55–58	−49.0	+ 8.75	Amplitude, 8, length, 200,	+ .50		Feb. 9–18, 1959..	42–43	−38.5	− 1.75	Amplitude, 7, length, 175,	− 2.00
	Feb. 16–19, 1962.	46–47	−44.0	+ 3.75	Amplitude, 13, length, 292.	+ 6.00	204.0	Aug. 4–18, 1958..	82–83	−33.7	.....	Amplitude, 11, length, 180.	.....
	Feb. 9–18, 1959..	42–43	−44.5	− 4.25	Amplitude, 3, length, 53.	− 4.50		June 1–6, 1961...	70–73	−41.5	− 7.75	Amplitude, 15, length, 339,	+ 4.50
140.5	Aug. 4–18, 1958..	82–83	−55.8	.....	Amplitude, 5, length 130.	.....		May 8–12, 1961..	64–66	−32.0	+ 1.75	Amplitude, 14, length, 650.	+ 3.00
	June 1–6, 1961...	70–73	−51.0	+ 4.75	Amplitude, 15, length, 506.	+10.00		Apr. 21–27, 1960.	56–61	−38.8	− 5.00	Amplitude, 13, length, 325.	+ 2.00
	May 8–12, 1961..	64–66	−48.5	+ 7.25	Amplitude, 14, length, 400.	+ 8.00		Mar. 20–25, 1961.	55–58	−47.0	−13.25	Amplitude, 8, length, 250.	− 3.50
	Apr. 21–27, 1960.	56–61	−51.8	+ 4.00	Amplitude, 10, length, 195.	+ 4.50		Feb. 16–19, 1962.	46–47	−40.4	− 6.60	Amplitude, 15, length, 260.	+ 4.50
	Mar. 20–25, 1961.	55–58	−53.0	+ 2.75	Amplitude, 10, length, 325.	+ 4.50		Feb. 9–18, 1959..	42–43	−38.8	− 5.00	Amplitude, 7, length, 135.	− 3.50
	Feb. 16–19, 1962.	46–47	−52.0	+ 3.75	Amplitude, 13, length, 242.	+ 7.50	212.0	Aug. 4–18, 1958..	82–83	−37.3	.....	Amplitude, 15, length, 225.	.....
	Feb. 9–18, 1959..	42–43	−58.5	− 2.75	(3)	− 5.50		June 1–6, 1961...	70–73	−37.5	− 0.25	Amplitude, 20, length, 1335.	+ 5.00
154.0	Aug. 4–18, 1958..	82–83	−34.3	.....	Amplitude 6, length, 128.	.....		May 8–12, 1961..	64–66	−28.0	+ 9.25	Amplitude, 23, length, 1050.	+ 9.00
	June 1–6, 1961...	70–73	−26.0	+ 8.75	Amplitude, 11, length, 289.	+ 5.00		Apr. 21–27, 1960.	56–61	−48.0	−10.75	Amplitude, 10, length, 225,	− 5.00
	May 8–12, 1961..	64–66	−26.0	+ 8.25	Amplitude, 13, length, 775.	+ 6.00		Mar. 20–25, 1961.	55–58	−42.0	− 4.75	Amplitude, 9, length, 200.	− 5.50

	Apr. 21–27, 1960.	56–61	−36.3	− 2.00	Amplitude, 9, length, 190.	+ 2.00
	Mar. 20–25, 1961.	55–58	−37.5	− 3.25	Amplitude, 15, length, 163.	+ 8.50
	Feb. 16–19, 1962.	46–47	−39.0	− 4.75	Amplitude, 10, length, 185.	+ 3.50
	Feb. 9–18, 1959..	42–43	−35.8	− 1.50	Amplitude, 4, length, 88.	− 3.00
163.0	Aug. 4–18, 1958..	82–83	−46.3	.....	Amplitude, 8, length, 130.	.....
	June 1–6, 1961...	70–73	−46.0	+ 0.25	Amplitude, 12, length, 201.	+ 3.50
	May 8–12, 1961..	64–66	−42.5	+ 3.75	Amplitude, 8, length, 275.	− .50
	Apr. 21–27, 1960.	56–61	−46.5	− .25	Amplitude, 8, length, 155.	− 1.00
	Mar. 20–25, 1961.	55–58	−50.0	− 3.75	Amplitude, 14, length, 200.	+ 5.00
	Feb. 16–19, 1962.	46–47	−48.9	− 2.65	Amplitude, 10, length, 237.	+ 2.00
	Feb. 9–18, 1959..	42–43	−48.5	− 2.00	Amplitude, 3, length, 93.	− 5.5
167.0	Aug. 4–18, 1958..	82–83	−54.8	.....	Amplitude, 12, length, 235.	.....
	June 1–6, 1961...	70–73	−47.0	+ 7.25	Amplitude, 24, length, 398.	+12.50
	May 8–12, 1961..	64–66	−47.0	+ 7.75	Amplitude, 16, length, 500.	+ 4.50
	Apr. 21–27, 1960.	56–61	−58.8	− 4.00	Amplitude, 10, length, 285.	− 1.50
	Mar. 20–25, 1961.	55–58	−52.5	+ 2.25	Amplitude, 11, length, 335.	− .50
	Feb. 16–19, 1962.	46–47	−53.5	+ 1.30	Amplitude, 21, length, 590.	+ 9.50
	Feb. 9–18, 1959..	42–43	−54.3	+ .50	Amplitude, 9, length, 200.	− 2.50
183.0	Aug. 4–18, 1958..	82–83	−42.5	.....	Amplitude, 14, length, 293.	.....
	June 1–6, 1961...	70–73	−49.5	− 7.00	Amplitude, 21, length, 466.	+ 6.50
	May 8–12, 1961..	64–66	−45.8	− 3.25	Amplitude, 20, length, 700.	+ 6.00
	Apr. 21–27, 1960.	56–61	−46.3	− 3.75	Amplitude, 15, length, 550.	+ 1.00
	Mar. 20–25, 1961.	55–58	−43.5	− 1.00	Amplitude, 13, length, 173.	− 1.00
	Feb. 16–19, 1962.	46–47	−43.1	− .60	Amplitude, 14, length, 479.	+ .00
	Feb. 9–18, 1959..	42–43	−43.9	− 1.25	Amplitude, 13, length, 215,	− 1.50
190.5	Aug. 4–18, 1958..	82–83	−35.3	.....	Amplitude, 12, length, 170.	.....
	June 1–6, 1961...	70–73	−39.0	− 3.75	Amplitude, 15, length, 615.	+ 4.00
	Feb. 16–19, 1962.	46–47	−40.9	− 3.60	Amplitude, 14, length, 365.	− 1.00
	Feb. 9–18, 1959..	42–43	−41.5	− 4.25	Amplitude, 9, length, 145,	− 5.50
223.5	Aug. 4–18, 1958..	82–83	−31.5	.....	Amplitude, 13, length, 463,	.....
	June 1–6, 1961...	70–73	−32.5	− 1.00	Amplitude, 15, length, 498.	+ 1.00
	May 8–12, 1961..	64–66	−31.3	+ .25	Amplitude, 15, length, 600.	+ 1.00
	Apr. 21–27, 1960.	56–61	−29.0	+ 2.50	Amplitude, 9, length, 260.	− 5.50
	Mar. 20–25, 1961.	55–58	−52.0	−20.50	Amplitude, 14, length, 475.	+ .50
	Feb. 16–19, 1962.	46–47	−28.9	+ 2.60	Amplitude, 16, length, 550.	+ 2.50
	Feb. 9–18, 1959.	42–43	−33.5	− 2.00	Amplitude, 13, length, 500.	− 1.00
236.0	Aug. 4–18, 1958..	82–83	−25.3	.....	Amplitude, 15, length, 225.	.....
	June 1–6, 1961...	70–73	−37.0	−11.25	Amplitude, 20, length, 803.	+ 4.50
	May 8–12, 1961..	64–66	−30.0	− 4.75	Amplitude, 12, length, 375.	− 3.00
	Apr. 21–27, 1960.	56–61	−34.0	− 8.75	Amplitude, 23, length, 1175.	+ 8.00
	Mar. 20–25, 1961.	55–58	−46.5	−21.25	Amplitude, 4, length, 150.	−10.50
	Feb. 16–19, 1962.	46–47	−33.3	− 8.00	Amplitude, 16, length, 306.	+ 1.00
	Feb. 9–18, 1959..	42–43	−30.8	− 5.50	Amplitude, 13, length, 215,	− 2.00
241.0	Aug. 4–18, 1958..	82–83	−21.5	.....	Amplitude, 12, length, 165.	.....
	June 1–6, 1961...	70–73	−36.0	−14.00	Amplitude, 15, length, 315.	+ 4.00
	May 8–12, 1961..	64–66	−35.0	−13.50	Amplitude, 9, length, 175.	− 2.50
	Apr. 21–27, 1960.	56–61	−26.0	− 4.50	(3)	−12.00
	Mar. 20–25, 1961.	55–58	−36.0	−14.50	Amplitude, 10, length, 138.	− 1.50
	Feb. 16–19, 1962.	46–47	−30.9	− 9.40	Amplitude, 8, length, 217.	− 4.00
	Feb. 9–18, 1959..	42–43	−24.5	− 3.00	Amplitude, 12, length, 325.	− .50
251.0	Aug. 4–18, 1958..	82–83	−14.5	.....	Amplitude, 14, length, 263.	.....
	June 1–6, 1961...	70–73	−22.0	− 7.50	Amplitude, 15, length, 372.	+ 1.00
	May 8–12, 1961..	64–66	−13.5	+ 1.00	Amplitude, 12, length, 325.	− 2.50
	Apr. 21–27, 1960.	56–61	−20.3	− 5.75	Amplitude, 12, length, 275.	− 2.00

See footnotes at end of table.

TABLE 8.—*Summary of data of effect of water temperature on riverbed configuration, Mississippi River, mile 103 to 298 A.H.P., August 4 to 18, 1958 (82° to 83° F.), versus temperatures of other thalweg runs—Continued*

Crossing mileage A.H.P.	Thalweg run dates	Temp. range (°F.)	High point M.L.W.[1]	Difference in elevation versus 1958 (feet)	Wave amplitude and length (feet)	Difference in amplitude versus 1958 (feet)
251.0	Mar. 20–25, 1961	55–58	−31.3	−16.75	Amplitude, 14, length, 213.	+ .00
	Feb. 16–19, 1962	46–47	(4)	(4)	(4)	(4)
	Feb. 9–18, 1959	42–43	−19.8	− 5.25	Amplitude, 9, length, 125.	− 4.50
258.5	Aug. 4–18, 1958	82–83	−19.0	.....	Amplitude, 15, length, 500.	.....
	June 1–6, 1961	70–73	−33.0	−14.00	Amplitude, 16, length, 708.	+ 1.00
	May 8–12, 1961	64–66	−31.5	−12.00	Amplitude, 14, length, 250.	− 1.50
	Apr. 21–27, 1960	56–61	−28.0	− 9.00	Smooth	−15.50
	Mar. 20–25, 1961	55–58	−26.9	− 7.75	Amplitude, 17, length, 350.	− 1.50
	Feb. 16–19, 1962	46–47	(4)	(4)	(4)	(4)
	Feb. 9–18, 1959	42–43	−21.3	− 1.25	Amplitude, 11, length, 215,	− 4.00
269.5	Aug. 4–18, 1958	82–83	−35.0	.....	Amplitude, 13, length, 238.	.....
	June 1–6, 1961	70–73	−60.0	−25.25	Amplitude, 11, length, 211.	− 3.00
	May 8–12, 1961	64–66	−52.0	−17.25	Amplitude, 16, length, 225.	+ 2.00
	Apr. 21–27, 1960	56–61	−41.8	− 7.00	Amplitude, 7, length, 163.	− 6.50
	Mar. 20–25, 1961	55–58	−47.8	−13.00	Amplitude, 12, length, 288.	− 1.50
	Feb. 16–19, 1962	46–47	−41.4	− 6.65	Amplitude, 17, length, 483.	+ 4.00
	Feb. 9–18, 1959	42–43	−35.5	− .75	Amplitude, 11, length, 250.	− 2.50
273.0	Aug. 4–18, 1958	82–83	− 8.0	.....	Amplitude, 19, length, 1125.	.....
	June 1–6, 1961	70–73	−25.5	−17.50	Amplitude, 14, length, 500.	− 4.00
	May 8–12, 1961	64–66	−17.0	− 8.50	Amplitude, 14, length, 425.	− 4.50
	Apr. 21–27, 1960	56–61	−31.8	− 5.75	(5)	−18.50
	Mar. 20–25, 1961	55–58	−27.8	−19.75	Amplitude, 11, length, 250.	− 7.50
	Feb. 16–19, 1962	46–47	−13.7	− 5.70	Amplitude, 13, length, 392.	− 6.00
	Feb. 9–18, 1959	42–43	−17.5	− 9.5	Amplitude, 10, length, 273.	− 8.00
279.0	Aug. 4–18, 1958	82–83	−30.8	.....	Amplitude, 22, length, 350.	.....
	June 1–6, 1961	70–73	−47.0	−16.25	Amplitude, 19, length, 611.	− 4.00
	May 8–12, 1961	64–66	−32.3	− 1.50	Amplitude, 20, length, 450.	− 2.50
	Apr. 21–27, 1960	56–61	−24.3	+ 6.50	Amplitude, 8, length, 470.	−14.50
	Mar. 20–25, 1961	55–58	−37.0	− 6.25	Amplitude, 15, length, 475.	− 7.50
	Feb. 16–19, 1962	46–47	−28.4	+ 2.35	Amplitude, 21, length, 1040.	− 2.50
	Feb. 9–18, 1959	42–43	−30.0	− 1.00	Amplitude, 8, length, 175.	− 14.5
286.5	Aug. 4–18, 1958	82–83	− 6.5	.....	Amplitude, 25, length, 388.	.....
	June 1–6, 1961	70–73	−30.0	−23.50	Amplitude, 16, length, 667.	− 9.00
	May 8–12, 1961	64–66	−22.5	−16.00	Amplitude, 15, length, 900.	−10.00
	Apr. 21–27, 1960	56–61	− 9.0	− 2.50	Amplitude, 20, length, 2200.	− 5.00
	Mar. 20–25, 1961	55–58	−30.5	−24.0	Amplitude, 7, length, 150.	−18.00
	Feb. 16–19, 1962	46–47	−26.4	−19.85	Amplitude, 5, length, 279.	−19.50
	Feb. 9–18, 1959	42–43	−23.8	−17.25	Amplitude, 9, length, 305.	−16.00
298.0	Aug. 4–18, 1958	82–82	− 1.0	.....	Amplitude, 10, length, 350.	.....
	June 1–6, 1961	70–73	−21.0	.....	Amplitude, 6, length, 548.	.....
	May 8–12, 1961	64–66	−16.5	−15.50	(6)	.....
	Apr. 21–27, 1960	56–61	− 8.8	− 7.75	Amplitude, 9, length, 400.	− 1.00
	Mar. 20–25, 1961	55–58	−21.5	−20.50	Amplitude, 10, length, 138.	− 1.00
	Feb. 16–19, 1962	46–47	− 7.8	− 6.75	Amplitude, 6, length, 111.	− 5.00
	Feb. 9–18, 1959	42–43	− 7.5	− 6.50	Amplitude, 9, length, 313.	− 1.50

[1] Average of upstream and downstream runs for same river rise.
[2] No wave system.
[3] Smooth.
[4] No data—boat went through Profit Island Chute instead of river.
[5] Smooth—no waves.
[6] No wave pattern.

TABLE 9.—*Mississippi River stage, discharge, and temperature data arranged to demonstrate effect of temperature on discharge for the period involved in thalweg profile runs, August 1958 through February 1962, at Red River Landing*[1]

Item [2]	Red River landing gage stages														
	32.0	32.5	33.0	33.5	34.0	34.5	35.0	35.5	36.0	36.5	37.0	37.5	38.0	8.5	39.0
Date	3/7/61	3/8/61	3/9/61	3/10/61	3/11/61	3/12/61	/313/61	3/14/61	3/15/61	3/16/61	3/16/61	3/17/61	3/17/61	3/18/61	3/19/61
Temp. °F	54	54	53	54	54	53	53	54	54	54	54	54	54	58	58
Discharge c.f.s.	726	743	756	769	780	789	799	808	820	833	847	862	879	898	917
Date	2/23/59	2/23/59	2/25/59	4/11/60	3/8/59	3/1/59	4/14/60	4/15/60	3/5/62	3/6/62	3/6/62	3/7/62	2/8/62	2/10/62	3/9/62
Temp. °F	46	46	46	60	54	51	54	54	48	46	46	46	44	44	46
Discharge c.f.s.	681	688	690	713	725	744	755	768	779	797	814	830	850	864	882
Date	4/9/60	4/9/60	4/10/60	3/9/59	4/12/60	4/14/60	2/1/62	2/2/62	2/3/62	2/4/62	4/22/60	2/6/62	3/8/62	3/9/62	5/3/61
Temp. °F	54	54	55	54	55	54	40	41	42	42	59	42	46	46	63
Discharge c.f.s.	643	666	688	706	725	742	752	765	778	792	809	825	847	863	820
Date	3/14/59	3/12/59	3/10/59	2/28/59	1/31/62	2/28/59	3/4/62	3/4/62	4/18/60	4/19/60	2/4/62	4/23/60	2/16/62	5/7/61	5/11/61
Temp. °F	55	57	56	46	40	46	49	49	56	57	42	59	46	65	65
Discharge c.f.s.	641	651	681	692	723	738	738	760	771	778	805	817	804	795	815
Date	7/26/58	7/27/58	3/1/62	3/2/62	2/24/62	1/31/62	2/21/62	2/20/62	2/19/62	2/19/62	2/18/62	2/17/62	6/12/61	5/9/61	6/12/61
Temp. °F	84	84	50	51	48	40	47	47	47	47	46	46	75	64	75
Discharge c.f.s.	616	622	650	670	705	737	723	739	756	768	781	792	698	793	722
Date	7/24/58	8/23/58	2/25/62	2/24/62	3/3/62	3/3/62	7/30/58	5/2/60	8/2/58	8/16/58	4/29/60	4/27/60		6/10/61	
Temp. °F	84	83	48	48	49	49	85	66	84	84	64	62		74	
Discharge c.f.s.	580	595	649	668	690	712	668	686	718	738	765	786		711	
Date	5/5/60	5/5/60	7/27/58	7/28/58	2/23/62	2/22/62	8/21/58	8/1/58	6/1/60	4/30/60	6/13/61	6/12/61			
Temp. °F	66	66	84	84	47	47	83	84	80	64	75	75			
Discharge c.f.s.	565	558	629	637	688	706	666	683	713	740	671	685			
Date	6/17/61	6/16/61	8/23/58	8/22/58	7/29/58	7/30/58	5/2/60	8/7/58	8/2/58	8/3/58					
Temp. °F	76	75	83	83	84	85	66	84	84	84					
Discharge c.f.s.	545	558	612	625	647	657	665	728	701	725					
Date			5/4/60	5/4/60	8/22/58	8/21/58	6/14/61	6/14/61		6/13/61					
Temp. °F			66	66	83	83	75	75		75					
Discharge c.f.s.			600	617	639	653	613	628		657					
Date			6/16/61	6/15/61	5/4/60	5/4/60									
Temp. °F			75	75	66	67									
Discharge c.f.s.			563	574	633	649									
Date					6/15/61	6/14/61									
Temp. °F					75	75									
Discharge c.f.s.					586	599									

[1] Except for February 1959 run, when later rise in March was used.
[2] Discharges are in thousands cubic feet per second from published Mississippi River Commission records.

feet) as the other runs. A glance at the actual values in the tables suggests that strong confirmation for the thesis of this paper could have been supplied. It is because of this reasoning that the results of the February 1959 run were not discarded: It was felt they serve a limited purpose if their limitations are kept in mind.

(2) Table 8: a summary tabulation of the major riverbed data presented in the tables above and arranged in order of temperature.

The most important riverbed statistics are considered to be the height referred to mean low water (M.L.W.) of the highest point of each of the crossings and the amplitude of the largest sand wave on each of the crossings. The tabulations show a general lowering of crossings with reduced water temperature and a general reduction in amplitude of the largest sand waves: Both of these statistics vary from crossing to crossing for each of the comparisons. Also, in a few cases, the crossings were not lower for the colder water. Finally, the lowering of crossings was more consistent than the reduction in amplitude of the maximum sand waves. The sand wave changes are complicated by the fact that, up to a certain stage (near bank full), sand waves become larger (see fig. 1); after which they tend to smooth out,[5] until possibly another series of (perhaps basically different type) waves or dunes tends to form again at high flood stages.

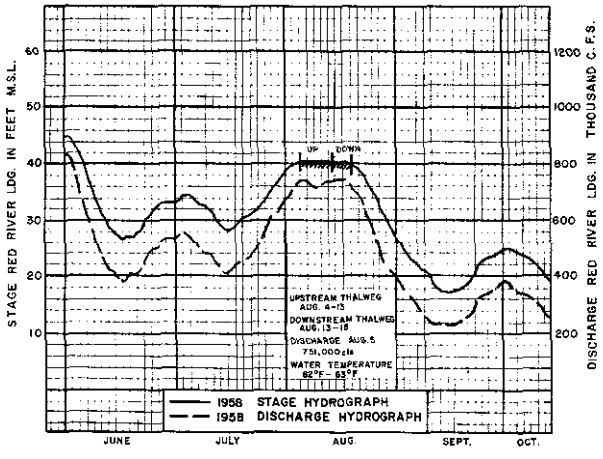


FIGURE 3. — 1958 stage and discharge hydrographs, Mississippi River, Red River Landing, La.

It is suggested that these deviations from a general trend may also, to some extent, be

[5] In which respect, the river conforms with Vanoni and Brooks' experience in their flume studies (see footnote 8) where they found that increased flow caused the ripples or dunes to disappear and the model bed to become smooth.

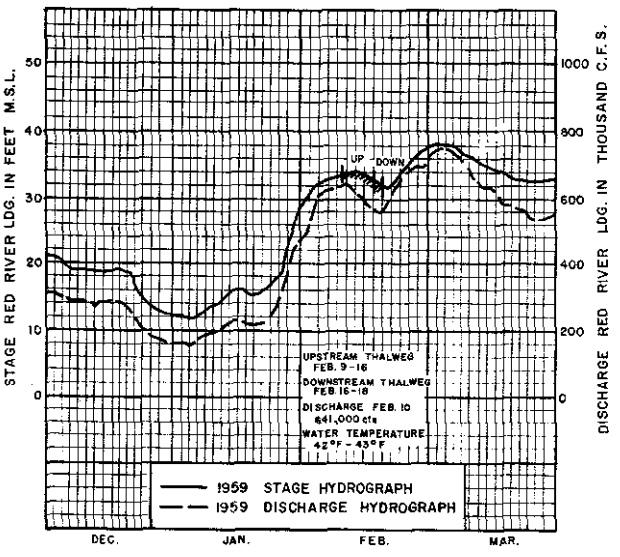


FIGURE 4. — 1959 flood: Stage and discharge hydrographs, Mississippi River, Red River Landing, La.

indications that we are dealing with a natural river flowing in a channel carved through somewhat heterogeneous formations containing important anomalies. That is to say, if the river flowed through homogeneous formations, with no anomalies, it would have a uniform alinement, like the channels in the Tyler-Friedkin models, and every crossing would behave in exactly the same way under the influence of varying temperature. Also, the comparisons as given are based upon crossings only. This is done for two reasons: First, it is believed that

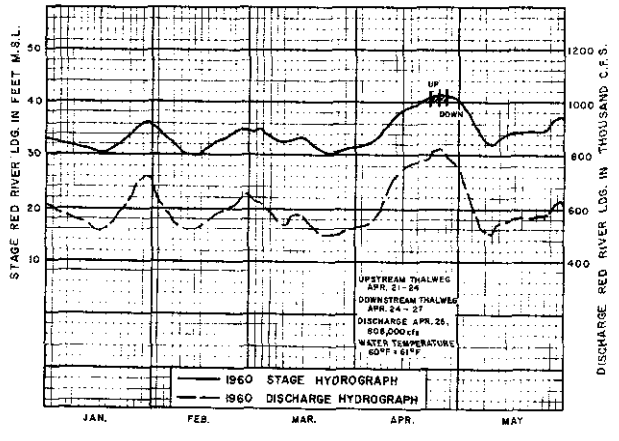


FIGURE 5.— 1960 flood: Stage and discharge hydrographs, Mississippi River, Red River Landing, La.

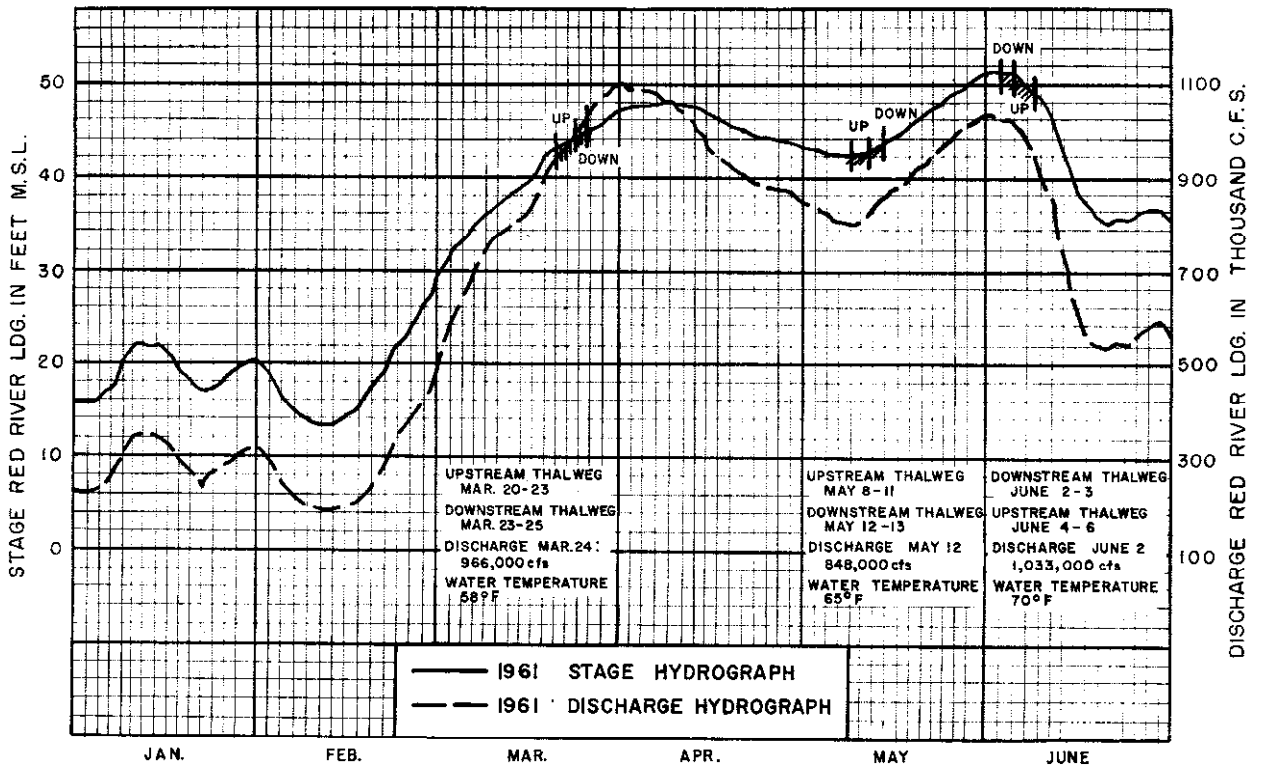


FIGURE 6. — 1961 flood: Stage and discharge hydrographs, Mississippi River, Red River Landing, La.

the crossings, being submerged weirs in effect, have the greatest influence on flow; and, second, it is more difficult to control the lateral position of the survey boat when running around bends. It may be mentioned that the basic data (thalweg profiles) indicate that the bends also deepen under the effect of colder water.

As already indicated, another basis for the thesis of this paper is afforded by a special presentation, based upon certain official published discharges of the Mississippi River Commission at Red River Landing (mile 300.6 A.H.P.)[6] at an approximate stage of 37 feet. Table 9 presents these data. Study of this table indicates that failure to observe temperature in discharge observations and to make proper allowance for its effect on flow in alluvial rivers may introduce an element of error into discharge measurements and computations.

[6] Just below Old River, after the Mississippi has divided; one part going down the Atchafalaya and the other part going down the Mississippi past New Orleans to the Gulf.

[7] This effect is confined to alluvial rivers; that is, there is no reason to expect any similar effect on the flow in canals, pipes, etc.

[8] See U.S. Army Engineer Division, Missouri River, Report E-68, Dec. 1957, "Laboratory Studies of Roughness and Suspended Load of Alluvial Streams," by Vito A. Vanoni and Norman H. Brooks.

Mechanisms by Which Water Temperature Affects Bed Configuration

Throughout the field of civil engineering, the primary effect of water temperature is ignored; that is to say, the change in viscosity of water in the normal range of temperatures has such a slight effect upon the manner in which water behaves that it can be, and is, ignored.

This fact undoubtedly is the reason for the failure to explore the possible secondary effect of temperature on flow in alluvial rivers;[7] that is, the effect of viscosity on the "fall velocity" of a particle of sediment and all the subsidiary phenomena that stem from this basic phenomenon. This "fall velocity" essentially determines or controls the ability of the water to put into suspension or motion certain sizes and quantities of sediment and ultimately, therefore, everything having to do with systematic variations in riverbed configuration.

Working with this major factor that affects riverbed configuration is another recently discovered factor — that muddy water has less resistance to flow than clear water and the muddier the water (within reason) the less the resistance.[8] Vanoni and Brooks' explanation, it may be noted, is that sediment particles dampen

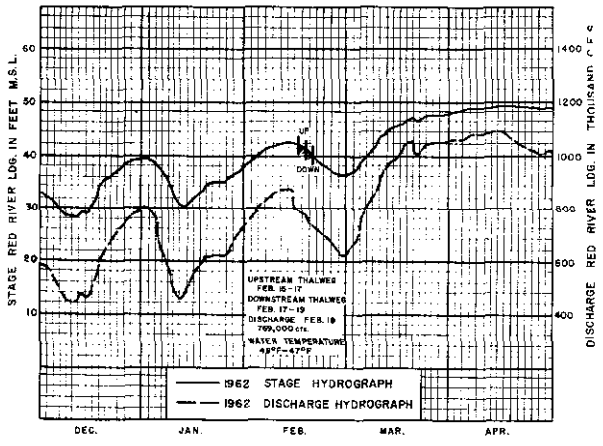


FIGURE 7. — 1962 flood: Stage and discharge hydrographs, Mississippi River, Red River Landing, La.

turbulence, hence the more particles of sediment the less turbulence, hence the less resistance to flow.

The present discovery and the Vanoni-Brooks discovery work together to account for the overall effect of temperature on riverbed configuration and, in turn, the effect on resistance to flow.

Conclusions

The author in recent rereading of various papers dealing with flow in rivers, movement of sediment and related subjects, notes that several writers were moving in the direction of recognizing the broad effect of temperature: In each instance, however, they veered off course just about the time when recognition might have been made. These other observers, however, did not have the fathometer at their disposal as a ready means of obtaining the many miles of thalweg profiles necessary for an empirical study of this phenomenon. Nevertheless and despite the foregoing, the existence of this hiatus in river hydraulics seems remarkable in the light of the fact that the full implications of the effect of water temperature on the fall velocity of sediment particles should clearly point toward the conclusion that everything pertaining to riverbed variation and to the quantities and sizes of sediment put into suspension or motion is affected to a major degree by water temperature. As a final concluding note, it would appear that, when the general thesis of this paper is further confirmed by more exhaustive empirical or laboratory investigations, it may be that hydrologists will adopt the practice of noting discharge at a "standard temperature." Perhaps tables of "correction factors" will be prepared to enable those making discharge computations to reduce the results to a standard temperature basis.

METHODS AND THEIR SUITABILITY FOR DETERMINING TOTAL SEDIMENT QUANTITIES

[Paper No. 32]

By JOHN R. SHEPPARD,[1] *hydraulic engineer, Sedimentation Section, Hydrology Branch, Bureau of Reclamation, Denver*

Abstract

Despite the many advances made in the last 15 years in the investigation and development of hydraulic and sedimentation phenomena, the hydraulic engineer must still use his experience and empirical data to estimate total sediment quantities. In the development of water resources projects, sedimentation is a question that must be answered in one way or another. The answer may be that sediment is of little concern or that a considerable amount of project investigation funds may be required for proper evaluation. In any event, some method must be used with the available data to derive the answer.

The Bureau of Reclamation uses several methods to estimate total sediment quantities that are dependent on the type of data available as well as the suitability of the method to the level of project development. Every effort is made to collect adequate field data for the proper level and to use that corresponding method in deriving the best estimate if possible.

The methods of determining total sediment quantities and a discussion of the suitability of each include: (1) The use of yield rates, (2) the use of suspended sediment sampling data with estimated bedload, (3) suspended data with bedload equations, (4) suspended data with the Modified Einstein procedure, and (5) suspended data with Colby curves of unmeasured load and velocity. A group of Colby curves for a number of western streams are presented.

Introduction

The quantity of total sediment load carried by a stream is a necessary determination in many phases of water resource development. Even if sediment appears to be no problem in planning for a project, it is wise to investigate

[1] Died June 14, 1964.

and estimate the quantities involved. The lack of sediment load, as well as too much sediment load, can create problems. Clear water releases below storage dams, or increased flows in conveyance or drainage channels may cause degradation or erosion of the channel from the lack of a sufficient sediment load.

The sediment load carried in a watercourse has been defined by various component parts. Measured and unmeasured sediment loads are defined and divided by the inability of a suspended sampler to sample near the streambed. Wash load, suspended load, saltation load, and bedload are defined by the predominant mode of transport of sediment particles. The total sediment load is generally understood as the sum of the component loads and is the whole quantity of sediment carried by any and all mode of transport by a stream.

Sediment may be carried by water in any type of conveyance such as pipelines, canals, penstocks, etc. The basic determination of sediment load in project development is that total load carried by a natural stream. For the purpose of this paper, the total sediment load quantities will be those natural loads of an alluvial-type stream that either supplies the water resource for the project or is involved in the project collection, conveyance, or distribution of water.

To the practicing engineer involved in the hydrology of a project, the determination of sediment load quantities is the first and basic step in analysis of sedimentation effects and problems associated with the project features and project operation. Estimates of reservoir storage depletion by sediment, stability studies of conveyance or drainage channels, design of sediment excluding devices at diversion structures, channel degradation estimates below storage structures, studies of channelization design, and other feature studies require the determination or estimate of the total sediment load and its characteristics. The practicing engineer may find it necessary to estimate sediment load quantities for past, present, or future conditions and to evaluate the estimate on the basis of suitability for the level of project development being investigated. The importance of sediment effects on the total project also must be considered in determining the suitability of the method used to derive the total sediment estimate.

A number of methods are described in this paper that have been used by the Bureau of Reclamation to estimate total sediment load quantities for project planning and project operation investigations. Each method has a suitability for certain levels of investigation, which is discussed. To some extent the method selection is also dependent on the type of data available and the expense of a data collection program.

Sediment Yield Rates

Sediment yield rates express the average sediment production of a drainage basin area in terms of a volume or weight per unit area for a unit time period. The usual units are acre-feet per square mile per year or tons per acre per year. Values of yield rates are based on sediment volumes from reservoir or stock pond resurveys, gross erosion rates with sediment delivery ratios, or sampling results on streams. The yield rates derived for storage loss or erosion rates generally are comparable to total sediment load quantities.

Yield rates are used primarily for small drainage areas where the vegetative cover, soils, and rainfall characteristics are uniform. Yield rates are, therefore, used in areas where the sediment produced is a function of drainage basin character rather than stream hydraulics. As such, yield rates can be estimated for drainage basins of similar characteristics.

The use of yield rates is limited to small structures in most cases because of drainage basin size or to appurtenant structures of larger projects. Yield rates are used in cases of tributary cross-drainage sediment contribution to canals where the intermittent flow of the tributary does not warrant a siphon or other cross drainage structure. Yield rates are often used in the preliminary or reconnaissance level of project development, because no other data are available and a data collection program has not been initiated.

The successful use of yield rates is more dependent on the experience of the engineer than upon his technical proficiency in hydraulics. The experience gained by the observation of varied drainage areas with different types of soils and geology, different types of erosion, and different types of vegetation, when combined with rainfall patterns and derived yield rates, is invaluable.

There are many sources of basic yield rate figures, such as the publications of the Soil Conservation Service, Department of Agriculture, on rates of sediment production for various sections of the United States. Any type of stock pond or reservoir resurvey data can provide rates of sediment yield, such as the file of reservoir sedimentation data summary sheets maintained by the Subcommittee on Sedimentation, Inter-Agency Committee on Water Resources. This file contains data from all Federal agencies who are members of the Subcommittee. State and local governmental agencies

may also publish reports that contain data for establishing yield rates.

Suspended Sediment Sampling Data

The largest amount of basic data for estimating total sediment load is the records of suspended sediment sampling measurements on many of our streams. These records report the observed concentration of suspended sediment particles in terms of parts per million by weight or in terms of suspended load in tons per day in conjunction with the observed or measured discharge of water.

With the sampling equipment presently used, suspended sediment sampling measurements do not measure the total sediment load or even the total suspended load. The construction of all samplers in general use at the present time prevents the nozzle from approaching the streambed by a definite distance when the sampler is resting on the bed of the stream. Because the concentration of suspended particles of sand size or larger increases with depth, a portion of the suspended load is not sampled; and the amount unsampled is proportionate to the unsampled depth and the total depth of the stream. The measured suspended load and the unmeasured load are the usual terms that define the total load by the combination of transported load in the two zones of flow.

To derive the average suspended load from the available suspended sampling data, the Bureau of Reclamation generally uses the flow-duration, sediment-rating curve method (*14*). The basic sediment-rating curve is a correlation of water discharge and suspended concentration or sediment load in tons per day. It was first developed by Campbell and Bauder (*1*). A flow-duration curve is a correlation between water discharge and the percentage of time the discharge occurs. Integration of the sediment-rating curve and flow-duration curve will give an average suspended sediment load.

The basic data for the sediment-rating curve and the flow-duration curve do not have to be from the same time period. Because discharge records are usually available over a longer time period than suspended sediment records, this method allows the expansion of a relatively small amount of sediment data to the longer period of discharge, provided no event or control has been imposed on the stream to change the basic sediment-water discharge relation.

The above description is a simple presentation of the method that represents a complex relation. Considerable deviation from a simple curve expressing the discharge–sediment relation is the rule rather than the exception, and may be due to a number of causes. A sediment-rating curve does not necessarily represent the transport capability of a stream but more often represents the water-sediment production capability of the drainage area. Sufficient data to define the average runoff conditions will produce an average curve that can be used with confidence. The water-sediment relation may change with the type of runoff, i.e., snowmelt or rainfall, and should be subdivided into more than one curve for different "seasons" of the time period. Operational changes on controlled streams may also produce different water-sediment relationships requiring subdivision of the sediment-rating curve and flow-duration time periods (*12*). Conservation measures on the watershed or control measures on the stream can produce a pronounced effect on the flow-duration and water-sediment relation.

The suitability of suspended load determinations is primarily dependent on the quality and length of the sediment and discharge data records available. Although a small amount of sediment data can successfully be used to estimate long-period conditions, those data can be biased by the conditions prevailing during the sediment sampling period. Sediment data collected during a low runoff period (compared to the long-period average) may give average loads that are high. Conversely, data collected during a high runoff period may result in average sediment loads that are too low. The suitability is also dependent on the discharge range through which the sampling was conducted in relation to the discharge range of the long-period flow-duration. In determining the average long-period sediment load, the major part is produced by the less-frequent, high discharges. The extension of the sediment-rating curve through this discharge range, because the sampling measurements did not include discharges in the range, can give results that are unreliable.

In project planning, the suspended sediment loads determined with data that may be biased can be estimated or corrected for the beginning levels of project investigation. As the project plan develops to higher levels, additional sampling programs may be desirable to correct the bias or obtain data through the greatest possible range of discharges.

The major part of available suspended sampling data is found in U.S. Geological Survey Water Supply Papers, although other Federal, State, or local agencies may conduct sampling programs to fill their needs.

Reports of published and unpublished sediment data inventories are issued periodically by the Subcommittee on Sedimentation, Inter-Agency Committee of Water Resources.

With the determination of suspended load estimates, unmeasured load estimates are

necessary to determine total sediment load quantities. Various methods are presently used that include estimating bedload, bedload equations, total load methods, and extension of total load method results.

Estimating Bedload or Unmeasured Load

E. W. Lane and W. M. Borland presented a paper dealing with estimating bedload, which is a suitable guide (*8*). The following discussion and table are quoted from that paper.

The usual method of determining the amount of bedload correction is to assume that it is a certain percent of the load determined from the suspended load sediment samples. The percentages which have been used in the past have been based on judgment, which in many cases have been little better than a guess. Opinions differ considerably on the correct value of this percentage to assume. For different streams, percentages which have been used by various engineers range from 0 to 100 percent, and it is probable that the correct value for these streams covers a range of at least this magnitude. Very few quantitative measurements of the total load have been made to serve as a guide in making accurate estimates of bedload, but there are certain general relations which, if kept in mind, will greatly narrow the limit of error which an engineer is likely to make in estimating the bedload. There are three major variables which affect the amount of bedload a stream may carry. They are as follows:

1. Size of bed material or the fall velocity of these particles.
2. Slope of the stream or the average stream velocity.
3. Nature of the channel, including the depth, size, shape, and roughness of its bed and banks.

A minor factor is temperature, including the effect of ice. To aid in the evaluation of the effect which these variables have on the bedload the following criteria are set forth:

Criterion 1. The smaller the actual concentration of suspended material, the higher usually will be the percent of bedload. This is a very important factor in estimating bedload.

Criterion 2. The smaller the difference in the particle sizes of the bedload material and the suspended load material, the higher will be the percentage of bedload to the total load.

Criterion 3. The ratio of bedload to suspended load is apt to be larger for low or medium stages than it is for high stages. Thus, a stream in which the flow does not fluctuate widely, is likely to carry a larger percent of bedload. This criterion does not necessarily apply to very steep mountainous streams which move boulders at flood stage. In such cases, the bedload ratio may increase with an increase in discharge.

Criterion 4. Streams with wide shallow channels carry a higher proportion of sediment as bedload than streams with deep narrow channels.

Criterion 5. Stream channels with a high degree of turbulence tend to have smaller amounts of bedload.

A clue to the magnitude of the bedload correction can often be found in the nature of the source of the sediments involved. The three common types of erosion which produce sediment are: 1. Sheet erosion, 2. gully erosion, and 3. bank cutting. Sediment produced by sheet erosion tends to be fine; and, therefore, streams carrying sediment derived from this source tend to have a small amount of bedload. However, if the topsoil is predominantly sandy and the drainage-ways are comparatively small, then shallow-sheet erosion may produce considerable bedload. Gully erosion and bank cutting are likely to produce coarser material and, therefore, more bedload than is obtained from sheet erosion. However, if the material below the surface layer of the ground is fine or is composed of material which breaks down into fine material, as does shale, gully erosion or bank cutting do not produce a large amount of bedload. Since bedload is composed of the coarser particles, in order to have an appreciable amount of bedload, these coarser particles must be found in abundance on the watershed and must be so located that they can be easily moved in to the drainage-ways of the stream's channel system.

For some purposes it is desirable to estimate the bedload movement at a given time, and for others it is necessary to estimate the average movement over a period of years. Most of the conditions previously discussed deal more particularly with the conditions at a given time. When the average bedload over a period of time is concerned, the flow variability must be considered. Most of the sediment load of streams is carried during high-flow periods since at these times both the discharge and the sediment concentration are high. At times of high flow, however, the depth of flow and turbulence are greater and tend to lower the bedload content of the stream (at least in the case of the sand sizes, the amount of which is likely to be of greater magnitude than the coarser material). The bedload correction in times of high flow is, therefore, likely to be a smaller part of the suspended load than in times of low flow; consequently, with a more variable flow the bedload correction will be smaller, when expressed as a percent of the total quantity of measured suspended load for the period of flow involved.

From the foregoing discussion it can be seen that a great many factors are involved in the estimation of bedload and that it is difficult, if not practically impossible, to draw up a simple rule or formula which will give quantitative answers to all cases. Probably the best answer to the problem devised to date is that suggested by Thomas Maddock, Jr., and given in Table 1. This table takes into account only a portion of the variables previously mentioned, but those omitted can be given consideration in determining that part of the suggested bedload range to adopt.

Bedload Equations

Numerous bedload equations have been proposed over the years, such as Einstein (*4*), Schoklitsch (*13*), Kalinske (*7*), Meyer-Peter and Muller (*11*), Straub (*17*), and many others. It is necessary for the practicing engineer to have some knowledge of the data, methods, derivation, and limitations of the various bedload formulas before they are applied to a particular situation. Most of the formulas are based on results of the tractive force flume studies of Gilbert, Kramer, the U.S. Waterways Experiment Station, and others. The Schoklitsch (*13*) formula, however, was developed for coarser materials on European streams and was verified by measurements on these streams. The main deterrent to verification of most of the formulas in natural stream channels is the lack of suitable methods for measuring the actual bedload movement. The Geological Survey has made several studies where natural or artificial contracted sections have been used and the created turbulence was sufficient to

TABLE 1.—*Maddock's classification for determining bedload*

Concentration of suspended load	Type of material formingthe channel of the stream	Texture of the suspended material	Bedload in terms of suspended load
Parts per million			
Less than 1,000..	Sand.....	Similar.... to bed material.	25 to 150
Less than 1,000..	Gravel, rock, or consolidated clay.	Small..... amount of sand.	5 to 12
1,000 to 7,500...	Sand.....	Similar.... to bed material.	10 to 35
1,000 to 7,500...	Gravel, rock, or consolidated clay.	25 percent. sand or less.	5 to 12
Over 7,500.....	Sand.....	Similar.... to bed material.	5 to 15
Over 7,500.....	Gravel, rock, or consolidated clay.	25 percent. sand or less.	2 to 8

force most of the normal bedload into suspension where it could be sampled (*3,6*). Such contractions are not available on a sufficient number of streams of varied character to establish comprehensive criteria for bedload formula use, however.

Several formulas that apply to a situation can be used and the separate results compared. In this way the limitations and variance of each is developed by experience. As an example, the Schoklitsch formula, using a mean depth of section, and the Meyer-Peter and Muller formula, using the maximum depth of section, generally result in comparable bedload values for a given set of conditions. This is likely the result of data used in formula derivation; Schoklitsch based on natural European streams and Meyer-Peter and Muller based on laboratory flume studies.

An additional problem is created by the use of bedload formulas to represent the unmeasured load of a natural stream. Most basic data on which the formulas are based are the quantities of transported material. The flume experiments when conducted through a series of discharges did not consider that higher discharges would carry an increasing part of the bed material as suspended load. A formula based on such data could compute more than the unmeasured load because, if all other conditions were the same, the bed material in suspension would be computed as unmeasured load and also sampled as measured load.

Even though there are limitations to the use of bedload formulas, they are adaptable to changing hydraulic conditions in the stream channel and can be used to estimate loads for a variety of problems associated with project development and planning.

Comprehensive Total Load Methods

There are several methods, used in present practice, that compute the total load of a stream as an integrated load of both suspended and bed transport. E. M. Laursen developed parameters by descriptive analysis that are involved in sediment transport and established relationships between the parameters from approximations and empirical definitions (*9*). However, use of the Laursen computational procedure has not been successful on some major natural streams without revision (*10*).

The Modified Einstein procedure is another computational method which results in an estimated total sediment load (*3, 15*). The unique feature of this method, and possibly one of the main reasons for its acceptance among field engineers, is the use of field sampling measurement data in the computation. The Laursen computation used hydraulic and sediment parameters to result in a total sediment concentration. The Modified Einstein procedure uses the sampled concentration of suspended sediment and expands or integrates to determine the rest of the total load. The total load results by the Modified Einstein procedure are correlated to observed data, but the Laursen method computes what can be directly measured.

The Modified Einstein method can be used only in the middle range of sediment transport; i.e., where the transported materials are primarily the same in suspension and along the bed. To integrate the suspended load through the full stream transport depth, including bedload movement, the same size of particles must be found in both modes of transport. If the suspended particles are appreciably smaller than the bed material, the particles are probably transported as wash load. In such a case, separate computations of bedload and suspended load are warranted instead of a comprehensive total load method.

The necessary data and measurements required for a Modified Einstein computation are greater than those for a suspended sediment sampling program, because sampling of the streambed material and a size analysis of both bed and suspended particles are necessary. Perhaps the largest increase in cost of a total load program in comparison to a suspended sampling program is in the time and expense necessary to make the Modified Einstein computations. An engineer or engineering technician will require 2 to 4 hours to make one computation by hand. The Bureau of Reclamation has developed the Modified Einstein procedure for use on an IBM 650 electronic computer, thus saving engineering time but still requiring additional expense (*16*). From these

considerations, it should be clear that a total sediment load program can be economically justified only for problems of considerable importance or for project planning on the higher design levels.

The end result of a Modified Einstein computation is a total sediment load for one discharge value similar to the suspended sediment load for one discharge value of a suspended sediment sampling. The methods used to derive average loads as described for suspended sediment sampling data are, therefore, equally adaptable to total load data. The same limitations and qualifications are also applicable.

The critical determination that affects the total sediment load in a Modified Einstein computation is the rate of change of particle concentration with depth denoted as the Z value. Einstein (*5*) has expressed the concentration at any distance y from the bed, C_y, as a function of a known concentration at distance a as follows:

$$C_y = C_a \left(\frac{d-y}{y}\,\frac{a}{d-a}\right)^Z$$

Various methods have been developed to determine the Z values for various size fractions of the total load. Einstein (*5*) proposed the use of the formula:

$$Z = \frac{V_s}{0.4u_*}$$

Point sampling measurements have shown the Z value from this form of equation is too high. Corresponding Z values have been determined at natural sections for measurements at contracted sections by the U.S. Geological Survey. The U.S. Bureau of Reclamation has attempted point sampling measurements on the Colorado River, which have not been successful. The Modified Einstein procedure determines Z values by computing a concentration C_a near the bed level by the bedload function and integrating across the unmeasured depth to agree with the measured suspended concentration. None of these methods have proved completely successful. Until some method is devised to predict the Z value for other size fractions from the Z value of one size fraction and this method agrees with sampling data, the Modified Einstein computation will continue to contain inconsistencies.

The Corps of Engineers sampling program on the Missouri River has been devised to compute total suspended load by the collection of an equal transit rate sample for the whole stream and a point sample in each vertical. These two samples define a measured Z value for extending the suspended concentration to the bed, but the method does not include any bedload transport. This method has been adopted because, in the Missouri River, bedload is not a large part of the total transport. Some major streams of concern to the Bureau of Reclamation, such as the lower Colorado River, carry an average of 40 percent unmeasured load, and the approximation by the Modified Einstein procedure is considered a better estimate for these streams than a total suspended load value.

In any case, much more investigation is needed to establish the factors affecting the rate of concentration change with depth in natural streams. The Modified Einstein method is still useful as it is, because the Z values for the size fractions that make up the major part of the total transport can be determined to some degree of accuracy, although the relation between the various Z values is somewhat vague.

Velocity and Unmeasured Load Curves

B. R. Colby presented a relation of unmeasured sediment load to mean stream velocity as an approximation for determining total sediment load (*2*). The Bureau of Reclamation has checked this relation on a number of streams where Modified Einstein computations were available to derive such a curve. The main premise of the Colby relation is that most of the variables used in the Modified Einstein procedure have a relation or are affected by the mean stream velocity. The unmeasured load should, therefore, show a strong correlation with mean velocity. The stream width has been eliminated in this correlation by using the unmeasured load per foot of width.

Available total load results by the Modified Einstein procedure were used to determine individual curves of mean velocity vs. unmeasured load per foot of width for the following:

Rio Grande:
- Albuquerque, N. Mex.
- Belen, N. Mex.
- San Felipe, N. Mex.
- Bernalillo, N. Mex.
- Cochiti, N. Mex.
- Miscellaneous measurements.

Colorado River:
- Northerly International Boundary.
- Adobe Ruin.
- Taylor's Ferry.
- Palo Verde Dam.
- Water Wheel.
- R. S. 43.
- R. S. 41.
- Needles Bridge.
- R. S. 33.
- Columbus, Tex.
- Bay City, Tex.

Guadalupe River:
Victoria, Tex.
Sabine River:
Bon Weir, Tex.
Trinity River:
Romayor, Tex.
Brazos River:
Richmond, Tex.
Elk Creek:
Hobart, Okla.

Figure 1 shows a composite plot of the derived curves for the listed gaging stations, although the individual curves cannot be discerned.

The 57 Modified Einstein computation results for the Rio Grande at Cochiti, N. Mex. (fig. 2) were processed for a statistical correlation to approximate a curve of the following form:

$$Y = CX^D$$

From the correlation, it was found that 90 percent of the deviation of unmeasured sediment load from the curve could be explained by velocity variation. From statistical variation, 18 of the points would be expected to fall outside one standard deviation from the curve. For the Cochiti correlation, only 14 of the 57 values fell outside one standard deviation.

In recent studies of sediment movement in the Lower Colorado River at the northerly International Boundary, the flow-duration, total sediment load curve method of computing the yearly loads, was checked by using suspended sediment data and a derived curve of mean velocity vs. unmeasured load per foot of width based on 50 total load computations (fig. 3). Over a period of 5 years, the total accumulated sediment load determined by the two methods differed by only 3.5 percent.

Based on the previous studies and additional experience with unmeasured load curves, the Colby relation appears to be a valuable tool for use in total load estimates. The basis of the relation is total loads as determined from Modified Einstein computations, and any application should be in a situation where the Modified Einstein method is applicable. It does allow a total load investigation with less data than necessary to establish a total sediment load rating curve and can approximate conditions for similar streams where no data are available. However, sufficient data should be collected to define such curves based on the level of project development being investigated.

Additional unmeasured sediment load curves are shown in figures 4, 5, 6, 7, 8, and 9 for several of the listed gaging stations. It is hoped

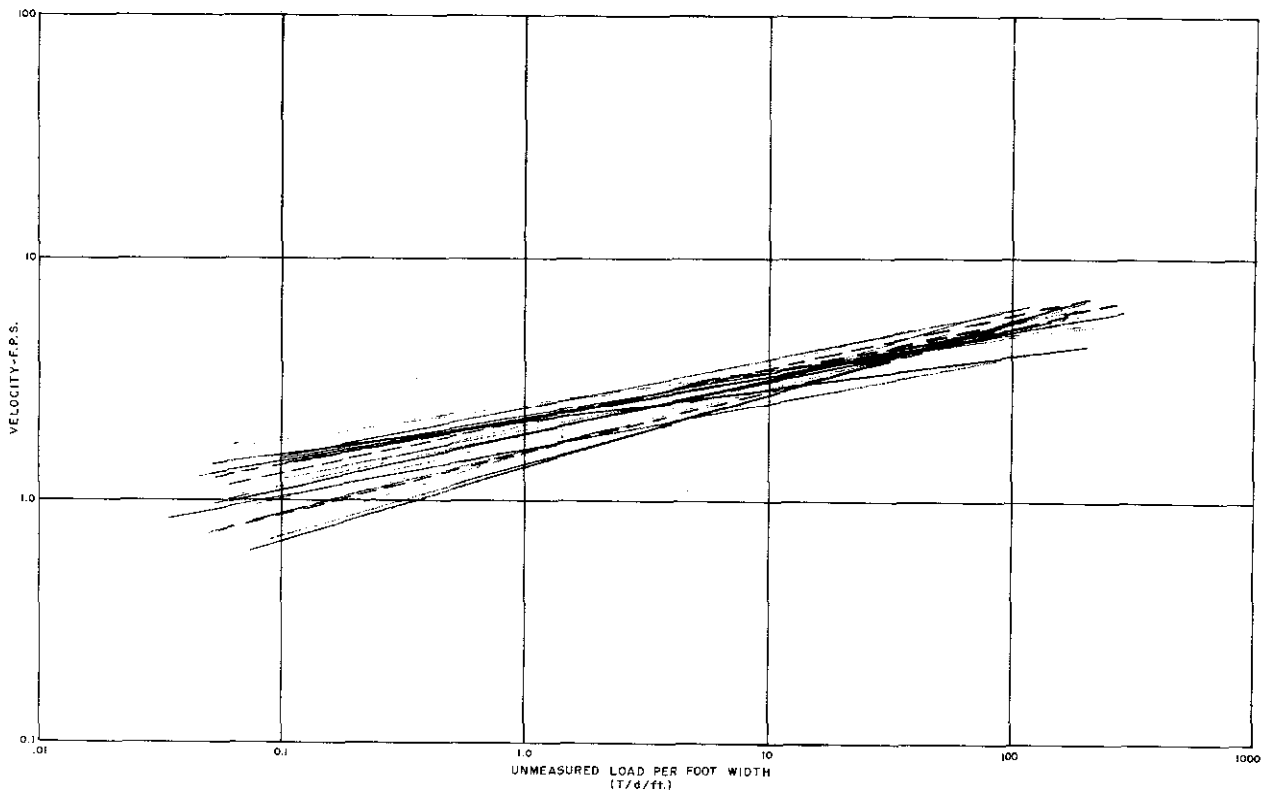


FIGURE 1. — Composite plot of relation between unmeasured sediment load and mean velocity.

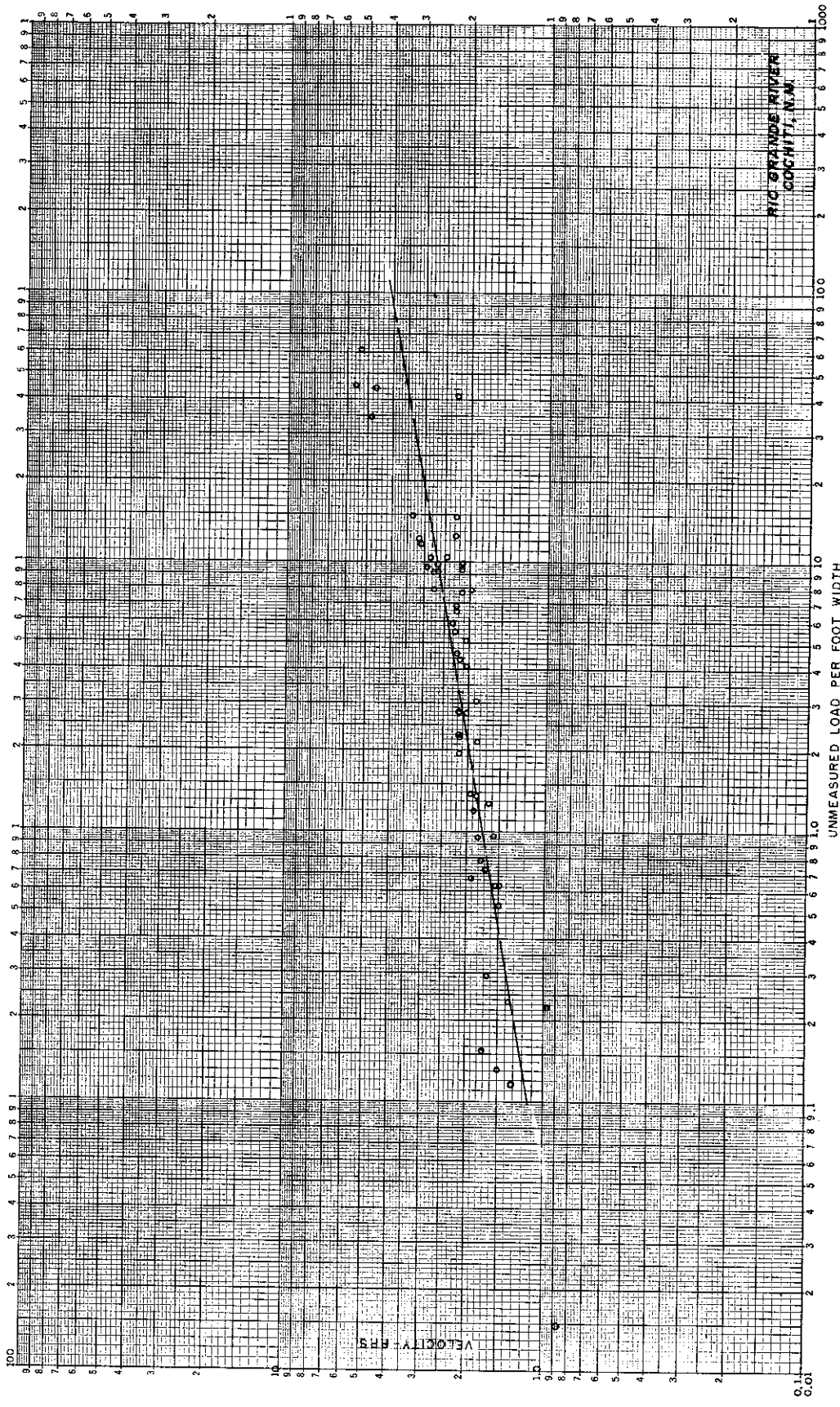


FIGURE 2. — Relation between unmeasured sediment load and mean velocity, Rio Grande, Cochiti, N. Mex.

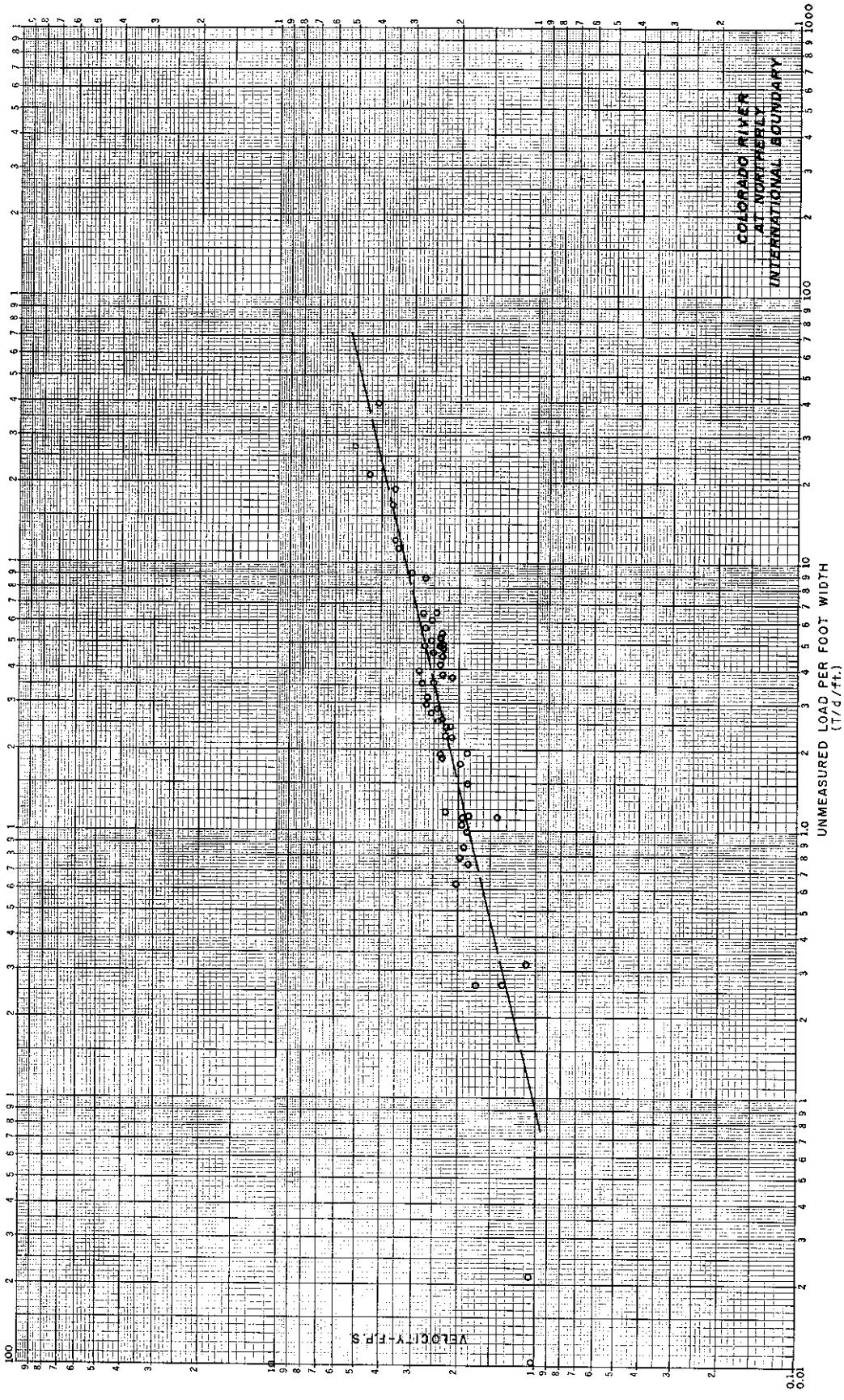


FIGURE 3.— Relation between unmeasured sediment load and mean velocity, Colorado River at northern International Boundary.

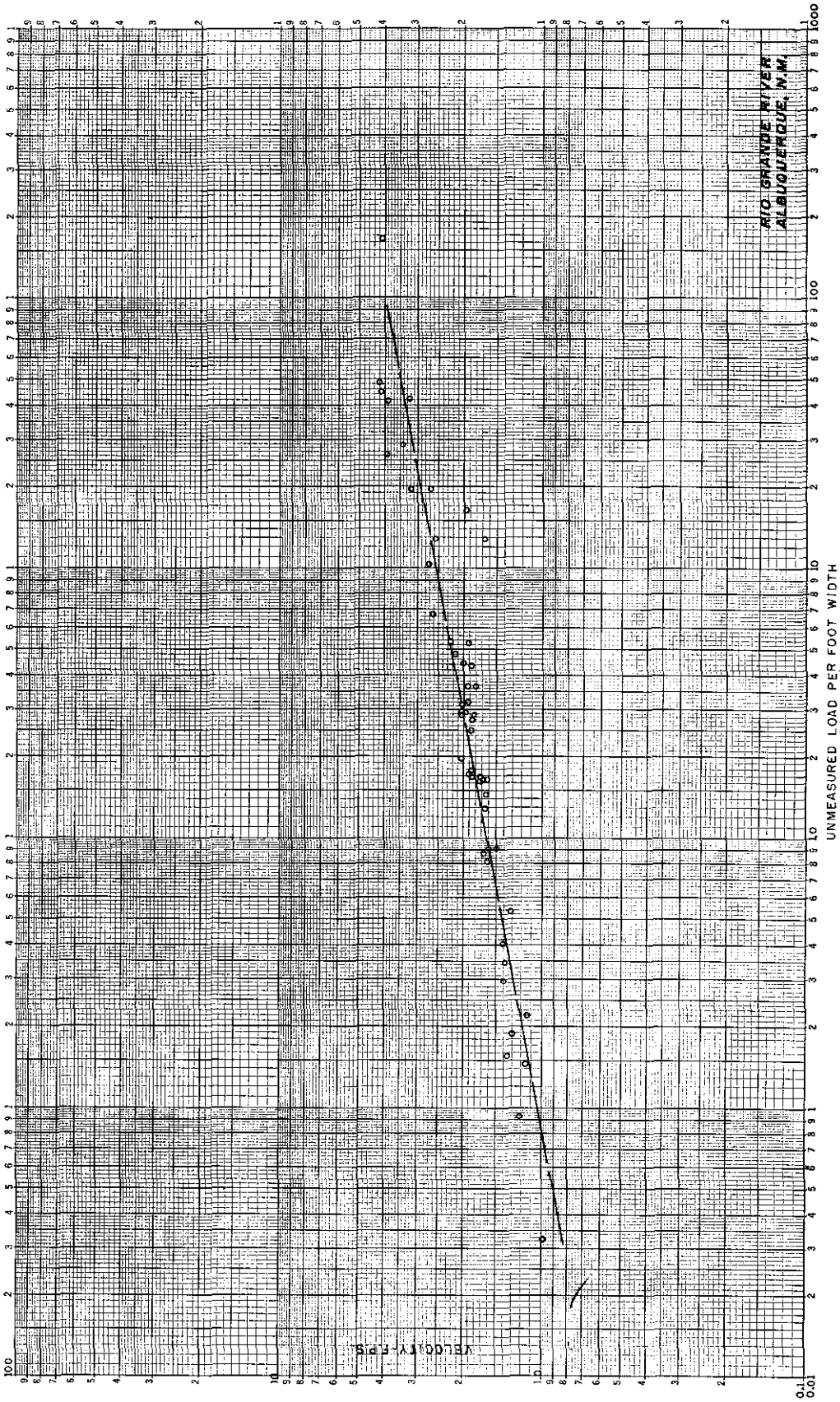


FIGURE 4.—Relation between unmeasured sediment load and mean velocity, Rio Grande, Albuquerque, N. Mex.

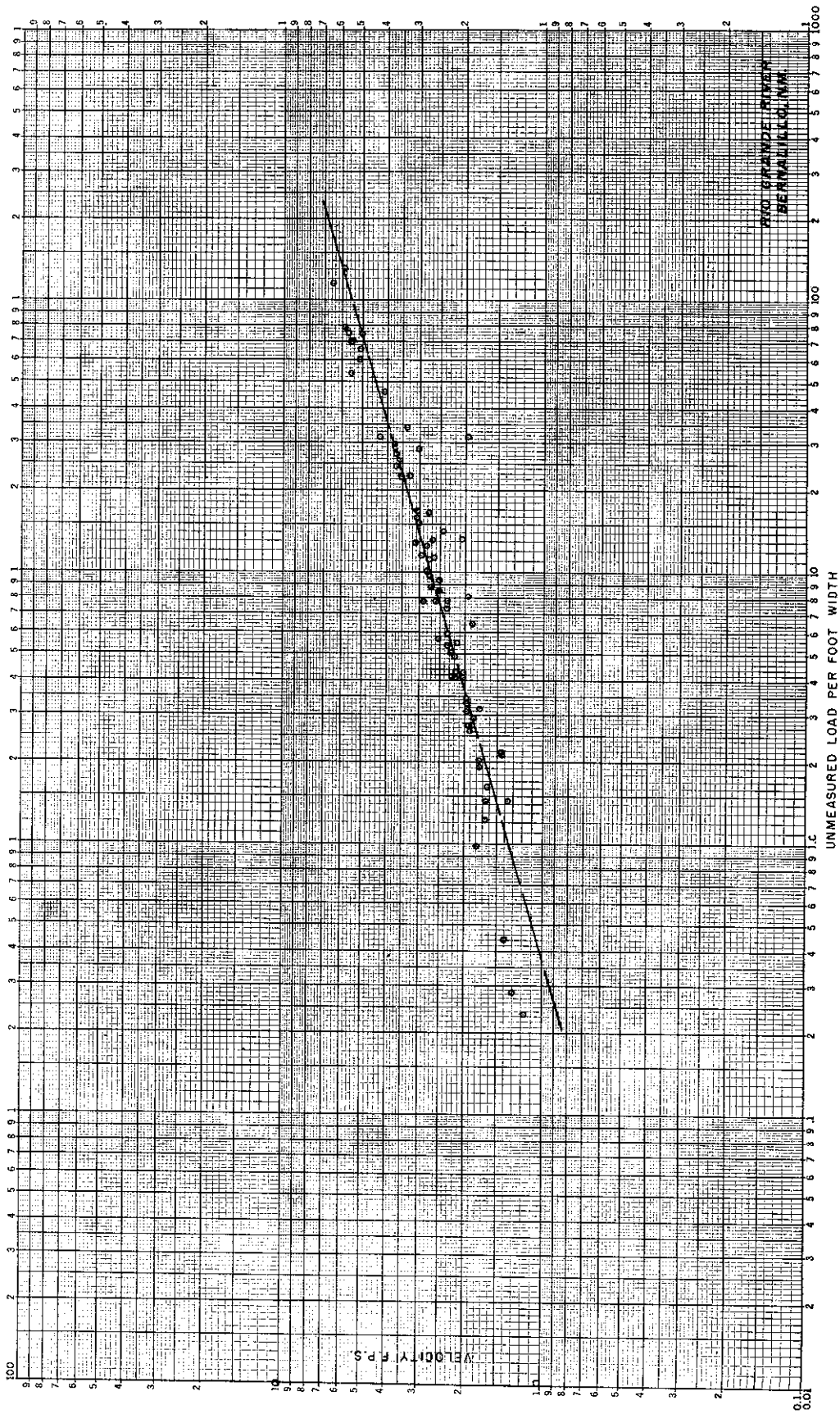


FIGURE 5. — Relation between unmeasured sediment load and mean velocity, Rio Grande, Bernalillo, N. Mex.

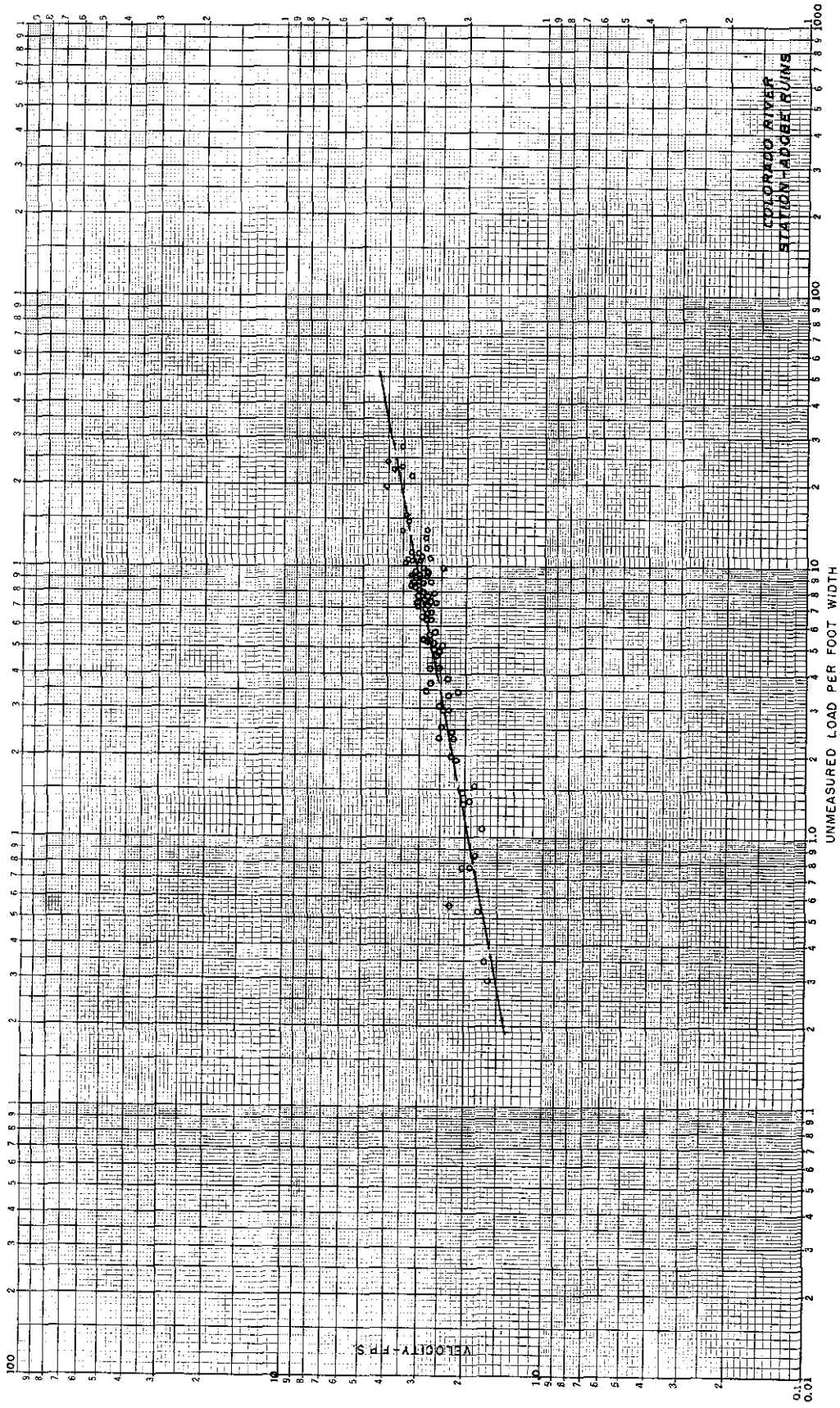


FIGURE 6.—Relation between unmeasured sediment load and mean velocity, Colorado River, Adobe Ruin Station.

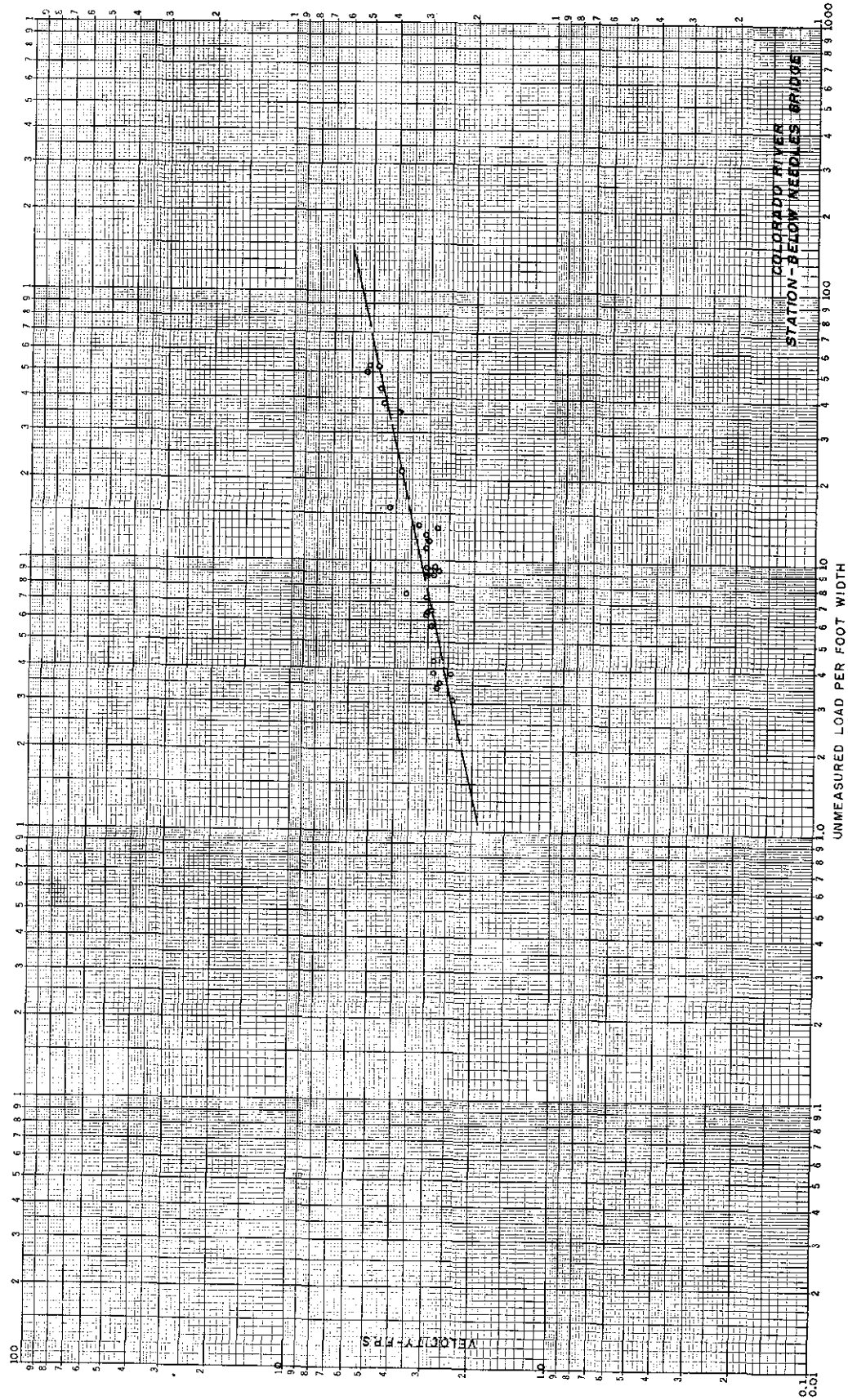


FIGURE 7. — *Relation between unmeasured sediment load and mean velocity, Colorado River, station below Needles Bridge.*

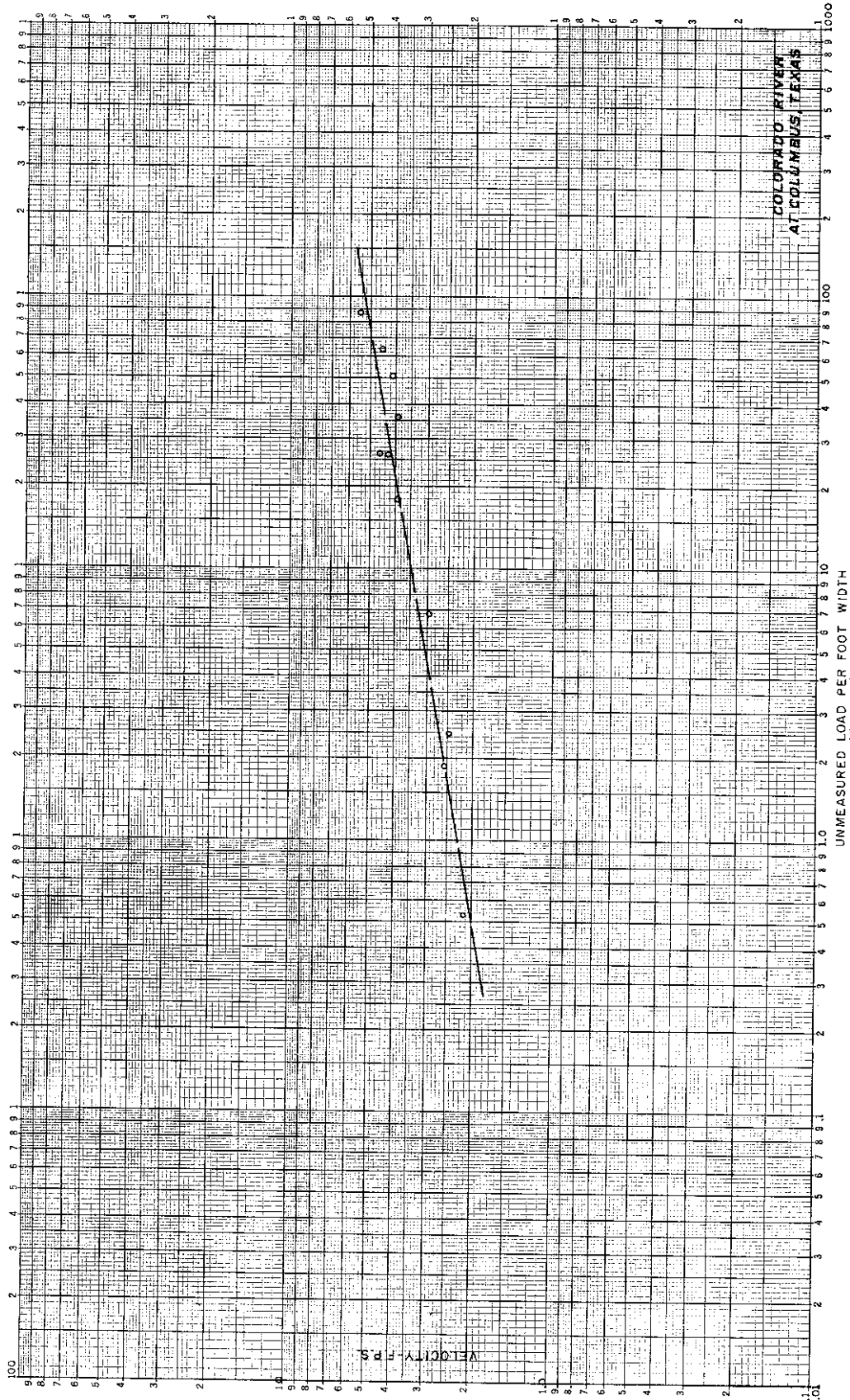


FIGURE 8.—Relation between unmeasured sediment load and mean velocity, Colorado River, at Columbus, Tex.

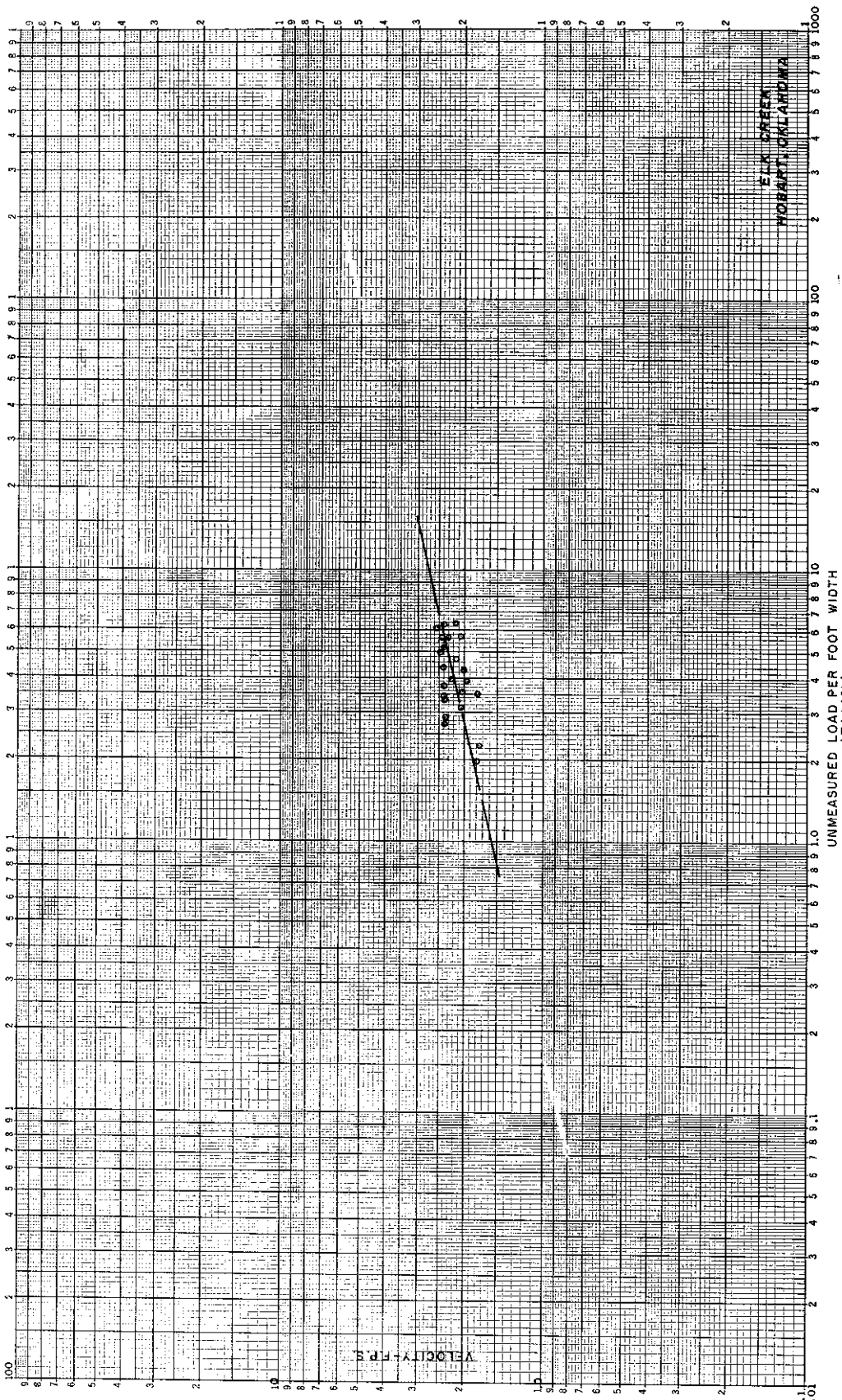


FIGURE 9.— Relation between unmeasured sediment load and mean velocity, Elk Creek, Hobart, Okla.

that in the future, the series of curves shown in figure 1 may be studied to determine variation in slope and position with additional stream or sediment characteristics.

Conclusion

The determination of total sediment quantities during the investigation phase of a water resource project is an important and often vital part of the design and operation of the project. Sediment can produce problems with major or minor structures where the transport of water is involved. This paper has been prepared as a guide to the methods generally used in Bureau of Reclamation practice to determine total sediment quantities in natural streams. The main purpose has been to present these methods not in complete detail, but in the frame of ideas for a practical determination of quantities. Sufficient references have been included if the reader wishes to study the methods in detail.

The prediction of greatly increased water resource needs for the future carries with it the implication of continued and increased concern with transported sediment. The methods presented in this paper may help in this concern, although new investigations, advancements, and studies will be necessary to provide the practicing sedimentation engineer with the necessary tools.

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SEDIMENT CONTROL AT A HEADWORKS USING GUIDE VANES

[Paper No. 33]

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Synopsis

The Bureau of Reclamation has in progress a general study program concerning the control at canal headworks of coarse sediments diverted from alluvial streams. The study described in this paper was conducted as one phase of the general program. Its purpose was to develop satisfactory arrangements of bottom and surface guide vanes, to compare the performance of each type, and to evaluate the effectiveness of each type in reducing sediment intake into a canal diverted from a large river. Thirty-seven tests were made on one type of canal entrance with a standard river and diversion discharge.

Tests indicated that both bottom and surface

vanes, as shown in figures 7 and 9, are effective in reducing sediment intake into a canal diverted from a large river. With either bottom or surface vane operation, it was possible to reduce the quantity of sediment entering the model canal to approximately one twenty-third of the quantity entering without the vanes in place.

Because the vanes are not overly sensitive to the tested variables, including length, spacing, placement, and depth, the information in this paper should be of value in design work. For intake conditions similar to those used in these tests, guide vanes having dimensions proportional to those given in figures 7 and 9 should prove beneficial. Results of these tests show that guide vanes to control sediment movement can be developed by means of model tests.

Introduction

Sediment removal from canals, laterals, and farm ditches is costly maintenance, so that it is desirable, if possible, to exclude or reduce the amount of sediment going into a canal headworks that takes water from a sediment-laden stream. Guide vanes placed near a headworks or sluiceway entrance have been studied [1] as one effective method of controlling sediment movement near the intake. They are used to control localized secondary currents (figs. 1 and 2) by diverting bottom water with its relatively heavy sediment load away from the canal headworks, and top water with its relatively light sediment load through the canal headworks.

Studies to develop and evaluate the effectiveness of bottom guide vanes for a particular arrangement of intake were made for the San Acacia Diversion dam and are reported in the Bureau of Reclamation, Hydraulics Branch Report Hyd–479.[2] A bottom vane arrangement, determined by model tests at the Denver laboratory that resulted in satisfactory performance is described in this paper. At the conclusion of the San Acacia tests, the model was used to extend the data on surface and bottom guide vanes as a research project. Dimensions referred to in this paper are for the San Acacia dam. Prototype dimensions are used in the narrative because they are easier to visualize. Some prototype dimensions are shown in terms of a model scale of 1: 20, in parentheses on the figures, but most of the dimensions are model dimensions and show the actual size of the test facility.

Although the scope of these studies is limited, the research indicated that efficient guide vanes can be developed by means of model studies and that additional research would provide valuable generalized design information.

Experimental Work and Data Obtained

The model (figs. 3 and 4) was constructed in a test box lined with sheet metal. The prototype spillway consisted of 12 river bays with 20-foot by 7-foot 6-inch gates and an adjacent sluiceway area. A movable bed extended approximately 600 feet upstream from the dam axis. The canal headworks was approximately 160 feet upstream from the dam axis and discharged into a canal of 8-foot bottom width and 2:1 side slopes. The low-flow channel used in the San Acacia study (fig. 3) with the five entrance conduits, was not operated in these tests.

Such major features as river gates, conduits, slide gates, and sampling equipment were generally constructed of sheet metal. Treated wood was used for piers between radial gates, and a part of the canal was constructed of metal lath covered with concrete.

A fine sand of near uniform size gradation (fig. 5,*A*) was used to form the movable bed and to represent sediment bedload in the model. The average diameter of the model sediment was approximately 0.2 mm. (millimeter). Figure 5,*B*, shows the settling velocities of the model sand.

Two pumps were used to supply water and sediment to the model. No. 1 pump, positioned at the downstream end of the model (fig. 3), recirculated sediment-laden water through the model. No. 2 pump drew clear water from a laboratory reservoir and supplied a small quantity of water to replace sampling and other losses. This water was introduced into the upstream end of the model and maintained a constant head on pump No. 1. Excess water discharged over a weir at the downstream end of the model. Discharges and water surface elevations in the canal were maintained constant by the use of slide gates at the downstream end of the canal. Backwater was maintained on the radial river gates by the use of the slide gates installed for this purpose downstream from the diversion dam spillway (figs. 3 and 4).

Samples of water and sediment discharging from the canal, the sluiceway, and the river gates were obtained by passing a hand-operated sediment sampler through the discharging water (fig. 4,*B*). The sediment-laden water flowed through the sampler to a volumetric col-

[1] POTAPOV, M., and PYCHKINE, B. METHODS OF TRANSVERSE CIRCULATION AND ITS APPLICATION TO HYDROTECHNICS. Moscow Academy of Sciences, U.S.S.R. Translation 46 of Service des Etude et Recherches Hydrauliques, Paris. 1947.

[2] BUREAU OF RECLAMATION. HYDRAULIC MODEL STUDY TO DETERMINE A SEDIMENT CONTROL ARRANGEMENT FOR SOCORRO MAIN CANAL HEADWORKS—SAN ACACIA DIVERSION DAM, MIDDLE RIO GRANDE PROJECT, NEW MEXICO. Hydraulics Branch Report Hyd-479. March 1962.

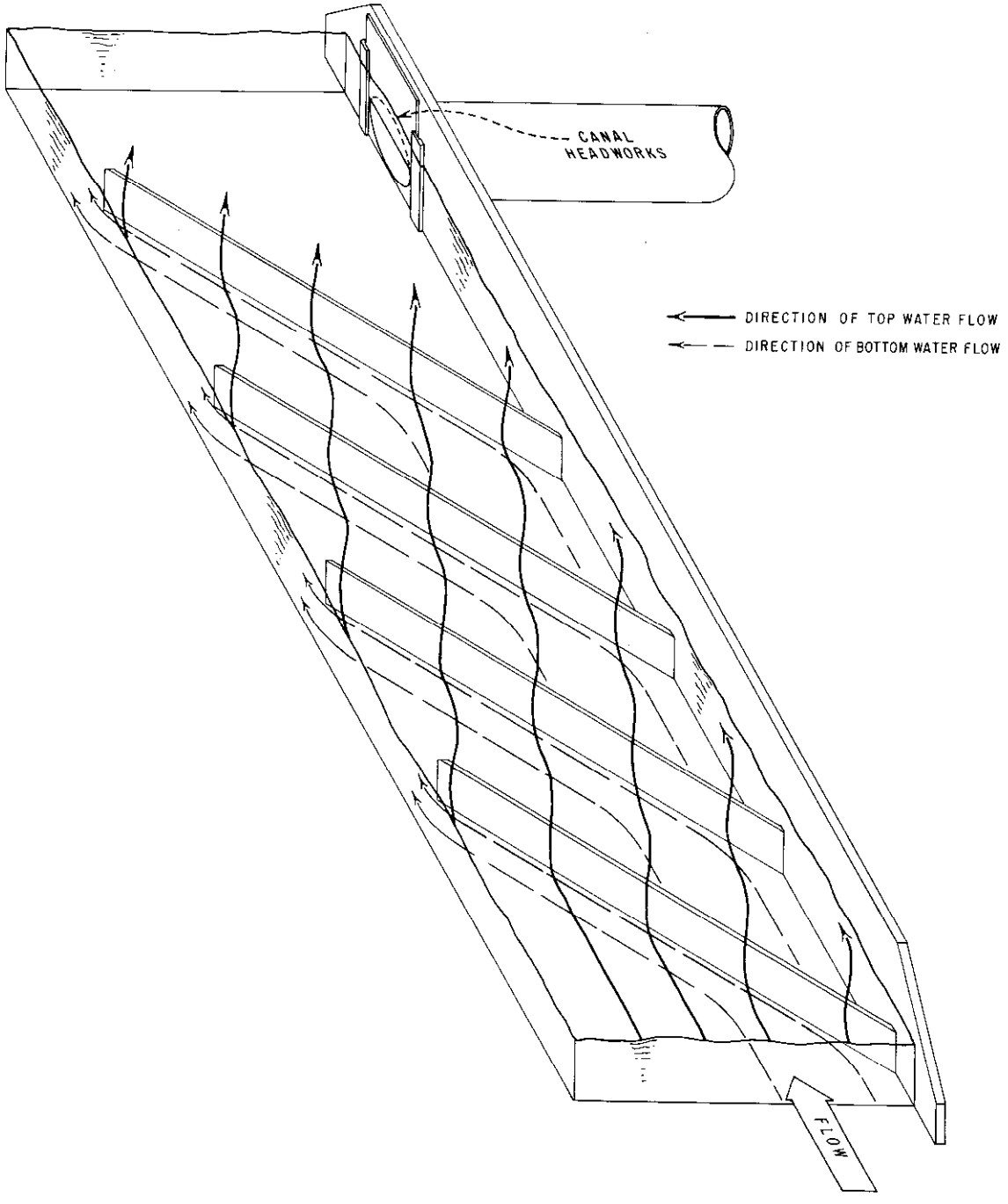


FIGURE 1. — Bottom guide vane method of producing secondary currents for sediment control.

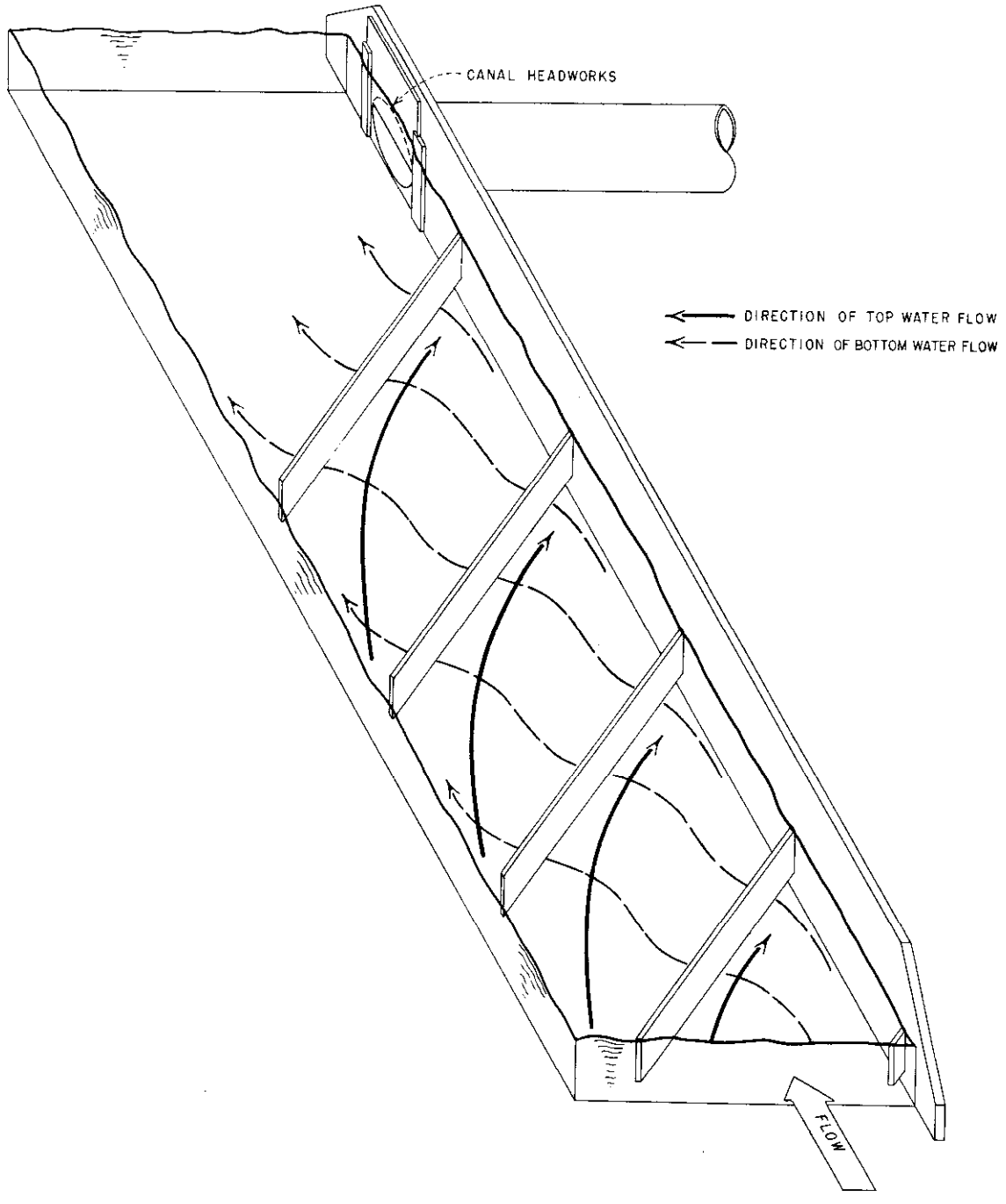


FIGURE 2. — Surface guide vane method of producing secondary currents for sediment control.

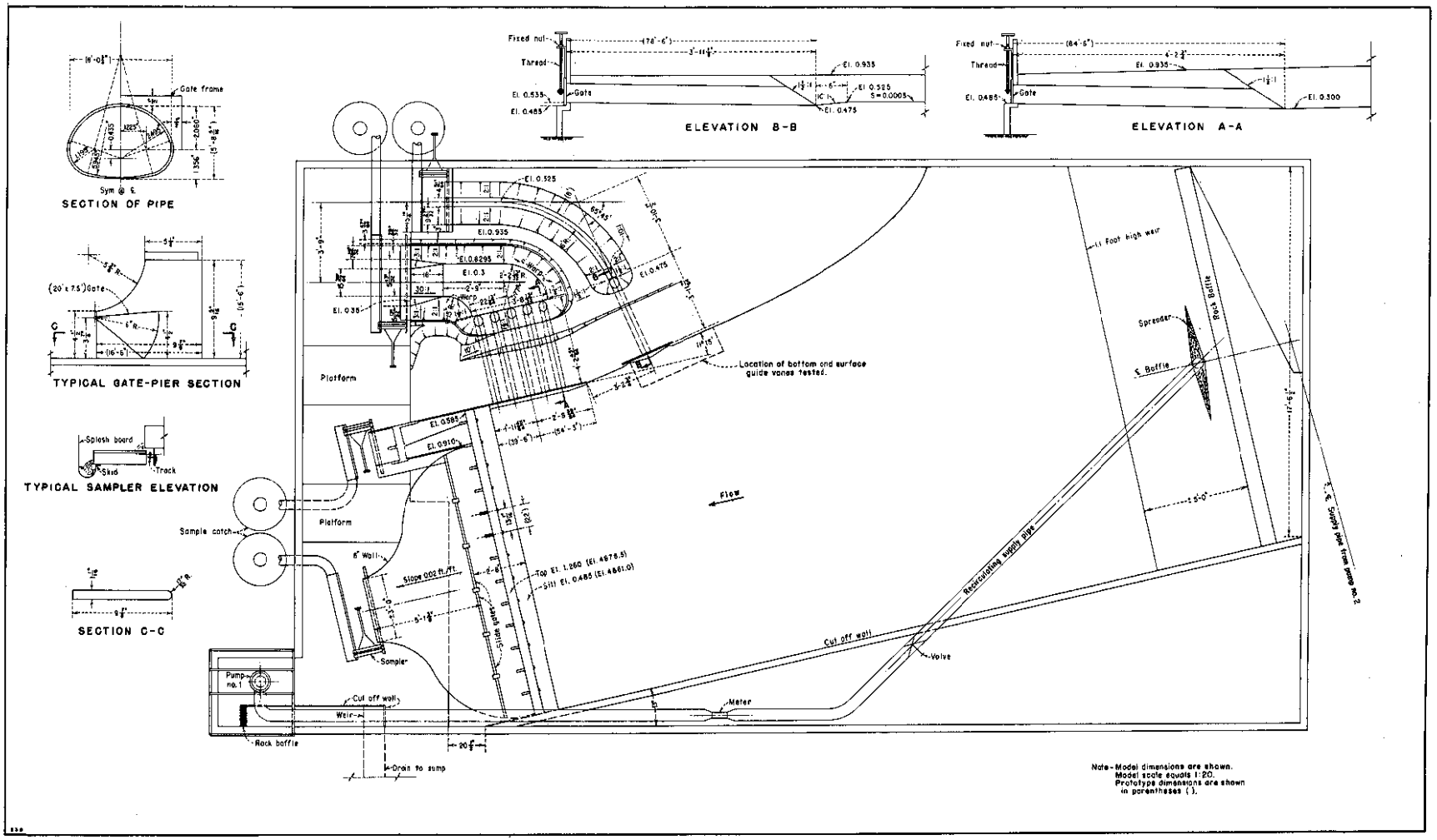
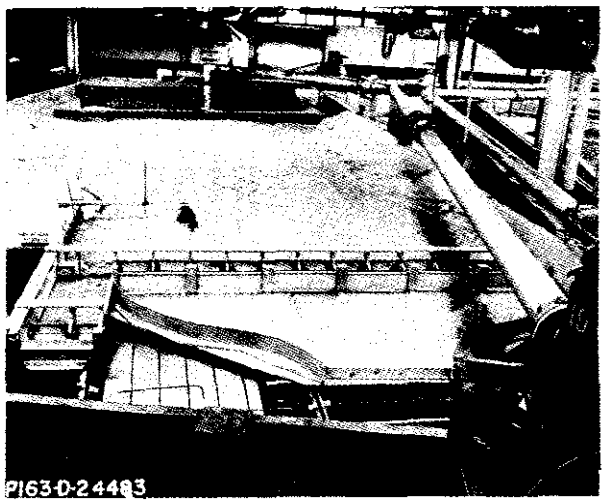


FIGURE 8. — General plan and sections of hydraulic model.



(a)
Overall View of Model



(b)
Sample Being Obtained from River Flow

FIGURE 4. — *A*, Overall view of model; and *B*, sample being obtained from river flow in hydraulic model study.

lector calibrated to indicate the amount of water and sediment in liters. After the sediment had settled into the small funnel at the bottom of the collector, its volume was determined. Thus, the concentrations of sediment passing through the sluiceway or the canal could be readily determined at any time during a test.

During a test some sediment deposited and remained in the canal. To account for these deposits that had entered the canal but had not been accounted for in the sampling process, the volume of sediment remaining in the canal was also measured. The sediment concentration was therefore based on the discharge and the average sediment concentrations that passed through various parts of the model, taking into account the amount of sediment deposited in the canal.

For all tests, the water surface elevation just upstream from the dam was held at 4,668.7 feet; the tailwater elevation below the radial river gates was held at 4,667.6, to correspond to the tailwater used in the San Acacia model study. The canal intake gate was calibrated while holding the headwater at normal elevation 4,668.7 and maintaining the canal water surface at the calculated normal elevation for the discharge. A standard test discharge, similar to that used on the San Acacia model study, consisting of 8,760 cubic feet per second in the river and 174 cubic feet per second diverted to the canal, was used throughout the study (1:20 model scale).

When contours of the movable bed configuration at the end of a test were desired, levels were obtained and appropriate plots made. To help evaluate results, both black and white and color photographs of sediment deposits and bed conditions were obtained for each test. Tests were compared on the basis of a ratio of the concentration of sediment entering the canal headworks to that moving in the river upstream from the headworks:

$$R = \frac{C_c}{C_{rus}}$$

where:

R = Concentration ratio;

C_c = Concentration in parts per million, by weight, of sediment in water entering the canal headgates;

C_{rus} = Concentration in parts per million, by weight, of sediment in the river water upstream from the canal headworks.

Thus, lower values of the ratio indicate a more satisfactory sediment exclusion device.

To develop a satisfactory set of vanes for the standard test discharge, it was assumed that for a given number of vanes having an established cross section, a satisfactory vane spacing for a given vane angle, location, length, and elevation would still be satisfactory if one of the other factors was varied. Accepting this assumption for all the variables involved allowed the following test plan to be adopted:

1. Determine satisfactory vane spacing.
2. Determine satisfactory vane angle.
3. Determine satisfactory placement of vanes with respect to canal headworks.
4. Determine satisfactory vane length.

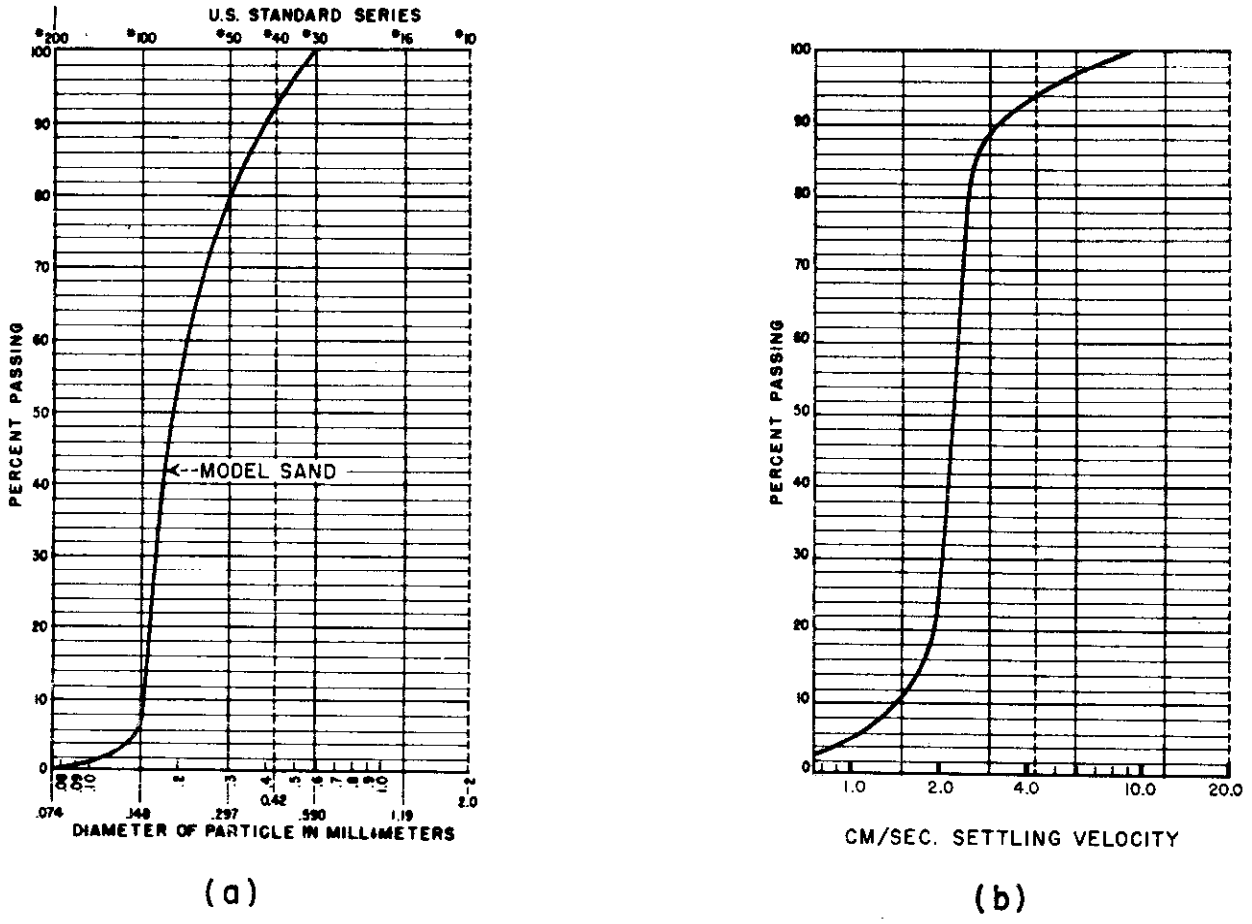


FIGURE 5. — *A*, Sand gradation analysis; and *B*, settling velocities in hydraulic model study.

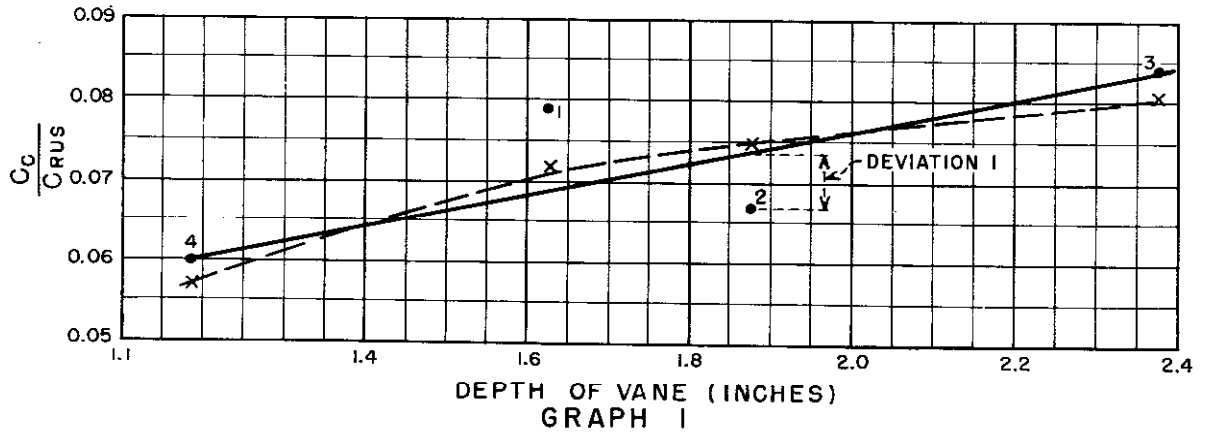
5. Determine satisfactory vane elevation or vane depth.
6. Determine effect of the number of vanes.
7. Determine effect of vane cross section.

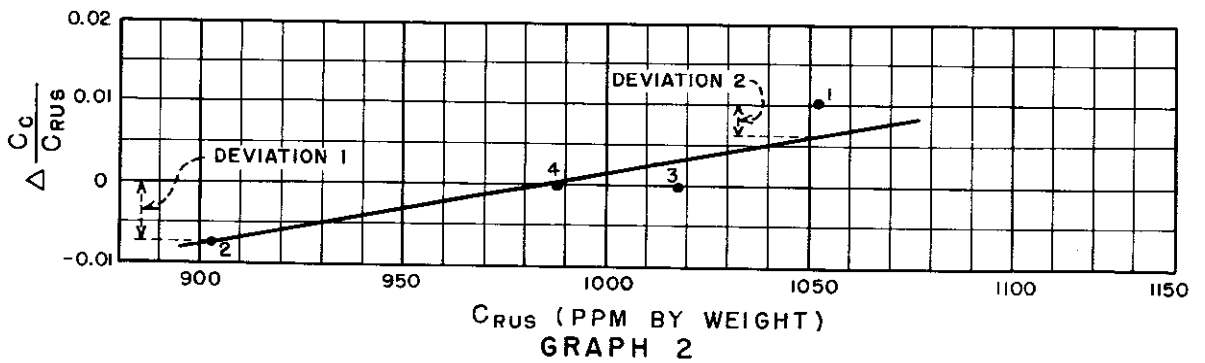
A graphical method of correlation analysis presented by Ezekiel[3] was used in analyzing results. In this method, a number of variables, such as the concentration ratio, any of the vane variables, concentration of total sediment moving in the river, and concentration of sediment moving near the headworks, are considered in the evaluation of a particular arrangement. The concentration ratios are first plotted as a function of the variable of immediate interest; for example, depth of vane (fig. 6). All points on the plot are numbered for future reference. The best average straight line is fitted to the points in graph 1, solid line. Deviations of points 1, 2, 3, and 4 from the fitted line (deviation 1) are then plotted as a function of the next most important variable; in this example, concentration of sediment moving in the river, C_{rus}. From this plotting, a new curve is fitted to the points (graph 2 of fig. 6), which is a correction curve to indicate the effect of the river concentration on the concentration ratio. Deviations from graph 2 (for example, deviation 2) are then plotted as a function of the variables of next importance; in this example, concentration of sediment in the sluiceway is C_s. From the resulting correction curve, C_s is shown to be of minor importance. This process is continued until all the independent variables desired have been introduced.

In the example, the concentration of sediment in the sluice was the last variable considered. A second approximation of the relation of the concentration ratio to the depth of vane may be determined by plotting the deviations from the last correction curve as deviations from the first approximation curve. A curve is fitted to the new points, and, if considerable change results or if a large number of points are available, it may be desirable to repeat the entire evaluation process.

Although a straight line of best fit can

[3] EZEKIEL, MORDECAI. METHODS OF CORRELATION ANALYSIS. Ed. 2. 1941. John Wiley and Sons, Inc.





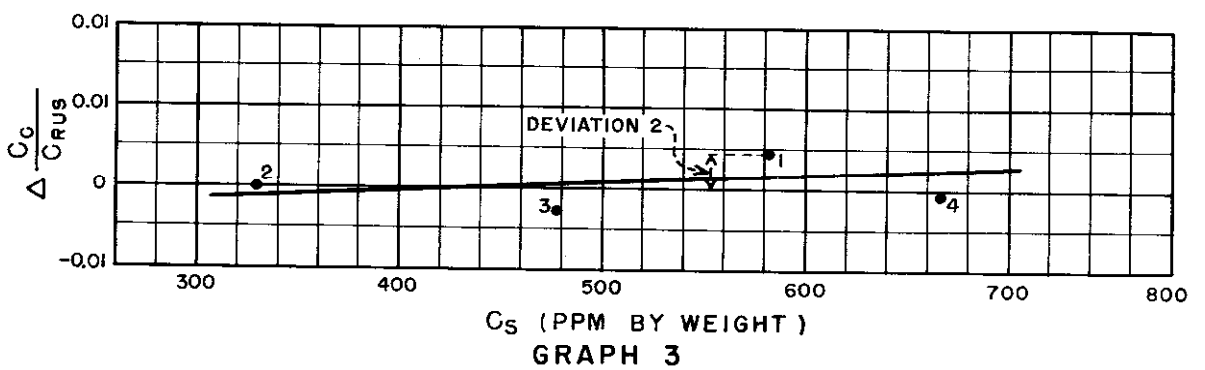




FIGURE 6. — Example of graphical method used for analysis of depth of vane, concentration of sediment in the river water upstream from canal headworks, and concentration of sediment in sluiceway.

usually be drawn to fit the points, this method is not limited to straight lines. If only a few points are involved a straight-line curve is easy and rapid to use. However, if too few points are involved results may be inconclusive. As only a limited number of points were available for these analyses (usually three or four), the conclusions drawn are necessarily limited.

Control Tests

Five tests were conducted without vanes but with a 160-foot by 40-foot 3-inch slab, at eleva-

tion 4,661.0, near the canal headworks on which the bottom vanes were later constructed. The standard discharges were used in the control tests and the river discharge was passed through all the river gates, which were opened equally. The duration of the five control tests averaged approximately 6 hours, and the control tests were spaced throughout the overall testing period. The control tests are tests 4 and 19 in the bottom guide vane series and tests 20, 24, and 36 in the surface guide vane series. Results are shown in table 1.

TABLE 1.—*Results of control tests with slab and without vanes for concentration of sediment*

Test No.	Hours test was conducted	Concentration of sediment entering canal headworks	Concentration of sediment in the river	Concentration ratio $\frac{Cc}{Crus}$
	Number	*P.p.m.*	*P.p.m.*	
4.....	7.0	1,601	385	4.16
19.....	4.4	1,635	345	4.74
20.....	8.4	1,589	1,331	1.20
24.....	4.1	886	835	1.06
36.....	5.0	622	861	.72

The average concentration ratio obtained from the control tests was 2.38. This value is therefore the datum used to determine the improvement resulting from the various guide vane arrangements. In the control tests, sediment deposits in the canal decreased the effective cross section of the canal and resulted in a gradual decrease in discharge as the test progressed. The average decrease in discharge was approximately 34 percent. When the vanes were in place, no appreciable decrease in canal discharge occurred.

Tests Conducted with Bottom Vanes

Bottom Vane Spacing

Four tests were made to determine a satisfactory vane spacing for tests using bottom vanes. These are shown in table 2 as tests 1, 2, 3, and 5 (test 4 was a control test). Vane arrangements for the tests are shown in figure 7, and the test data are summarized in table 2.

TABLE 2.—*Results of tests to determine satisfactory vane spacing with bottom vanes*

Test No.	Hours test was conducted	Vane spacing	Concentration of sediment entering canal headworks	Concentration of sediment in the river	Concentration ratio, $\frac{Cc}{Crus}$
	Number	*Ft.-in.*	*P.p.m.*	*P.p.m.*	
1.....	26.2	16–8	41	233	0.176
2.....	51.2	12–0	15	161	.093
3.....	31.2	26–0	34	349	.097
5.....	31.2	20–0	44	262	.168

For these tests the four vanes used were 50 feet long, their top elevation was at 4,665.0 feet, and they were placed at an angle of 40° to the direction of flow with the downstream end of the downstream vane on the canal headworks centerline.

In these tests the concentration ratio was considerably improved from the 2.38 average of the control tests. A multiple correlation of the data indicated the spacing of 26 feet on centers in test 3 to be most satisfactory. For this spacing the concentration ratio was reduced to 0.097 for 349 p.p.m. sediment concentration in the river. The 26-foot spacing was used in all following tests.

Although the value 0.093 in table 2 appears to be more satisfactory than the value 0.097, the multiple correlation methods indicates 0.097 to be better. This results from a number of variables being involved in the correlation. For instance, although the concentration in the river was 349 p.p.m. for test 3, it was only 161 p.p.m. for test 2. However, even the difference between 0.093 and 0.176, the highest concentration ratio obtained in this test series, is not great when compared to the control concentration ratio of 2.38.

Angle Between Bottom Vanes and Direction of Flow

Tests 3, 6, and 8 were utilized to determine a satisfactory angle between the vanes and the direction of flow. The standard test discharge was set, and the multiple correlation method was used for analyzing results. For all three tests the vane length was 50 feet, vane spacing was 26 feet on centers, the tip of the downstream vane was on the canal centerline, and vane top elevation was 4,665.0 feet (fig. 7). No noticeable decrease in discharge occurred in the canal for these tests. Results of the tests are shown in table 3.

TABLE 3.—*Results of tests to determine a satisfactory angle between the vanes and direction of flow*

Test No.	Hours test was conducted	Vane angle to riverflow	Concentration of sediment entering canal headworks	Concentration of sediment in the river	Concentration ratio, $\frac{Cc}{Crus}$
	Number		*P.p.m.*	*P.p.m.*	
3.....	31.2	40°	34	349	0.097
6.....	29.0	35°	65	367	.177
8.....	51.4	45°	33	310	.106

In the previous series of tests the concentration ratio was not sensitive to vane spacing; in these tests the concentration ratio was not overly sensitive to the angle at which the vanes were placed in the river. However, from a multiple correlation analysis of the data, the 45° angle was considered to be most satisfactory. All angles tested indicated considerable improvement in the concentration ratio as compared to the ratio obtained when no vanes were

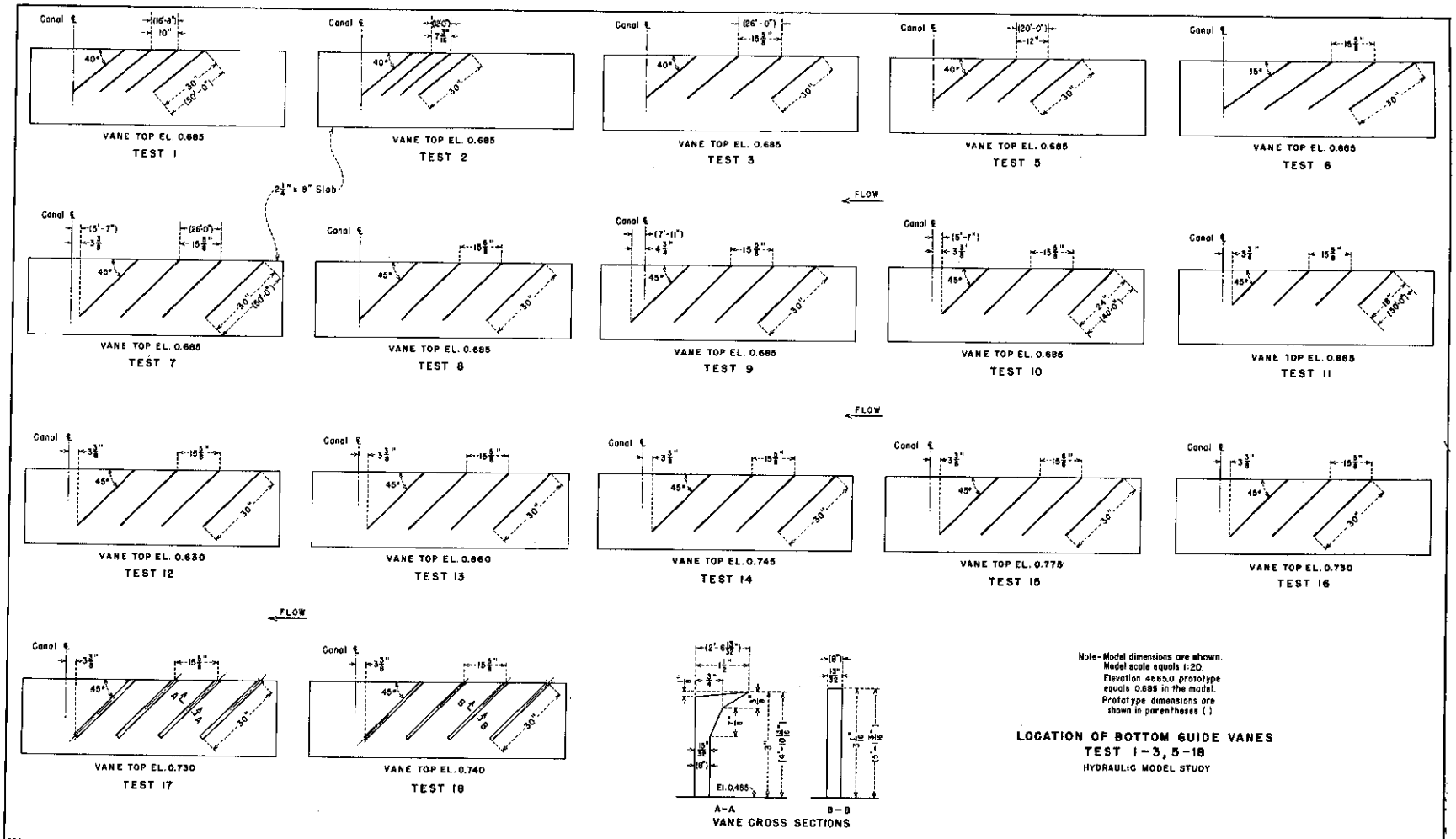


FIGURE 7. — Location of bottom guide vanes for tests 1 to 3 and 5 to 18.

installed. The 45° angle indicated an improvement in $\frac{C_c}{C_{rus}}$ from 2.38 to 0.106.

Bottom Vane Location

Tests 7, 8, and 9 were utilized to determine a satisfactory placement or location of vanes with respect to the canal headworks. The standard test discharges of 8,760 cubic feet per second in the river and 174 cubic feet per second in the canal were used, and the multiple correlation method was used in analyzing results. For all tests, the vane length was 50 feet, vane spacing 26 feet, vane elevation 4,665.0 feet, and the angle of the vane with the direction of flow was 45°. Test results are summarized in table 4.

TABLE 4.—*Results of tests to determine satisfactory placement of vanes with respect to canal headworks*

Test	Hours test was conducted	Tip location of downstream vane [1]	Concentration of sediment entering canal headworks	Concentration of sediment in the river	Concentration ratio, $\frac{Cc}{C_{rus}}$
	Number		*P.p.m.*	*P.p.m.*	
7....	26.8	5 feet, 7 inches upstream from canal (centerline).	22	467	0.047
8....	51.4	At canal (centerline).	33	310	.106
9....	27.9	7 feet, 11 inches downstream from canal (centerline).	28	380	.074

[1] See figure 7.

Visual observations of trial locations indicated that placing the vanes either farther upstream or downstream from the canal headworks would reduce the efficiency of the vanes. The multiple correlation analysis of the three tests indicated that placing the vanes 5 feet 7 inches upstream from the canal centerline was the most satisfactory arrangement.

Bottom Vane Length

Tests 7, 10, and 11 were used to determine a satisfactory vane length. The standard test discharge was set and the following conditions were constant for the three tests: vane spacing 26 feet on centers, vane elevation 4,665.0 feet, angle of vane with direction of flow 45°, and the tip of the downstream vane 5 feet 7 inches upstream from the centerline of the canal headworks. Table 5 summarizes results of these tests.

Plots and analysis of data indicated the 50-foot vane length to be most satisfactory. The results showed a considerable improvement over the average concentration ratio of 2.38 with no vanes in place.

TABLE 5.—*Results of tests to determine a satisfactory bottom vane length*

Test No.	Hours test was conducted	Vane length	Concentration of sediment entering canal headworks	Concentration of sediment in the river	Concentration ratio, $\frac{Cc}{C_{rus}}$
	Number	*Feet*	*P.p.m.*	*P.p.m.*	
7....	26.8	50	22	467	0.047
10....	30.6	40	16	303	.053
11....	29.8	30	34	333	.102

Vane Top Elevation

Tests 7, 12, 13, 14, and 15 were utilized to establish a satisfactory vane top elevation for the test discharge. For this series of tests, vane length was 50 feet, vane spacing was 26 feet on centers, angle of vane with direction of flow was 45°, and the tip of the downstream vane was placed 5 feet 7 inches upstream from the centerline of the canal headworks. Four vanes were used in all tests. Table 6 summarizes results of these tests.

TABLE 6.—*Results of tests to determine a satisfactory top elevation, bottom vane*

Test No.	Hours test was conducted	Vane top elevation	Concentration of sediment entering canal headworks	Concentration of sediment in the river	Concentration ratio, $\frac{Cc}{C_{rus}}$
	Number	*Feet*	*P.p.m.*	*P.p.m.*	
7....	26.8	4,665.0	22	467	0.047
12....	29.0	4,663.9	90	362	.249
13....	29.5	4,664.5	26	175	.149
14....	49.0	4,666.2	4	319	.013
15....	23.9	4,666.8	41	456	.090

The vanes installed for test 12 appeared to be too low and allowed considerable sediment to pass over them. An average decrease in discharge of approximately 5 percent occurred in the canal during test 12. No significant decrease in discharge occurred during the other tests of this series.

Analyses of the data indicated the most satisfactory surface elevation to be between 4,665.9 and 4,666.2 feet. The elevation selected as most satisfactory, after these and additional test data were analyzed, was 4,666.1 feet. However, in some succeeding tests the value 4,665.9 was used.

Number of Vanes

Test 16 was used to establish whether fewer than four vanes would produce sufficiently strong secondary currents to reduce sediment intake into the canal.

Three vanes were tested in a manner similar to that for four vanes. The 50-foot-long vanes were placed with the tip of the downstream vane 5 feet 7 inches upstream from the canal headworks. Vane top elevation was 4,665.9; the vanes were 26 feet on centers and placed at an angle of 45° with the direction of flow. The standard discharges were set, and tests of 18.5 and 7.0 hours were conducted. The canal discharge remained constant during the tests, and the resulting average concentration ratio was 0.067.

From visual observations and comparison of tests 16 and 14, it was concluded that four vanes produced a more satisfactory concentration ratio than three vanes.

Effect of Vane Cross Section

Tests 17 and 18 were utilized to determine the effect of vane cross section on the concentration ratio. In previous tests, the vanes in the model were constructed of sheet metal, equivalent to a thickness of approximately 1 inch in a prototype structure 20 times as large as the model. A prototype vane would be somewhat thicker, particularly a concrete vane, and tests were required to evaluate the effect of vane thickness. Vanes 8 inches thick were therefore investigated. Figure 7 shows cross sections of the vanes used in these tests.

Four 50-foot-long vanes were used in these tests, spaced 26 feet on centers, placed at an angle of 45° with the direction of flow, and with the tip of the downstream vane 5 feet 7 inches upstream from the canal headworks centerline. Vane top elevation for test 17 was 4,665.9 feet and for test 18 was 4,666.1 feet. The test discharges of 8,760 cubic feet per second in the river and 174 cubic feet per second in the canal were used for the tests.

In test 17, a set of four bottom vanes with sharp-edged lips extending 2 feet $6\frac{13}{32}$ inches upstream, shown in figure 7, were tested during runs of 18.7, 6.2, 17.0, and 6.9 hours. The canal discharge remained constant, but no significant improvement was shown over vanes made with a rectangular cross section. The average concentration ratio was 0.110.

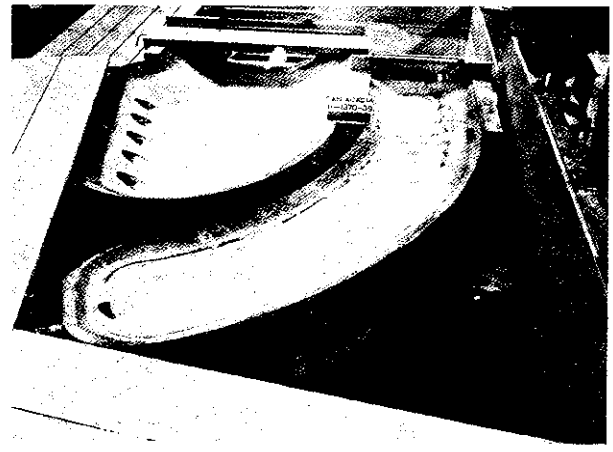
Rectangular vanes 8 inches thick (fig. 7) were installed for test 18, and runs of 16.9 and 6.7 hours were conducted.

The canal discharge remained constant during both runs, and the resulting average concentration ratio was 0.094. The rectangular vane cross section used for this test appeared to be more satisfactory than the cross section used in test 17 and would certainly be easier to construct in a field installation.

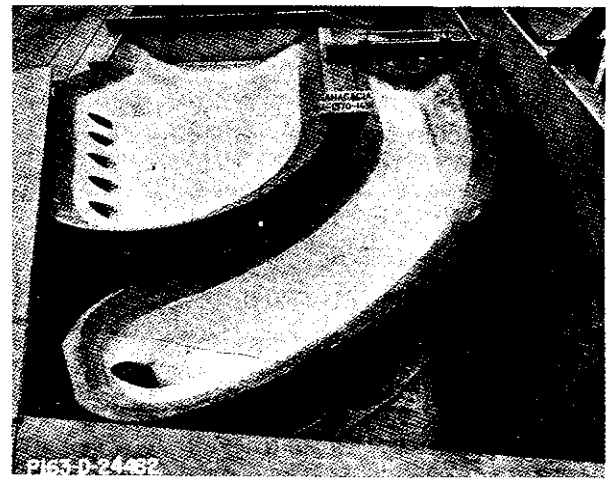
Photographs of the sediment deposits that resulted in the canal when no control was used and when the vanes of test 18 were used are shown in figure 8.

Summary of Bottom Vane Tests

From these tests it was concluded that bottom vanes are effective in reducing heavy sediment intake into a canal supplied by water diverted from a large river. The most efficient bottom vanes developed were a group of four 50-foot-long vanes installed upstream from the intake at an angle of 45° to the direction of flow. The vanes were spaced 26 feet on centers, the downstream tip of the downstream vane was located 5 feet 7 inches upstream from the canal headworks centerline and vane top elevation



(a)
Without Bottom Guide Vanes



(b)
With Bottom Guide Vanes

FIGURE 8. — Sediment deposits in canal (*a*) without bottom guide vanes and (*b*) with bottom vanes in hydraulic model study.

was at 4,666.1. This arrangement reduced the concentration ratio for the test discharge from 2.38, the average for the five control tests with no vanes in place, to less than 0.1. This ratio reduction means, in effect, that the vanes allowed only 1/23 of the usual amount of heavy sediment to enter the canal. Tests conducted during the San Acacia model study [4] indicated that the vanes were also of benefit when the ratio of river to canal discharge was varied.

If the standard test discharge used in these tests is only considered, the dimensions of the variables, location, spacing, angle, length, depth, number, and cross section were not critical with respect to performance. Minor changes in the dimensions tested could be made, therefore, without changing performance significantly. Surface vanes were next investigated to determine whether they were more or less efficient than bottom vanes.

Tests with Surface Vanes

Surface Vane Spacing

Tests 21, 22, 23, and 25 were used to determine the effect of spacing of surface vanes (tests 20 and 24 were control tests). Vane arrangements for the tests are shown in figure 9 and the test data are given in table 7.

TABLE 7.—*Results of tests to determine satisfactory vane spacing for surface vanes*

Test No.	Hours test was conducted	Vane spacing	Concentration of sediment entering canal headworks	Concentration of sediment in the river	Concentration ratio, $\frac{C_c}{C_{rus}}$
	Number	*Ft.-in.*	*P.p.m.*	*P.p.m.*	
21....	29.9	16–8	59	944	0.062
22....	4.8	12–0	113	953	.119
23....	5.8	26–0	87	917	.095
25....	3.0	20–0	118	1,030	.114

Four vanes 50 feet long and 2 feet 8½ inches deep were placed with their bottoms at elevation 4,667.68; vane tops were at the normal water surface. The vanes were placed at an angle of 140° (measured from the same reference as the bottom vanes), with the downstream end of the downstream vane at the canal headworks centerline. In this test and all tests conducted with surface vanes, the vane thickness was 8 inches. The vane spacing was varied in each test as indicated in column 3 of table 7.

The test ratios all showed considerable improvement over the 2.38 average concentration ratio of the control tests and indicated that the concentration ratio was not greatly affected by the spacing of the vanes. However, a multiple correlation of the data showed that a spacing of approximately 18 feet 4 inches would be the most efficient for these vanes. From this series of tests it was concluded that surface vanes could be used effectively to reduce the sediment entering a canal headworks.

[4] See footnote 2.

Surface Vane Angle

In the bottom vane tests, the vanes were installed to guide bottom water away from the right bank into the river. When surface vanes are used to produce the same flow conditions they must be pointed toward the bank (looking downstream). In this position the vanes create secondary currents that move the bottom water away from the riverbank into the river. This action helps to exclude sediment from the headworks (fig. 2). Tests 23, 26, 27, and 28 were used to test various vane angles.

For all four tests the vanes were 50 feet long, vane spacing was 26 feet on centers, the downstream tip of the downstream vane was at the canal centerline, and the vane was 2 feet 8½ inches deep, with bottom elevation at 4,667.68 feet (fig. 9). Tests were made using the standard test discharges, and the multiple correlation method was used for analyzing results. Results of the tests are shown in table 8.

TABLE 8.—*Results of tests to determine satisfactory surface vane angle*

Test No.	Hours test was conducted	Vane angle	Concentration of sediment entering canal headworks	Concentration of sediment in the river	Concentration ratio, $\frac{C_c}{C_{rus}}$
	Number		*P.p.m.*	*P.p.m.*	
23....	5.8	140°	87	917	0.095
26....	4.8	145°	97	875	.111
27....	3.0	135°	76	936	.081
28....	5.0	130°	48	1,007	.047

Although the concentration ratio was not very sensitive to the angle at which the vanes were placed, a correlation analysis indicated the 130° angle to be most satisfactory.

Surface Vane Location

Tests 27, 29, and 30 were used to test surface vane placement (location). For all tests, the vane length was 50 feet, vane spacing was 26 feet on centers, angle of the vane was 135°, vane depth was 2 feet 8½ inches, and vane bottom elevation was 4,667.68 feet (fig. 9). The standard test discharges of 8,760 cubic feet per second in the river and 174 cubic feet per second in the canal were used, and the multiple correlation method was used in analyzing data. Test results are summarized in table 9.

These three tests indicated that the vanes were remarkably insensitive to exact location. Concentration ratios were very similar for all three tests. However, visual observations indicated

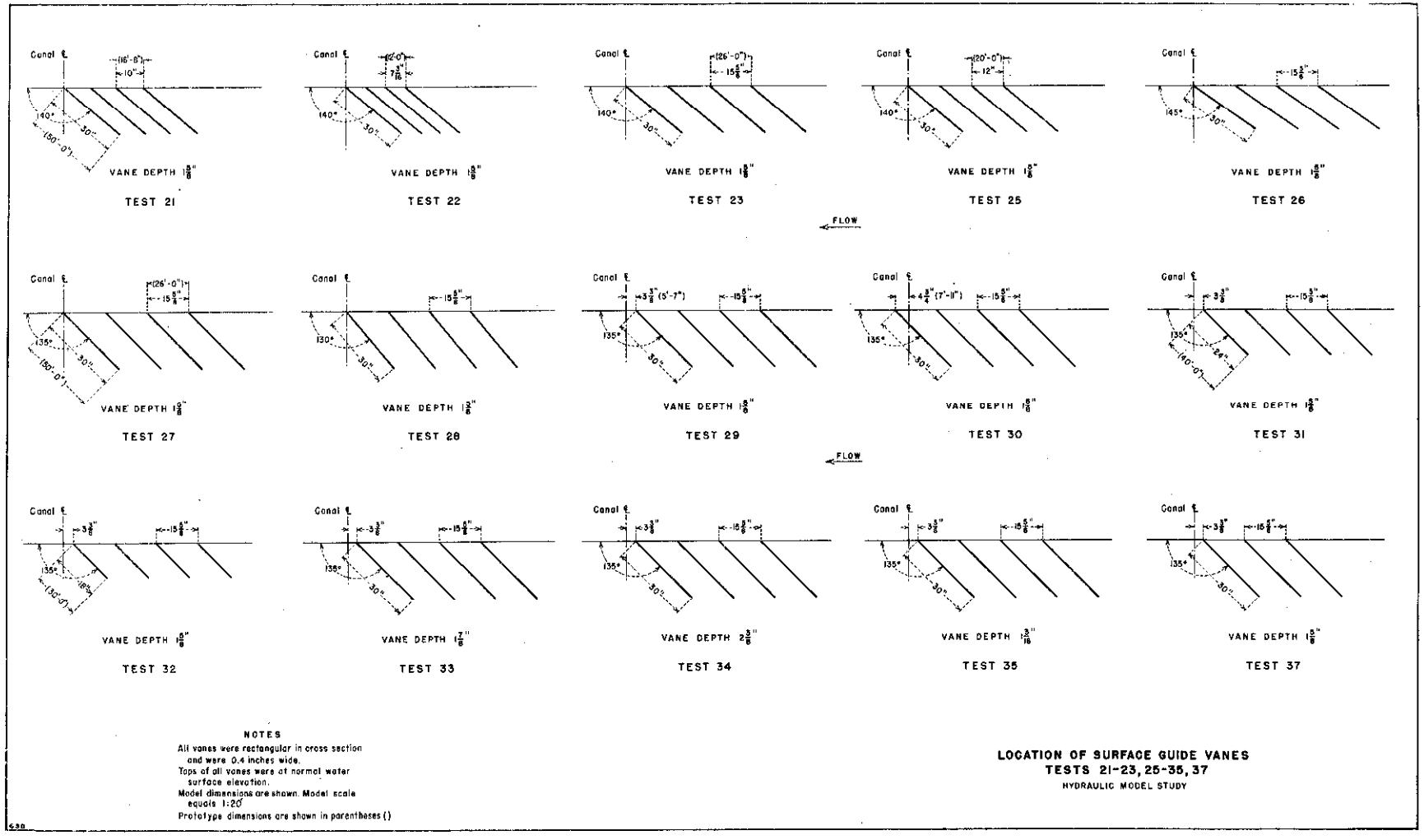


FIGURE 9. — Location of surface guide vanes for tests 21 to 23, 25 to 35, and 37.

that moving the surface vanes farther upstream or downstream would have reduced their efficiency.

TABLE 9.—*Results of tests to determine satisfactory placement of surface vanes with respect to canal headworks*

Test No.	Hours test was conducted	Tip location of downstream vane	Concentration of sediment entering canal headworks	Concentration of sediment in the river	Concentration ratio, $\frac{Ce}{Crus}$
	Number		*P.p.m.*	*P.p.m.*	
27...	3	At canal (centerline).	76	936	0.081
29...	5	5 feet, 7 inches upstream from canal (centerline).	83	1,052	.079
30...	5	7 feet, 11 inches downstream from canal (centerline).	62	824	.075

Surface Vane Lengths

Tests 29, 31, and 32 were used to show the effect of varying the length of surface vanes on their efficiency in controlling sediment movements. The standard test discharges were used and the following conditions were maintained constant: Vane spacing was 26 feet on centers, angle of vane was 135°, the tip of the downstream vane was placed 5 feet 7 inches upstream from the canal centerline, the vanes were 2 feet 8½ inches deep, and vane bottom elevation was 4667.68 feet (fig. 9). Table 10 summarizes results of these tests.

TABLE 10.—*Results of tests to determine a satisfactory surface vane length*

Test No.	Hours test was conducted	Vane length	Concentration of sediment entering canal headworks	Concentration of sediment in the river	Concentration ratio, $\frac{Ce}{Crus}$
	Number	*Feet*	*P.p.m.*	*P.p.m.*	
29....	5	50	83	1,052	0.079
31....	5	40	57	861	.066
32....	5	30	62	863	.072

The analysis indicated that a surface vane longer than 40 feet did not improve the concentration ratio, and that a vane less than 40 feet long was slightly less efficient.

Surface Vane Height

Tests 29, 33, 34, and 35 were utilized to establish a satisfactory height of surface vanes. For this series of tests the vane length was 50 feet, vane spacing was 26 feet on centers, and the angle of the vanes was 135°. Four vanes were used, and the tip of the downstream vane was placed 5 feet 7 inches upstream from the centerline of the canal headworks. Table 11 summarizes results of these tests.

TABLE 11.—*Results of tests to determine a satisfactory surface vane height*

Test No.	Hours test was conducted	Vane height	Concentration of sediment entering canal headworks	Concentration of sediment in the river	Concentration ratio, $\frac{Ce}{Crus}$
	Number	*Ft.-in.*	*P.p.m.*	*P.p.m.*	
29....	5	2— 8½	83	1,052	0.079
33....	5	3— 1½	61	903	.067
34....	5	3—11½	84	1,018	.084
35....	5	1—11¾	59	988	.060

In all cases, the vanes were effective, but analysis indicated the most effective depth to be 1 foot 11¾ inches.

Number of Surface Vanes

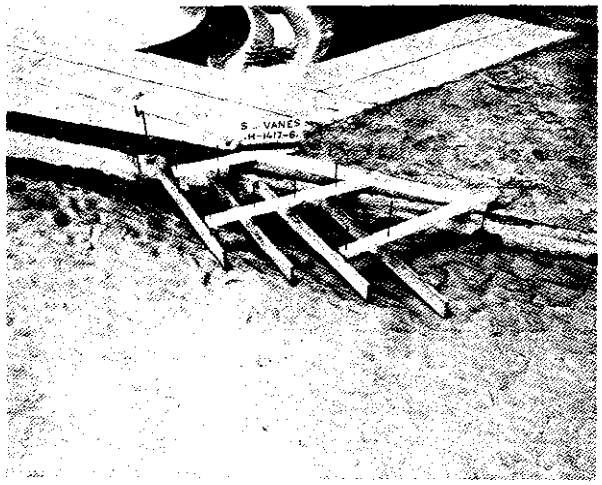
Test 37 was conducted to establish the effect of fewer than four vanes (test 36 was a control test). Three 50-foot-long vanes were placed with the tip of the downstream vane 5 feet 7 inches upstream from the canal headworks. Vane depth was 2 feet 8½ inches, spacing was 26 feet on centers, and vanes were placed at an angle of 135°. The standard discharges were tested and the resulting concentration ratio was 0.074, almost identical with the ratio 0.079 of test 29 in which four vanes were used.

Summary of Surface Vane Tests

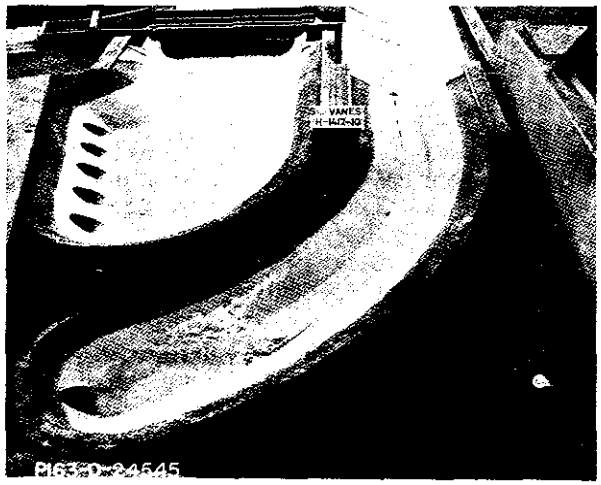
From the tests with surface vanes, it was concluded that surface vanes are effective in reducing heavy sediment intake into a canal supplied with water diverted from a large river and are about as efficient as bottom vanes. The dimensions of the vane variables, location, spacing, height, angle, length, and number were not critical with respect to the performance of the set of vanes. Installed in the same relative position as the bottom vanes but angled so as to divert top water into the canal headworks (bottom vanes are angled so as to divert bottom water away from the headworks) the surface vanes produced approximately equivalent results to bottom vanes and reduced the concentration ratio for the test discharge from 2.38, the average for the control tests, to less than 0.1. All tests on the surface vanes were conducted at standard discharges of 8,760 cubic feet per second in the river and 174 cubic feet per second diverted to the canal.

Photographs showing the surface vanes in

place for test 21 and the resulting deposits in the canal following the tests are shown in figure 10.



(a)
Surface Vanes in Place for Test 21



(b)
Sediment Deposits in Canal After Test 21

FIGURE 10. — Surface vanes in place (*a*) and sediment deposits in canal (*b*) after test 21.

Comparison of Vanes and Discussion

Both surface and bottom guide vanes reduced the concentration ratio $\frac{C_c}{C_{rus}}$ (concentration of sediment entering the canal to concentration of sediment in the river upstream) from 2.38, the average of the control tests, to less than 0.1. In other words, the sediment entering the headworks was only 1/23 the amount that entered when no vanes were used. A comparison of the vane variables that produced the most satisfactory results for the bottom and surface vanes is shown in table 12. Although the results of tests with both types of vanes showed that the vanes were not overly sensitive to the variables tested (within the test range), the surface vanes appeared to be the least sensitive to location, length, and numbers of vanes. The angle at which the bottom vanes worked best was 45°. Also, the ratio, height of vane divided by the water depth above the slab, may be less for the surface vanes. The lesser height of the surface vanes appears to result in less total projected vane area in the flow prism. However, as some sediment was always present on the supporting slab when the bottom vanes were in place (fig. 11) the y/d ratio and projected area shown for the bottom vanes in table 12 is no doubt too large. Few generalizations can be made from these tests because only one arrangement of canal intake and only one discharge were tested. The tests demonstrated, however, that satisfactory vane arrangements can be developed by the use of the testing and analysis principles discussed.

The most effective set of bottom vanes tested consisted of four 50-foot-long vanes installed upstream from the canal headworks. The vanes were placed along the right bank of the river model at an angle of 45° to the direction of flow. Vane spacing was 26 feet on centers, vane top elevation was 4,666.1 feet, and the downstream tip of the downstream vane was located 5 feet 7 inches upstream from the canal headworks centerline. The bottom vanes tested are shown in figure 7.

The most effective set of surface vanes, indicated by the tests, included either three or four vanes 40 to 50 feet long placed near the canal headworks (table 12). The vanes were installed along the right bank of the river model at an angle of 130° (same reference datum as bottom vanes). Vane spacing was 18 feet 4 inches on centers and vane height was 1 foot 11¾ inches. The surface vanes tested are shown in figure 9.

Both bottom and surface vanes were found to be extremely valuable in helping to gain control of heavy sediments by creating localized secondary currents to reduce the sediment intake into a canal. Consideration should be given to their use where flow conditions are similar to those tested in this study. For example, where a relatively small discharge is being diverted from a relatively large flow and it is desired that the small discharge have a relatively light sediment load, either bottom or surface vanes may be used with the dimensions (or propor-

TABLE 12.—*Optimum dimensions of variables of bottom and surface vanes*

Vanes	Best spacing		Best angle tested	Best placement tested	Best vane length tested	Best y/d ratio tested[1]	Effect on concentration ratio from reducing number of vanes from 4 to 3	Total vane surface in flow above slab at best condition
	Feet	*Inches*			*Feet*			*Sq. ft.*
Bottom	26	0	45°	Tip of downstream vane 5 ft. 7 in. upstream from canal (centerline)	50	0.544	Increases	83.3
No. 7 Surface	18	4	130°	Tip of downstream vane from 7 feet, 11 inches downstream from canal (centerline) to 5 feet, 7 inches upstream from canal (centerline).	30–50	.257	No significant change.	33.3

[1] y = height of vane; d = depth of water above slab.

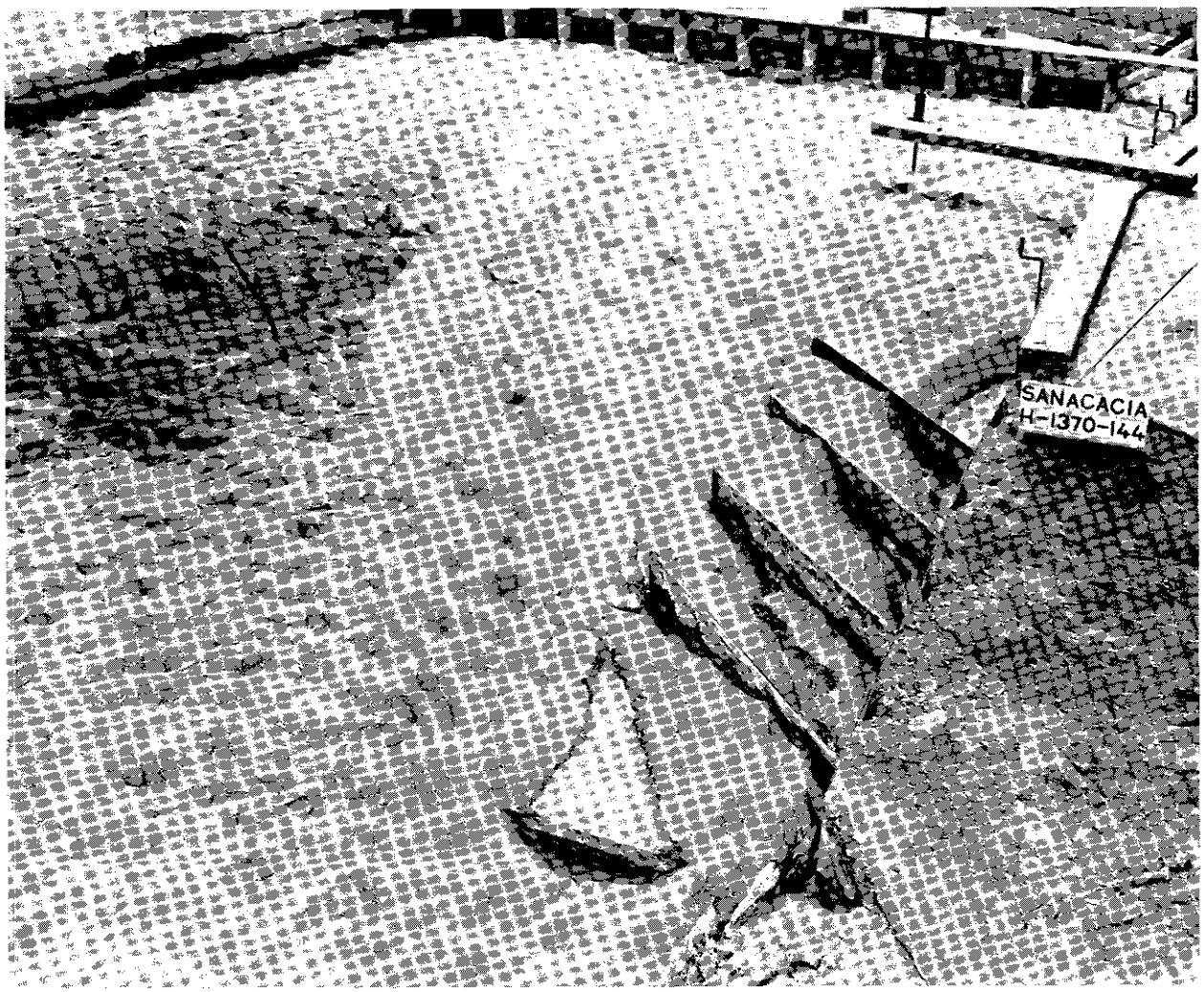


FIGURE 11. — Bottom vanes after test 18, showing sediment deposits partially covering vane area.

tional dimensions) given in this paper. Further investigation in a model should be made, however, if discharges are significantly different than those tested.

Further research should be conducted to determine the general performance of vanes in confined spaces and their possible use on increasing sediment loads in canal sluiceways. The effect of varying discharges should also be investigated.

SOME ASPECTS OF THE PROBLEM OF SCOUR AT BRIDGE CROSSINGS

[Paper No. 34]

By EMMETT M. LAURSEN, *professor and head, Department of Civil Engineering, The University of Arizona*

General Aspects of the Problem

As an agent of geological change, a stream is ever-changing. As an agent of sediment transportation, a stream is always tending toward equilibrium. However, as conditions change (such as the flow of the stream, the sediment supply to the stream, and the overall slope available), the equilibrium toward which a stream is tending will vary. In a natural stream the flow will contract and expand, and the streambed will be molded so that the capacity to transport sediment at every point is equal to the supply of sediment to that point. If a road crosses an alluvial valley, the embankment and the structures interfere with the natural pattern of flow. Generally there is a severe contraction of the flow and, consequently, severe remolding of the streambed — scour around the piers and abutments.

Ideally, the analysis of scour should start with a determination of the flow pattern and the boundary stresses, then proceed to a determination of the sediment transportation from point to point as a function of the bed material characteristics and the flow. The rate of scour then would be the difference between the transport capacity and the supply. As the scour progresses the boundary would change, changing the flow pattern and the stresses on the boundary. This would result in a change in capacity and supply from point to point and, thus, a change in the rate of scour. Eventually a stable condition, or limit, would be attained.

Unfortunately, the boundary geometries encountered are such that the flow pattern and the boundary stresses cannot be specified, and, moreover, their relation to the rate of sediment transport is not yet well established. However, in a few simple cases conditions can be described reasonably well with a minimum of conflict of opinion. The more complex geometries can then be approximated from these simple cases. Laboratory experiments tend to verify the analyses, but field investigations are needed for confidence in usage.

Clear-Water Scour and Scour by a Sediment-Transporting Flow

So many of the past investigations of scour seem to have confused the case of clear-water scour and scour with a sediment supply; it is worth while to make the distinction between these two cases. The simplest case of scour is that of a long contraction in which the transition between the two widths and such niceties as dune formation are ignored. Given two widths, a depth of flow (and slopes to compensate for the resistance to flow) the velocity of flow can be so low that there is no movement of the sediment anywhere, no matter how small the sediment or how great the contraction. There is clear water flowing in both reaches under these conditions. The flow pattern is that of the fixed boundary and, if the flow is subcritical, the depth of flow in the contraction is less than that in the approach reach. Therefore, the velocity and the boundary shear in the contraction are greater than in the approach reach. If the depth of flow is kept constant in the approach channel and the rate of flow is increased, the critical tractive force will be attained first in the contraction. Any further increase in the rate of flow will result in a boundary stress that will cause movement in the contraction and, therefore, scour. The greater the flow, the greater must be the scour, since it will continue until the depth is increased and the velocity decreased so that the boundary stresses again are critical for movement.

It should be readily apparent that in clear-water scour, the velocity and the sediment size, together with the geometry of the location, are important factors in determining the depth of scour.

If the discharge is increased to such an extent that in the approach flow the boundary shear is much greater than the critical tractive force of the sediment, there will be a continuous supply of sediment into the contraction. If the rate of movement is high enough, the critical

tractive force of the boundary sediments will have little influence. The bed in the contraction will be scoured until the boundary stresses are able only to transport the material supplied to the contraction. Now if the geometry were to remain the same for a higher discharge, the velocities everywhere should be increased proportionately and the shears also should increase everywhere proportionately. Therefore, the sediment transport rate at every point should increase in the same proportion; there should be no difference in the depth of scour no matter what the velocity or, using the same argument, what the sediment size. Few qualifications are necessary for this argument to be a good approximation. The Froude number should not be changed so greatly as to result in a change of the geometry of the free surface. The mode of movement and, therefore, the relation between the flow and sediment transport should be the same in the approach and in the contraction, and the rate of movement should be sufficiently high that the critical tractive force can be ignored. These qualifications are usually well satisfied for rivers in flood, and they have not given difficulty in laboratory experiments.

It should be clear that there is a difference between clear-water scour and scour by sediment-transporting flow and that conclusions reached from experiments on either one cannot be simply transposed to apply to the other.

The Iowa Investigations

Beginning in 1948, a series of investigations of scour around bridge piers and abutments were conducted at the Iowa Institute of Hydraulic Research, sponsored by the Iowa State Highway Commission and the U.S. Bureau of Public Roads. The first investigations were qualitative studies of typical pier and abutment geometries; followed by studies of the effect of sediment size and of velocity (*8*).

The experimental results of the second phase of the investigation indicated the lack of effect of these two factors on the depth of scour. It was noticed in the course of this phase of the investigation that the flow pattern was entirely different from that of two-dimensional flow. The sediment approaching the scour hole came straight down the flume and dropped into the scour hole. The sediment movement and the flow outside of the scour hole did not seem to pay any attention to the obstruction. The flow approaching the scour hole ducked down into the scour hole and came out as a spiral roller.

The third phase of the Iowa experiments involved the embankment on the flood plain that forces the overbank flow back into the main channel. Again the same characteristics were noted and served eventually as the basis for an approximate analysis of the scour phenomena. The results have been published elsewhere and will be merely summarized here (*5*, *6*). The analysis begins by extending the ideas of Straub for the long contraction (*10*). The definition sketch (fig. 1) shows an approach reach with

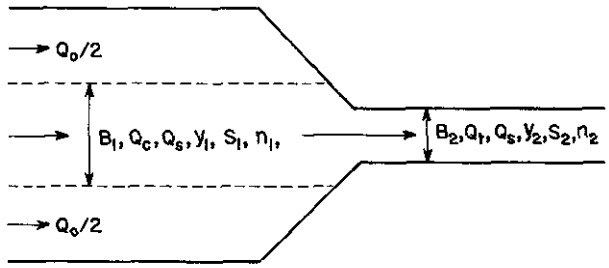


FIGURE 1. — Definition sketch of long contraction.

an overbank flow that does not transport any sediment and a contraction that constricts not only the flood-plain but the normal channel (see p. — for list of symbols). If the Manning formula to describe the flow in the two reaches and a total sediment load relation proposed by the writer (*4*) are used, the following expression for the depth ratio in the two reaches can be obtained:

$$\frac{y_2}{y_1} = \left(\frac{Q_t}{Q_c}\right)^{\frac{6}{7}} \left(\frac{B_1}{B_2}\right)^{\frac{6}{7}\,\frac{2+a}{3+a}} \left(\frac{n_2}{n_1}\right)^{\frac{6}{7}\,\frac{a}{3+a}} \qquad (1)$$

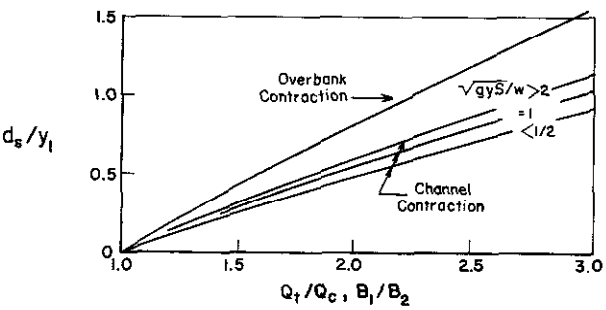


FIGURE 2. — Scour in a long contraction.

Figure 2 presents in graphical form the depth of scour as the difference between the two depths of flow as a function of a pure channel contraction or a pure overbank contraction, neglecting any effect of the n values, or kinetic terms. This solution for the long contraction can be extended to the overbank constricted by an embankment with the definitions shown in figure 3. Here the width of the approach channel is the length of the embankment plus 2.75

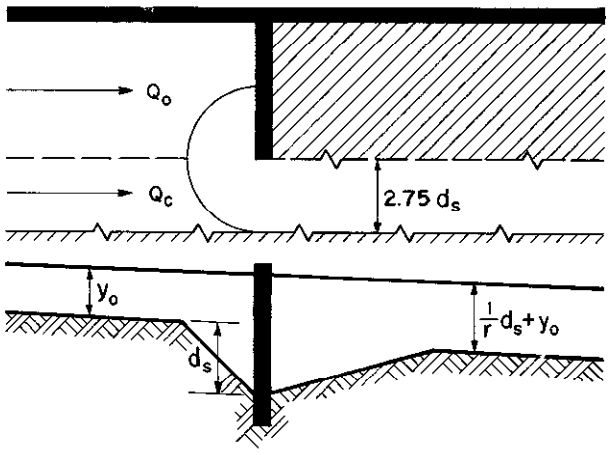


FIGURE 3. — Definition sketch of overbank constriction.

times the depth of scour at the abutment (i.e., the width of the scour hole), and the width of the contraction is 2.75 times the depth of scour at the abutment. The scour in the contraction is taken as a fraction $1/r$ of the scour at the abutment. Substitution of these definitions into the solution for the long contraction gives a relation for the depth of scour at the abutment of an overbank constriction.

$$\frac{Q_o}{Q_w}\frac{w}{6\bar{h}} = 2.75\frac{d_s}{y_o}\left[\left(\frac{1}{r}\frac{d_s}{y_o}+1\right)^{7/6}-1\right] \qquad (2)$$

Successive trials of w are made until w is approximately 2.75 d_s; Q_w is the flow in the width w. The results of the laboratory experiments are shown in figure 4 and a value of r of 4.1 gives rather good agreement between analysis and measurement. The points plot slightly above the line, but this is due to the fact that the flow on the overbank was actually greater than measured, because of crossflow in the approach. A similar relation can be obtained for the encroachment abutment.

$$\frac{l}{y_o} = 2.75\frac{d_s}{y_o}\left[\left(\frac{1}{r}\frac{d_s}{y_o}+1\right)^{1.70}-1\right] \qquad (3)$$

Good agreement with experiment is obtained with an r of 11.5, as shown in figure 5. The solution for the encroaching abutment can be adapted to the pier alined with the flow if the half width of the pier $b/2$ is substituted for the length of the encroaching embankment. In figure 6 the analytical relationship is compared to a design curve drawn conservatively on the

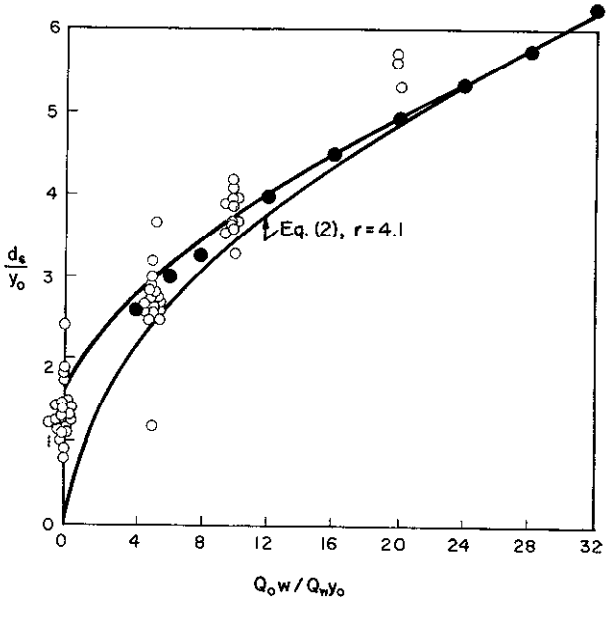


FIGURE 4. — Scour at an overbank constriction.

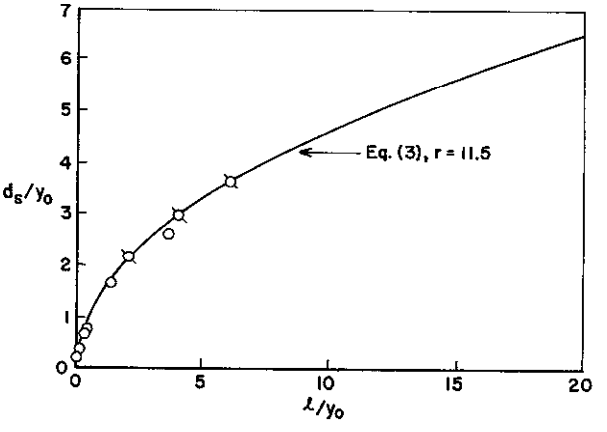


FIGURE 5. — Scour at an encroaching abutment.

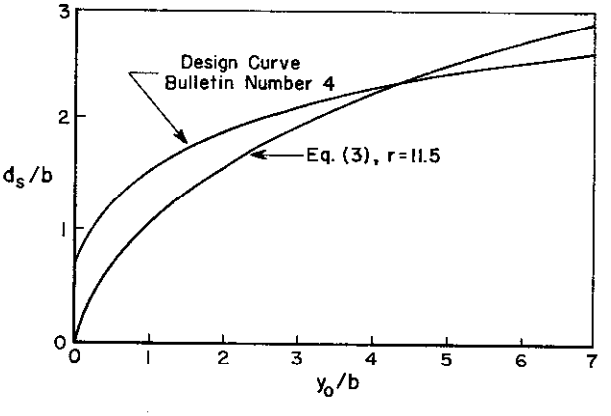


FIGURE 6. — Scour around basic bridge pier.

basis of the experiments with the piers. Geometry variations, such as a skewed crossing or pier, or variations in the mode of movement can be compensated for by correction factors.

A similar analysis has been performed for the case of clear-water scour (*7*). For the case of sediment-transporting flow, the conditions imposed were those of continuity of the water discharge and of the sediment transportation. For the case of clear-water flow, continuity for the discharge is again imposed, but, in place of the sediment-transport rate, the shear in the approach and in the contraction is specified. The shear is evaluated from the Manning formula and Strickler's definition of n. In the contraction the boundary shear is equated to the critical tractive force. The definition sketch for the long contraction is shown in figure 7. The

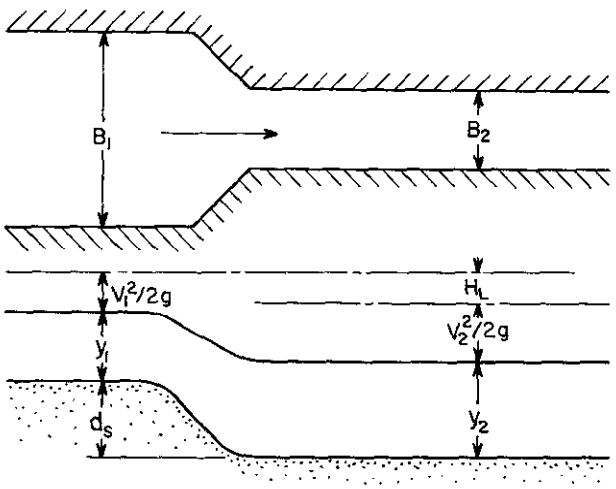


FIGURE 7. — Definition sketch of long contraction.

expression for depth of scour for this case of the long contraction can be obtained, considering the kinetic terms (the velocity heads and the loss in the transition) as:

$$\frac{d_s}{y_1}=\left(\frac{\tau_0'}{\tau_c}\right)^{3/7}\left(\frac{B_1}{B_2}\right)^{6/7}-1+1.87\,(1+K)\left(\frac{D}{y_1}\right)^{2/3}\left[\frac{(B_1/B_2)^{2/7}}{(\tau'_0/\tau_c)^{6/7}}-1\right]\left(\frac{\tau_0'}{\tau_c}\right) \qquad (4)$$

or neglecting the kinetic terms, a much simpler expression is obtained:

$$\frac{d_s}{y_1}=\left(\frac{\tau'_0{}'}{\tau_c}\right)^{3/7}\left(\frac{B_1}{B_2}\right)^{6/7}-1 \qquad (5)$$

This latter relation is shown in figure 8. It is readily apparent that not only the geometry but the velocity of flow and the sediment size are important in determining the depth of scour. The solution for the long contraction can again be applied to the pier and abutment by defining the depth of scour in the long contraction as a fraction of that at the pier or abutment and the width of the contraction as the

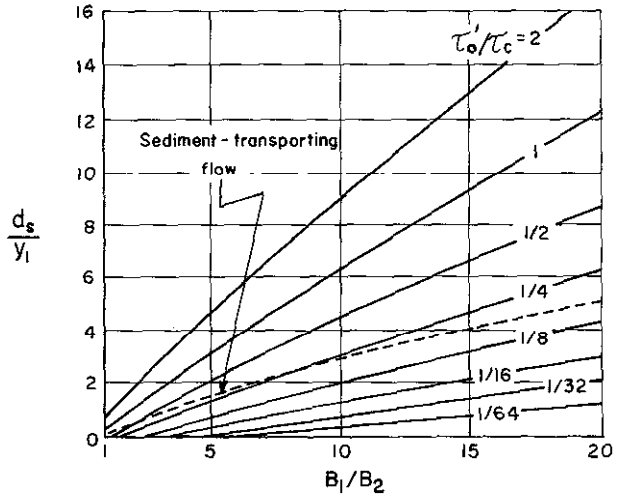


FIGURE 8. — Effect of shear ratio and width ratio on scour in a long contraction.

width of the scour hole. If the kinetic terms are neglected, the expression for the depth of scour for the abutment is:

$$\frac{l}{y_0}=2.75\,\frac{d_s}{y_0}\left[\frac{\left(\frac{1}{r}\frac{d_s}{y_0}+1\right)^{7/6}}{(\tau_0'/\tau_c)^{1/2}}-1\right] \qquad (6)$$

where $\tau_0'/\tau_c=V_0^2/120\,D^{2/3}y_0^{1/3}$.

In terms of pier geometry the half-width $b/2$ is substituted for the length of the embankment. The results are shown graphically in figures 9 and 10.

This approximate analysis of clear-water scour gives results that seem "right." Comparison with experiments from Poona (*3*), Chatou (*2*), and Fort Collins (*9*) are promising.

Implications of the Investigations

Several possibilities would seem to be implied, based on the results of the experiments and analyses. It would seem clear that the flow through a waterway opening is highly nonuniform, that dividing the total discharge by the nominal opening does not give a realistic "velocity" of flow which will result in a safe design. Unless the scour is so deep that scour holes overlap, the scour at the piers will not be affected by the overall contraction of the flow. The flow around the abutment is the flow interrupted by the embankment on the flood plain and

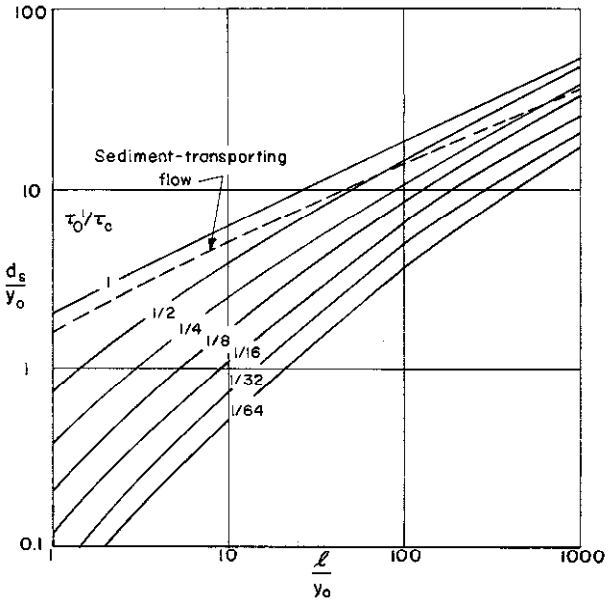


FIGURE 9. — Clear-water scour at an abutment.

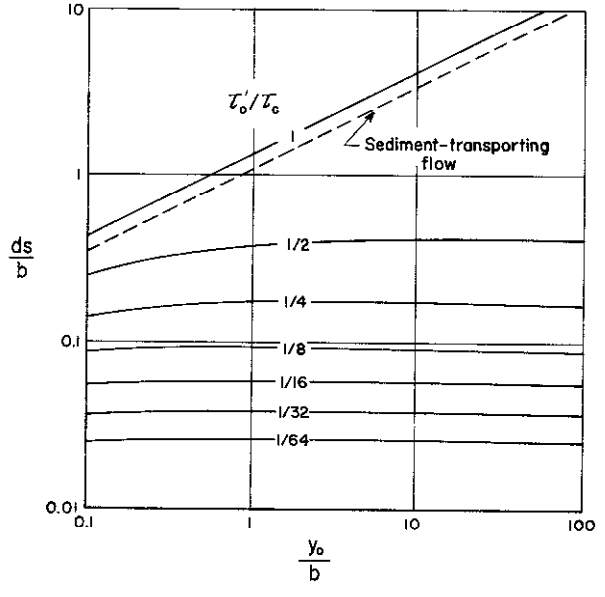


FIGURE 10. — Clear-water scour at a pier.

the length of the bridge itself is of small consequence. These conclusions are certainly to be taken as first approximations, because the geometry of a real crossing is much more complex than that of the schematic laboratory models. Moreover, abutments set back on the flood plain may be protected from scour because of the cohesive nature of the sediments. The typical layout with a cleared area upstream of the embankment, perhaps even a parallel ditch, tends to concentrate the overbank flow on the flood plain and, therefore, results in a different flow pattern from that of the laboratory. However, the concept that scour around bridge piers and abutments is a local phenomena should be considered in any criterion for waterway opening requirements.

Riprap and mattings have often been used in the past to protect against scour. The question has always been what size riprap should be used. The experiments and the analyses imply that riprap can be designed on the basis of clear-water scour and that the deeper the riprap is placed below the bed of the stream, the smaller it can be. An idea of the areal extent of the riprap can be gained from Schneible's experiments with a rigid mat extension of the pier placed below the bed of the stream.[1]

An ability to predict scour for a given geometry for different floods permits design from an "insurance" concept. There will be a certain depth of scour for a 50-year flood that has a certain chance of occurrence (one minus the chance of nonoccurrence) in the life of a bridge. The scour will be deeper for a 100-year flood, but there will be a smaller probability of occurrence. The possible loss weighted by the decrease in probability of occurrence can be balanced against the extra cost for rare floods, which can be either in the form of deeper foundations or riprap protection.

The scour in the long contraction is generally much less than a local scour around the piers or abutments. If the long contraction scour can be obtained by proper design of guide dikes upstream of the bridge, the cost of such dikes may be less than the cost of deeper foundations. Much shorter bridges are a possibility. A clue to the proper design of the guide dikes may be the constant-pressure transition curve (*1*).

Further Investigations Needed

The understanding of scour in the field is not yet at the point that predictions based on analyses and laboratory experiments can be used with complete confidence. What is needed is a variety of field measurements of scour to verify the relationships proposed or to point out deficiencies in those relationships. It is to be expected that the complexity of the field has not been adequately evaluated in the schematic laboratory and analytical models. The field measurements should be made in situations of simple geometry that can be correlated with the laboratory models and in situations of complex geometry that go beyond the schematic

[1] SCHNEIBLE, D. E. AN INVESTIGATION OF THE EFFECT OF BRIDGE-PIER SHAPE ON THE RELATIVE DEPTH OF SCOUR. Master's thesis. On file, library, State University of Iowa. 1951.

laboratory models. One might expect that as a result of these field measurements further experimental studies would be indicated. Judgment will always be an important factor. The floods to be expected are those of rare occurrence. The field conditions of the future can change drastically from those of today. However, it is better to have judgment related to these questions than directly to the question of how deep a bridge foundation should be placed.

List of Symbols

a ..... An exponent in an approximate sediment transport equation (*4, 5, 6*).

b ..... Width of a pier.

B ..... Width of flow section; subscript 1 refers to approach channel, subscript 2 refers to contracted channel.

d_s ..... Depth of scour measured from original stream bed.

D ..... Diameter of sediment particles.

K ..... Loss coefficient through constriction.

l ..... Effective length of constricting embankment.

n ..... Manning coefficient; subscript 1 refers to approach channel, subscript 2 refers to contracted channel.

Q ..... Rate of flow or discharge; subscript t refers to total flow, subscript c refers to flow in normal channel, subscript w to the flow in the channel over a width w measured out from the abutment.

r ..... Ratio between depth of scour at pier or abutment to depth of scour in fictitious long contraction.

w ..... Width of the fictitious long contraction equal to $2.75\ d_s$.

y ..... Depth of flow; subscript 1 refers to approach channel, subscript 2 refers to contracted channel, subscript 0 refers to depth of flow near pier or abutment.

τ_0 ..... Shear associated with sediment particle.

τ_c ..... Critical tractive force of sediment particle.

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DISCONTINUOUS DEPTH-DISCHARGE RELATIONS FOR SAND-CHANNEL STREAMS AND THEIR EFFECT ON SEDIMENT TRANSPORT

[Paper No. 35]

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Abstract

A discontinuity occurs in the depth-discharge relation of many sand-channel streams. For the higher part of the relation, after the discontinuity, Froude number and Chezy C resistance coefficient remain constant. The Chezy C approximately doubles, owing to the discontinuity. A concomitant increase on the order of eight times occurs in the transported sands. The effect of heavy loads of suspended fine material on these relations is discussed.

A discontinuity occurs in the depth-discharge relation of many sand-channel streams. For a given depth, there may be no unique relation with velocity. This is due to the fact that the form of bed roughness and, hence, resistance to flow are a function of fluid, sediment, and flow characteristics.

Laboratory studies (*10*) have been used to define the various regimes of flow in terms of configuration of the sand-bed channel. Those which apply to the field case are as follows:

Regime of flow	*Description of bed*
Lower regime:	
Ripple	Small, uniform sand waves, with little sediment movement.
Dunes	Much larger, more irregular sand waves, with a great deal of turbulence.
Upper regime:	
Plane bed	Dunes are smoothed out. Both the water surface and the bed are plane, with little turbulence.
Standing waves	Both the water surface and the bed are characterized by standing waves, often termed "sand waves."
Antidunes	The "sand waves" move upstream, until at some critical point they break, then re-form.

The discontinuity occurs between the dune regime and the plane bed regime. The roughness of the channel in the lower regime is about twice the roughness for upper regime. For upper regime, roughness may be considered essentially a grain roughness, whereas for lower regimes, there is both grain and form roughness (*8*).

The general approach to the determination of roughness is through the Chezy formula:

$$V = C\sqrt{RS}$$

which means that the velocity (V) at a cross-section varies as the square root of the hydraulic radius (R), if slope (S) and roughness (C) are constant. The usual adaptation to rigid channels of this relation is the well-known Manning's formula, where

$$C = \frac{1.5\,R^{1/6}}{n}$$

In this relation, velocity varies as the two-thirds power of hydraulic radius, if slope and Manning's n are constant. If slope can be considered as a constant with only statistical variation, a comparison of the variation of velocity with hydraulic radius can be used to study the variation of roughness.

Figure 1 shows such plots for four U.S.G.S. gaging stations on sand-channel streams. On each graph, the trend for the upper measurements is represented by a curve of relation. Each of these curves can be expressed as

$$V = kR^{1/2}$$

where k represents a constant in the relation. According to this relation, the Froude number remains constant, because

$$F = k/g^{1/2}$$

If the energy slope remains constant at a given stream site, each curve also represents a constant value of the Chezy roughness coefficient C for

$$C = k/S^{1/2}$$

contains only constant terms.

These stations are excellent examples of the non-uniqueness described by Brooks (*3*). This type of rating was described by Einstein and Chien in their discussion of Brooks, but they believed this type to occur rarely, if ever, in nature. It is now general knowledge that discontinuities are not rare in nature, but occur in many sand bed channels with "sufficient" slopes and depths (*7*). The stream gager in the field has long known that stage-discharge relations for many sand bed channels "went to Hell in a handbasket." Colby (*5*) in his Pigeon Roost report was the first to present a meaningful statement of the problem of nonuniqueness in the field.

If order can be applied to these data, as stated before, then the upper regime flows (plane bed and standing waves) should have resistance that is almost entirely due to grain resistance. Lower regime flows should have the same grain resistance plus an added form resistance. Streeter (*12*) relates the Darcy-Weisbach resistance coefficient, f $(= \frac{8gRS}{V^2} = 8(\frac{V_*}{V})^2)$, to the Reynolds number, Re $(= \frac{4VR}{\nu})$, through a relative roughness concept. Some values of resistance computed from this plot are given in table 1. These velocities computed on the basis of pipe curves in table 1 average about 15 percent too high.

TABLE 1.—*Values of resistance computed from Reynolds number-resistance plot forpipe flow for certain stations*

Station	$4VR/\eta$ x10^{-6} at 1 ft.	$4R/k_s$ at 1 ft.	$V/V_* = C/\sqrt{g}$ Computed	$V/V_* = C/\sqrt{g}$ Measured	Difference Percent
Bernalillo, N. Mex.	1.23	3,300	22.3	20.1	+11
Byhalia, Miss.	1.12	2,600	22.0	18.9	+21
Logan, N. Mex.	1.16	6,200	23.4	24.4	− 4
Kaycee, Wyo.	1.26	2,000	21.6	16.1	+34
	At 2 ft.	At 2 ft.			
Bernalillo, N. Mex.	3.45	6,600	24.1	20.1	+20
Byhalia, Miss.	3.14	5,200	23.6	18.9	+25
Logan, N. Mex.	3.69	12,400	25.6	27.4	− 7
Kaycee, Wyo.	4.0	4,000	23.5	18.1	+30

One might suppose lower regime flows could be determined from the Einstein and Barbarossa bar-resistance plot. In figure 1, *D*, the rating computed from the bar-resistance plot for Bernalillo is compared with the measured discontinuous rating. The computed rating goes through the discontinuity as a smooth curve. Perhaps this is because the upper end of the

curve is based on data from the Elkhorn River near Waterloo, Nebr., which has both upper and lower regime flows (*2*). The bar resistance plot did not anticipate and therefore does not

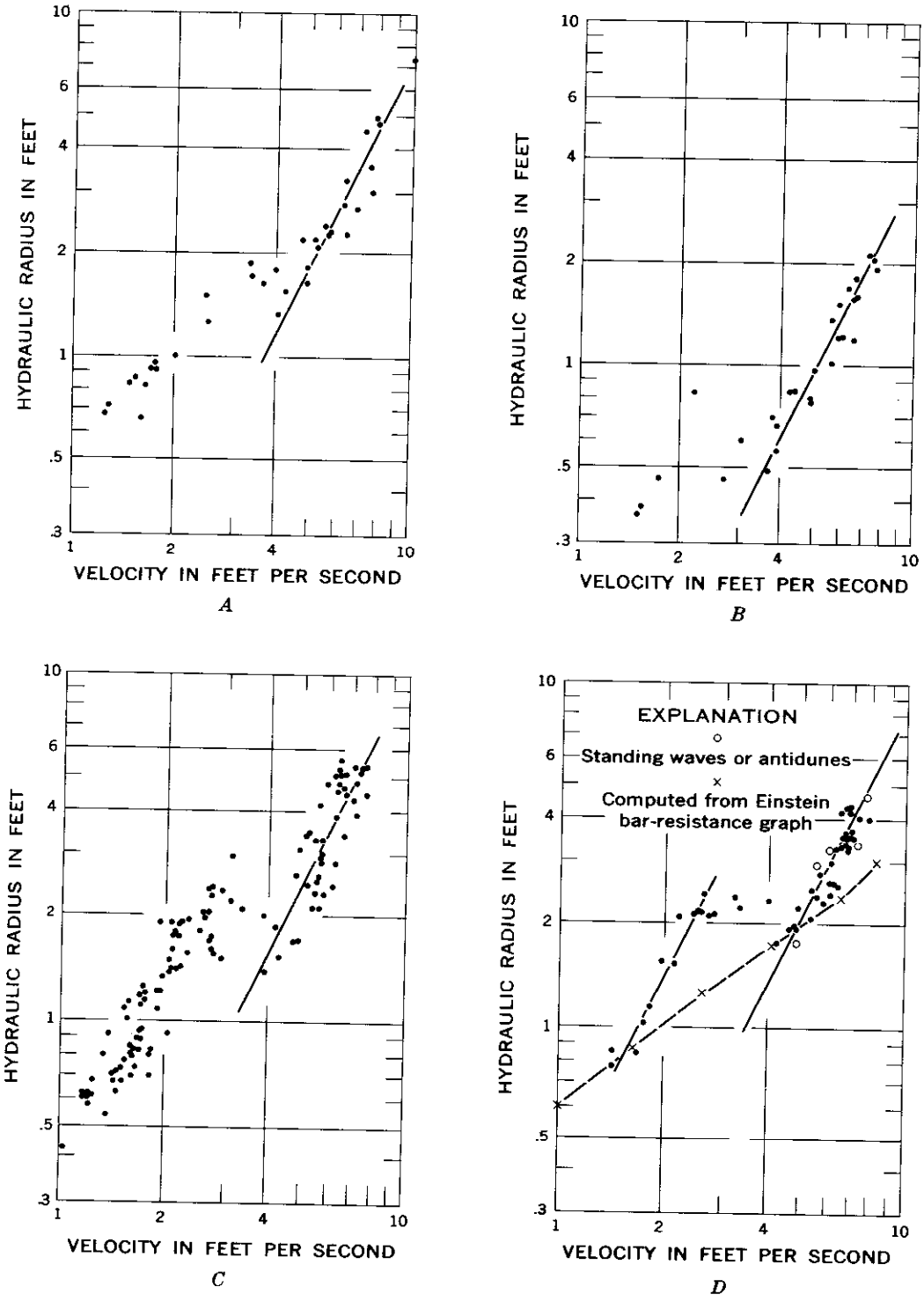


FIGURE 1.— Relation of velocity to hydraulic radius: *A*, Middle Loup River at St. Paul, Nebr.; *B*, Huerfano River near Undercliffe, Colo.; *C*, Pigeon Roost Creek near Byhalia, Miss.; and *D*, Rio Grande near Bernalillo, N. Mex.

handle the discontinuity. The bar resistance relation was developed for long reaches, and it is here applied to a section. A section plot for lower regime, perhaps including normal fixed boundary cobblestones and gravel reaches, might be sufficient. Resistance for upper regime flows seems possible through the pipe resistance type of approach.

The major problem still remains for the fluid mechanician to explain the mechanics and predict the occurrence of the discontinuity. The field man can fairly accurately tell regime of flow at the time of measurement by the appearance of the stream surface. The office engineer, somewhat less accurately, can use the appearance of a gage-height record for the same purpose. When these methods leave doubt, only intuition is left. As is shown by the plots, intuition can be wrong by an order of two. The intuitive method of guessing Manning's n is of no help, for the bed configuration after a flood event is of no help in determining regime at peak flow nor in determining if peak gage-height and peak discharge coincided.

For the gaging stations shown, the discontinuity is quite abrupt. There is a concomitant discontinuity in the shear-transport relation. Figure 2 shows a relation for the Rio Grande near Belen, N. Mex., Casa Colorado reach, of the Bagnold (*1*) shear against total sediment transport functions. The data are for measured suspended load only. By one interpretation (*6*), Bagnold predicts a rather abrupt eightfold increase of total load and explains it as development of a suspended load component. Bagnold's table 3 actually predicts a 3.54- to 3.9-fold increase for a 0.3 mm. sand, with a constant resistance coefficient. If $C/\sqrt{g}$ doubles in addition, the prediction becomes a 7.1- to 7.8-fold increase. Bagnold had flume data for total load only. I believe, but for a scale factor, Bagnold's flume results apply to the field case, and the two curves are for lower and upper regimes. The lower regime is with dunes on the bed. If, for dune flow, suspended load about equals bedload, Bagnold's curve seems to apply. If, in upper regime flows, bedload is negligible with respect to suspended load, as indicated by Simons (unpublished manuscript), then total load at Casa Colorada for both upper and lower regimes is about twice what Bagnold predicts. Interestingly, the slope of Bagnold's relation seems to be correct, because

$$\phi = K_1 \theta^{3/2} \doteq K_2 R^{3/2}$$

then if $$V = K_3 R^{1/2}$$

then $$\phi \doteq K_2 \left(\frac{V}{K_3}\right)^3 = K_4 V^3$$

which is Colby's (*4*) empirical formula relating total load to velocity. Colby's formula takes care of the discontinuity, for if velocity for a given depth doubles at the discontinuity, then load should increase by a factor of eight, as predicted by Bagnold and somewhat borne out by field data. However, velocity must be known in order to use the Colby formula, so that it begs the major question.

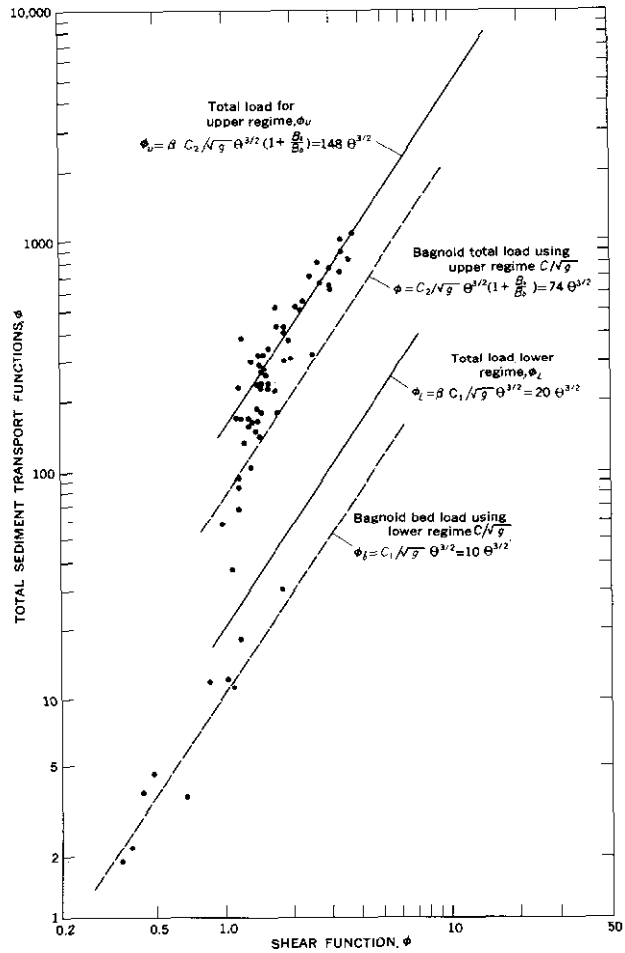


FIGURE 2. — Bagnold shear-transport functions: Rio Grande near Belen, N. Mex., Casa Colorado reach.

All discussion so far has concerned stations with little or no fine-material load. The addition of a charge of fine sediment has definite effects on the discontinuity in both the velocity and transport relations. Figure 3 shows the V-R relation for South Fork Powder River near Kaycee, Wyo. At this site large amounts of suspended fine material have been measured (*13*). Concentrations by weight of up to 120,000 p.p.m. of suspended material smaller than 0.062 mm. have been measured, and concentrations over 20,000 p.p.m. are common. A similar station is Canadian River near Logan, N. Mex. (fig. 4). On the other hand, Rio Grande at Bernalillo seldom has over 3,000 p.p.m. of sus-

pended fine material except during times of local tributary flow (*6*), Middle Loup at St. Paul seldom has over 3,000 p.p.m. of suspended fines at any time (*13*), and it is believed Huerfano River and Pigeon Roost Creek have similar amounts of suspended fines.

The effects of suspended fine sediment on bed form, and thus on the *V-R* relation, have been shown by flume studies (*11*). Apparently these effects combine to eliminate the abrupt discontinuity and produce a gradual change as evidenced by South Fork Powder and Canadian Rivers. The major effect of the fine material seems to be in changing the shape, and thus lowering the form resistance, of the dunes in the dune regime. The break occurs sooner, seems to be more gradual, and the data are decidedly less scattered, particularly for measurements in upper regime.

The slope of the average line for upper regime flows seems to be closer to two-thirds than to one-half for both the Kaycee and Logan stations. This would give them a constant Manning's n, and would indicate they more nearly approximate the pipe relation. Streeter (*12, p. 408*) states "a twofold change in (relative roughness) will cause a probable variation in f of less than 10 percent." This compares with a 12 percent change if Manning's n is constant.

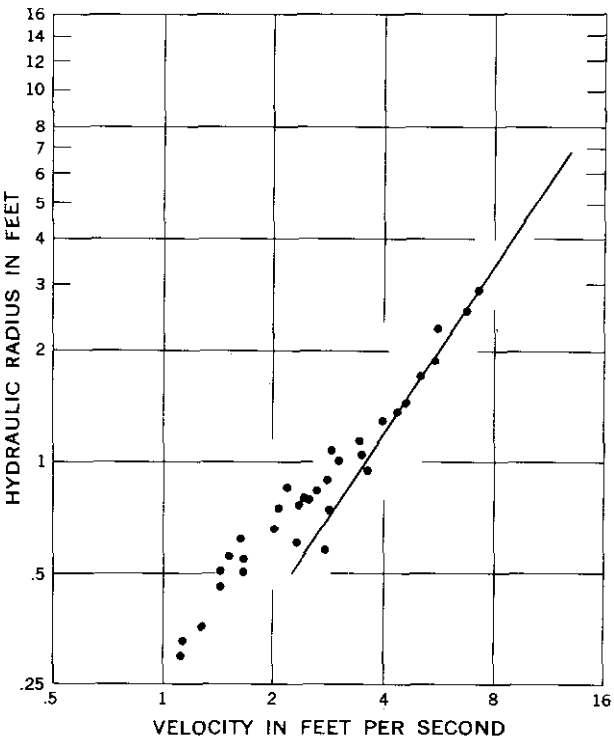


FIGURE 3.—Velocity-hydraulic radius (*V-R*) relation for South Fork Powder River near Kaycee, Wyo.

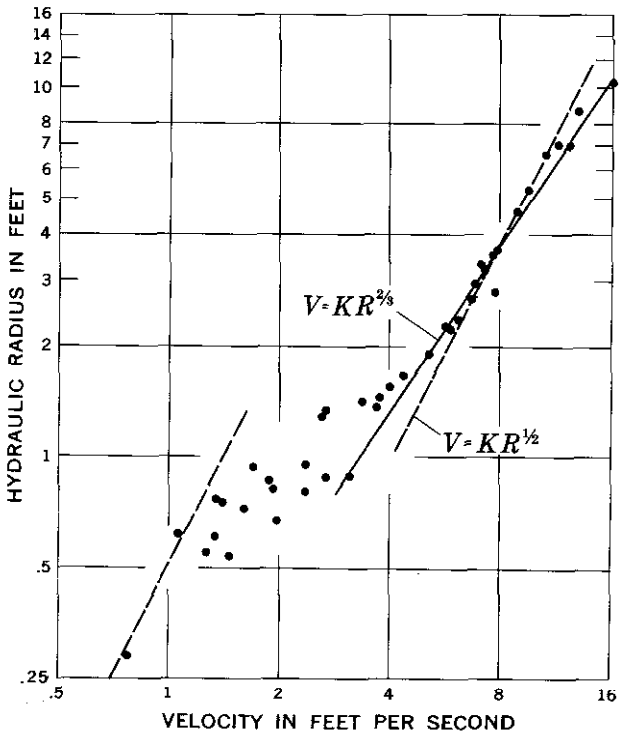


FIGURE 4.—Velocity-hydraulic radius (*V-R*) relation for Canadian River near Logan, N. Mex.

Therefore, probably neither Manning's n nor Chezy C is constant, but each is a fair approximation under certain conditions. Perhaps an alternative approach is through the use of the data to define a curve on a relative-roughness plot. This approach might result in not only a more rational discussion but also perhaps a better understanding of the definition of resistance to flow.

Conflicting evidence concerning the effect of suspended sediment on resistance to flow has been presented in the literature. Vanoni and Nomicos (*14*) conclude that sediment dampens the turbulence and thus reduces resistance to flow. Simons (personal communication) concludes that *fine* sediment increases antidune activity in upper regime flows and thus increases resistance. More or less valid arguments, depending upon the viewpoint of the individual, may be presented for and against either case. The available field data seem to indicate neither increase nor decrease of resistance for "in regime" flows. If resistance were a function of the amount of suspended sediment, then variation of suspended sediment should add one more unexplained variable to the *V-R* plot, thus increasing scatter about the mean line. Rather, the damping (or, perhaps better, suppression)

of turbulence seems to decrease scatter. Heuristically, one might argue the damping of turbulence tends to eliminate some instability phenomenon, which is characteristic of upper regime flows. Although resistance to flow seems to decrease with stage (perhaps because of a relative roughness factor), this does not support Vanoni's conclusion. Therefore, at this point one may believe almost anything concerning the effect of suspended sediment on resistance to flow and quote evidence in the literature to support his stand.

The addition of fine material load to the stream also has an effect on the shear-transport relation. Knapp (*9*) stated a condition for sediment that added rather than consumed energy. If this condition is met, "there is no limit to the concentration of such material that can be transported in suspension." This condition is similar to Bagnold's ratio of mean velocity of the sediment times slope to fall velocity (*1, p. 277*). Fine materials are those which meet Knapp's condition. There is not an unlimited supply of these in the bed. Therefore, they add energy to the stream, and this energy is expended in the transport of sand-size particles, which consume energy. The extra sand transport capacity with a fine material charge is borne out by both field and flume data.

In conclusion, in order to estimate velocity or transport in a sand channel stream, the regime of flow must be predicted. At this time it cannot. Further, it seems that in order to determine transport in the presence of a high concentration of fine material, one must measure it, a rather discouraging prospect.

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INFLUENCE OF SECONDARY FLOW ON LOCAL SCOUR AT OBSTRUCTIONS IN A CHANNEL

[Paper No. 36]

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Introduction

It has long been recognized that an obstruction, such as a pier or bridge abutment, placed in an alluvial channel will disturb the flow and result in removal of some of the bed material in a localized region near the obstruction so that a scour hole is formed. In some instances the scour hole has developed to unexpected depths, causing undermining of piers and failure of bridge structures. In other instances piers have been extended to depths much greater than necessary to prevent undermining, which

has resulted in excessive construction expense. Frequently, the obstructions cause a contraction in the flow channel. Variations of the geometry of the contracted region due to local scour can influence the flow capacity of the contracted region.

A number of excellent laboratory investigations have been made of these local scour problems (*3,4,5,6,7,10*). These have provided information on the shape of the scour pattern, on methods for predicting the depth of local scour, and in some instances on methods for controlling the depth of scour by selecting favorable shape parameters or by installing special scour control devices.

In these studies some limited observations were made of the details of the flow pattern at the obstruction. The spiral vortex forming in the scour hole at a circular pier is well known. In this paper an attempt will be made to develop a better understanding of some of the characteristics of the three-dimensional flow patterns occurring at such obstructions, which may lead to a better basis for devising methods to control the local scour.

Secondary Flows

For the purposes of this paper a secondary flow will be considered as one for which the velocities differ markedly from those indicated by the simple two-dimensional methods of analysis commonly applied to determine the flow around an object. Figure 1 shows the results of the simple analysis for flow past a cylindrical pier with a uniform approach velocity. If the axis of the pier is placed vertical, the velocities will all be in horizontal planes and their magnitudes given by potential flow theory except in the wake zone and in the boundary layer. Along the upstream element of the cylinder, which is the stagnation line, the velocity is zero and the pressure is a maximum equal to the dynamic pressure $\rho U_0^2/2$ for the approach velocity U_0.

Moving from the stagnation line around the cylinder, the tangential component of velocity increases to a maximum of about twice the free-stream velocity at the maximum breadth point. In this vicinity the flow separates from the cylinder, producing the wake zone of relatively low velocities. The normal pressure on the surface of the cylinder decreases from its maximum at the stagnation line to zero, or the static pressure, at $\theta=30°$ for an inviscid fluid and a somewhat greater angle for a real fluid. The normal pressure then continues to decrease and becomes negative, that is, less than the free-stream pressure by an increasing magnitude, until the separation point is reached. Within the wake zone the negative pressure may be considered as essentially constant and of the

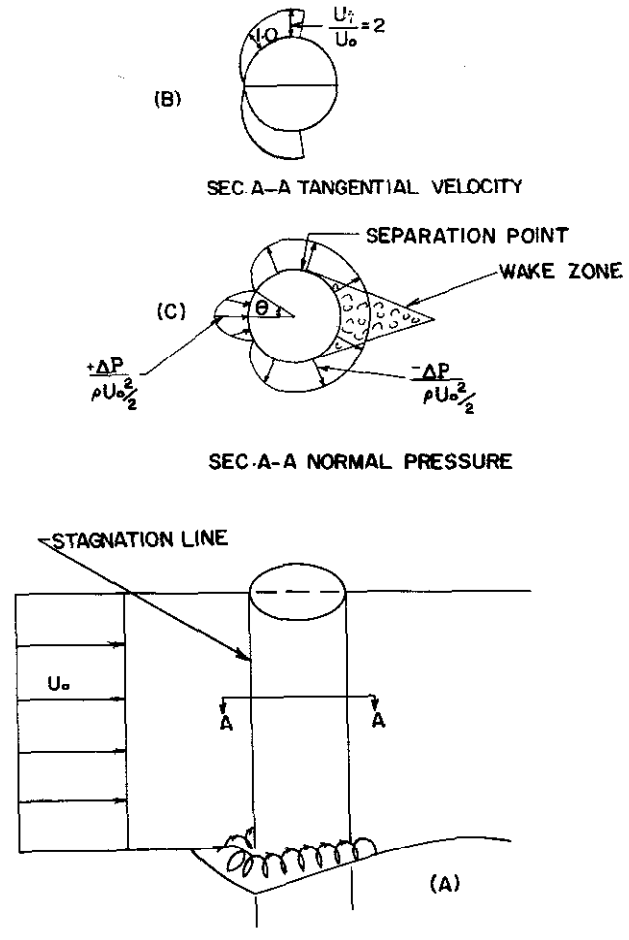


FIGURE 1.—Cylindrical pier with uniform approach velocity.

same magnitude as at the separation point. For these conditions the tangential velocity U_t will be given by

$$\frac{U_t}{U_0}=2\sin\theta;\left(0<\theta<\frac{\pi}{2}\right) \tag{1}$$

where U_0 is the free-stream velocity and θ the angle measured from the direction of the approaching flow. The variation in piezometric head around the surface of the cylinder is given by

$$\Delta h=h-h_0=\frac{U_0^2}{2g}(1-4\sin^2\theta);\left(0<\theta<\frac{\pi}{2}\right) \tag{2}$$

where the piezometric head h is the sum of the pressure head and elevation head, or

$$h=\frac{p}{\gamma}+y \tag{3}$$

Now consider the case where the approach velocity is not constant along the length of the cylinder but is larger at the upper end, as shown in figure 2,*A*. Since U_0 is now a function of y, it is clear that along the stagnation line the pie-

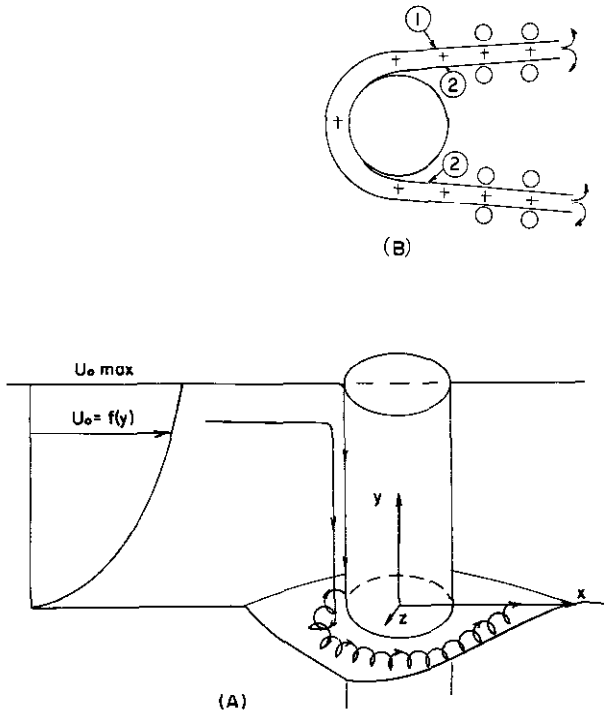


FIGURE 2. — Cylindrical pier with nonuniform approach velocity.

zometric head is also a function of y. If equation 2 is differentiated with respect to y and $\theta=0$ is substituted, it gives

$$\frac{\partial h}{\partial y}=\frac{\partial}{\partial y}\left(\frac{U_0^2}{2g}\right);\ (\theta=0) \qquad (4)$$

This gradient of piezometric head in the y direction implies an acceleration of the fluid along the stagnation line in the negative y direction. If, for simplicity, we neglect viscous effects and consider a stream line running downward along the stagnation line of the cylinder, Euler's equation states that

$$\frac{\partial h}{\partial y}=-\frac{\partial}{\partial y}\left(\frac{V_y^2}{2g}\right) \qquad (5)$$

which with equation 4 yields

$$\frac{\partial}{\partial y}\left(\frac{U_0^2}{2g}\right)=-\frac{\partial}{\partial y}\left(\frac{V_y^2}{2g}\right) \qquad (6)$$

This may be integrated along the stagnation line, which is a secondary flow stream line, to yield

$$\frac{V_y^2}{2g}=\frac{U_0^2{}_{\max}}{2g}-\frac{U_0^2}{2g} \qquad (7)$$

the constant of integration having been determined by setting $V_y=0$ where $U_o=U_{o\,\max}$, which occurs at the top of the cylinder. This gives the interesting result that at the bottom of the cylinder where U_o is zero, the downward velocity along the stagnation line V_y is equal to the surface velocity of the stream, $U_{o\,\max}$, or $V_y=U_{o\,\max}$ at $y=0$.

These concepts give the essential basis for a mechanism producing a strong downward vertical jet along the upstream nose of the pier and account for the fact that the maximum depth of the scour hole occurs at the most upstream point of the pier rather than at the maximum breadth points, as would be indicated by the velocity pattern for two-dimensional flow. As shown in figure 2,*A*, this downward secondary flow contributes to the formation and the sustaining of the spiral vortex in the scour hole around the upstream side of the pier.

What would happen if the cylinder rested on a rigid bed? In this case the downward secondary flow would not reach a maximum at the base of the cylinder but would encounter a second stagnation point at the bed, resulting in an increase in pressure and causing the central stream line to turn upstream. It would continue upstream until the resistance of the oncoming flow reduced the velocity to zero, where the streamline would split and move around the cylinder. This upstream flow at the base of cylinders in a boundary layer has been observed by dye traces (*12*) and also in studies of cylindrical total-pressure probes in flows with a transverse gradient of total pressure (*11*). The stream line pattern on the central plane is shown in figure 3 for the streamline with maximum approach velocity and for one streamline starting at a slightly lower level. It is clear that the actual pattern will be a function of the velocity distribution in the approaching flow. The pattern presented in figure 3 is not intended to

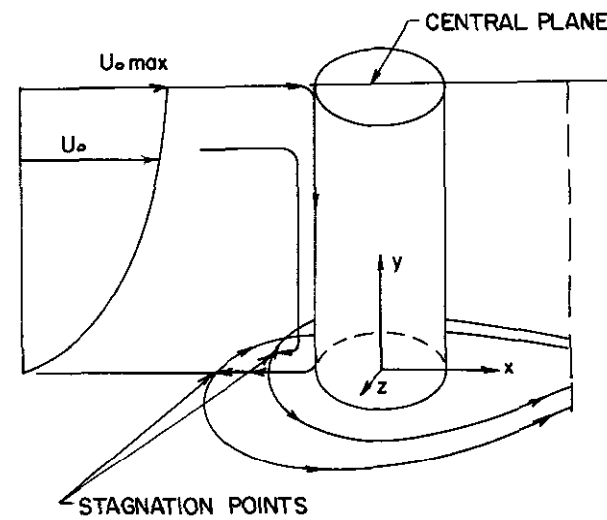


FIGURE 3. — Cylindrical pier with nonuniform approach velocity — rigid bed.

imply that water moving along the surface of the stream approaching a bridge pier will actually plunge downward along the stagnation line all the way to the bottom. Due to unsteadiness present in the flow, this ideal picture would not be realized and any particular fluid particle starting on the central plane would undoubtedly slide around the pier before being deflected a great distance in the downward direction. Baines (*1*) has described a similar flow on the upwind side of models of tall buildings tested with a velocity gradient in the approaching flow.

As seen in figure 1,*C*, the pressure for the entire forward part of the cylinder is increased, causing the piezometric head in this region to be larger than the static head. The magnitude of the piezometric head decreases in the downward direction because of the velocity gradient in the primary flow.

Behind the static pressure point, where the pressure is reduced and the piezometric head is less than in the free stream, the negative pressures are greater near the top, causing a gradient of piezometric head in the reverse direction that induces an upward flow in the wake zone. The entire pattern may be visualized as shown in figure 2,*B*, where line 1 represents a sheet of vorticity wrapped around the leading edge of the cylinder with downward flows, as indicated by crosses on the inside of the line, and upward flows as indicated by circles on the outside of the line. Two other sheets of vorticity, indicated by lines 2, attached near the static pressure elements of the cylinder would have opposite sense with downward flows on the outside of the lines and upward flows on the inside. Upward flow in the separation zone has been observed by Masch (*13*).

This problem of secondary flows has been analyzed in detail by Lighthill (*8,9*) for the simplified case of the flow of inviscid fluid with a constant velocity gradient past an infinitely long cylinder with no separation. A refinement of this analysis by Toomre (*14*) takes into account the viscous effect of the boundary on the secondary flow, which gives the secondary flow field in the plane of symmetry in front of the cylinder as shown in figure 4. In this figure, R is the Reynolds number, V_y the secondary flow velocity parallel to the cylinder, a the radius of the cylinder, and A the velocity gradient in the expression

$$U_o = U_1 + Ay$$

The velocity U_1 is a reference velocity at some arbitrary level.

As an example of the result of this analysis, consider a stream 10 feet deep with a surface velocity of 8 feet per second and a velocity near the bottom of 3 feet per second that corresponds

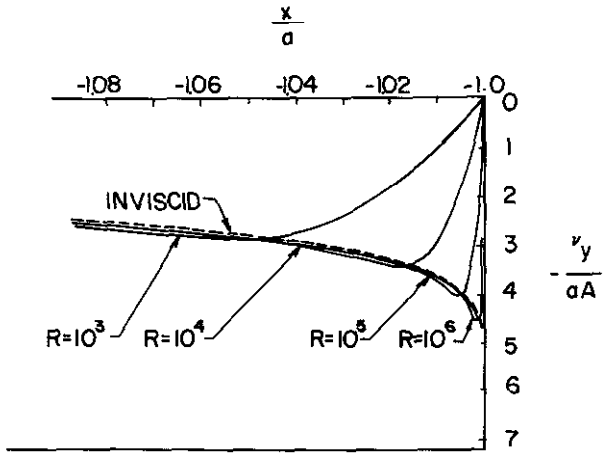


FIGURE 4.— Profiles of viscous secondary flow in the plane of symmetry in front of circular cylinder.

to A= 0.5. From figure 4, $V_y/aA = -4$, yielding $V_y = 6$ feet per second downward velocity near the upstream element of a 6-foot diameter cylinder. This simplified analysis for an infinitely long cylinder thus gives results with a reasonable order of magnitude.

Hawthorne (*2*) presents mathematical solutions for the secondary flow at sharp-nosed struts and airfoils in a velocity gradient flow. He also describes limited scour experiments in a sand bed, which support his analysis.

For a bridge abutment the secondary flow pattern may be visualized as sketched in figure 5. Line 1 again represents a sheet of vorticity

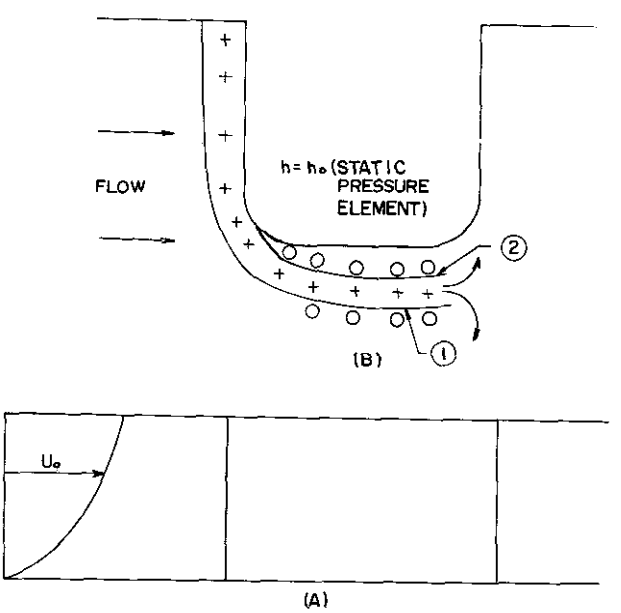


FIGURE 5. — Secondary flow at a bridge abutment.

wrapped around the forward part of the abutment with downward velocities, represented by crosses inside the line. This line of vorticity would curve upward and end at the water surface near the bank. Line 2 represents a sheet of vorticity of opposite sense attached to the abutment near the static pressure element with downward velocities, represented by crosses, outside the line and upward velocities represented by circles inside the line.

Relation Between Secondary Flows and Observed Scour Patterns

The implications of these secondary flows on local scour patterns at obstructions are examined, and the flows compared with the scour patterns observed in the laboratory. From earlier discussion, it is apparent that the strongest secondary flow is downward along the most forward element of the cylinder. This corresponds to the point of maximum observed scour depth in laboratory studies (fig. 6). Further, it will be noted that the two trailing sheets of vorticity as shown in figure 2,B, imply strong downward secondary flows in the regions corresponding to the scour tails extending downstream on each side of the scour hole. The vorticity represented by line 2 implies inward flows along the bottom, accounting for the piling up of sediment downstream and inside of the two scour tails as has been observed in the laboratory.

In the case of the abutment, the secondary flows indicated by figure 5,*B*, imply that the deepest scour would occur between the two sheets of vorticity and that material would be moved toward the abutment by the vortex system represented by line 2. This pattern has also been observed in the laboratory as shown in figure 7.

Proposals for Control of Scour

The concepts of secondary flows as the cause of scour suggest several possible methods for limiting the scour depth at piers. Four basically different ideas will be discussed briefly, each one of which raises a host of questions that must be answered before a suitable design could be chosen.

Horizontal Fins

Horizontal fins placed below the streambed and extending outward from the pier have been proposed and investigated as a means of limiting scour hole development. Based on the concepts of this paper, it appears that the lateral extent of these fins might be reduced by putting a lip at the outer edge to deflect the secondary flows back toward the surface and away from the bottom. It would also seem that a fin or a shoulder placed above the streambed would be helpful in breaking up the downward secondary flows and thus reducing the scour. However, with the fin placed above the bed, the secondary flows may develop again below the fin due to the high velocity gradient present near the streambed.

A Pier Located Upstream from the Main Pier

An auxiliary pier placed upstream has been proposed before, mainly with the idea of creating a scour hole upstream and far enough away to prevent damage to the main structure. Based on the concepts of this paper, it follows that such an upstream pier could be quite small, thus reducing its own scour hole. It could be rather short so the top would be submerged; the only requirement being that the pier be long enough to destroy the velocity gradient in the flow approaching the nose of the main pier. It is believed that the secondary flow concepts will be helpful in understanding and predicting the action of such a scour control device.

A Sharp Leading Edge

Because a blunt-nosed pier gives a larger region of retarded flow, it also results in a larger and stronger secondary downward flow at the nose, thus increasing the tendency to scour. A sharp nose with gradually tapering sides will produce a less intense secondary flow and reduce the maximum depth and extent of scour.

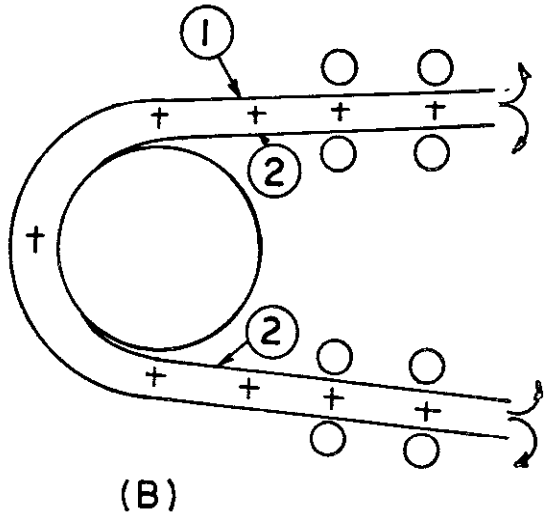
A Sloped Leading Edge

It is known that if a cylinder is placed in a flow with its axis swept back in the direction of flow, a flow will be induced along the leading edge in the direction of the sweep-back. This raises the intriguing possibility that a pier swept backward and upward at the proper angle could produce a secondary flow just equal and opposite to that induced by the velocity gradient in the approaching flow. It would appear that this swept-back leading edge would be needed only near the streambed where it could stop the downward flow along the leading edge before it attacked the bed of the stream. The effectiveness of this swept-back leading edge would probably depend upon its angle, its location relative to the streambed, the sharpness of its leading edge, and on how far it extended below the streambed.

Discussion

The concepts of secondary flow as developed here give some possibilities for the development of effective methods of scour protection for piers. Research is currently under way at the University of Texas to investigate in detail the nature of the secondary flows that develop

around obstructions in velocity gradient fields where the bed is rigid. Plans are being made for a systematic study of the effects of these secondary flows on movable beds and of devices to control the depth of scour by controlling the secondary flows.



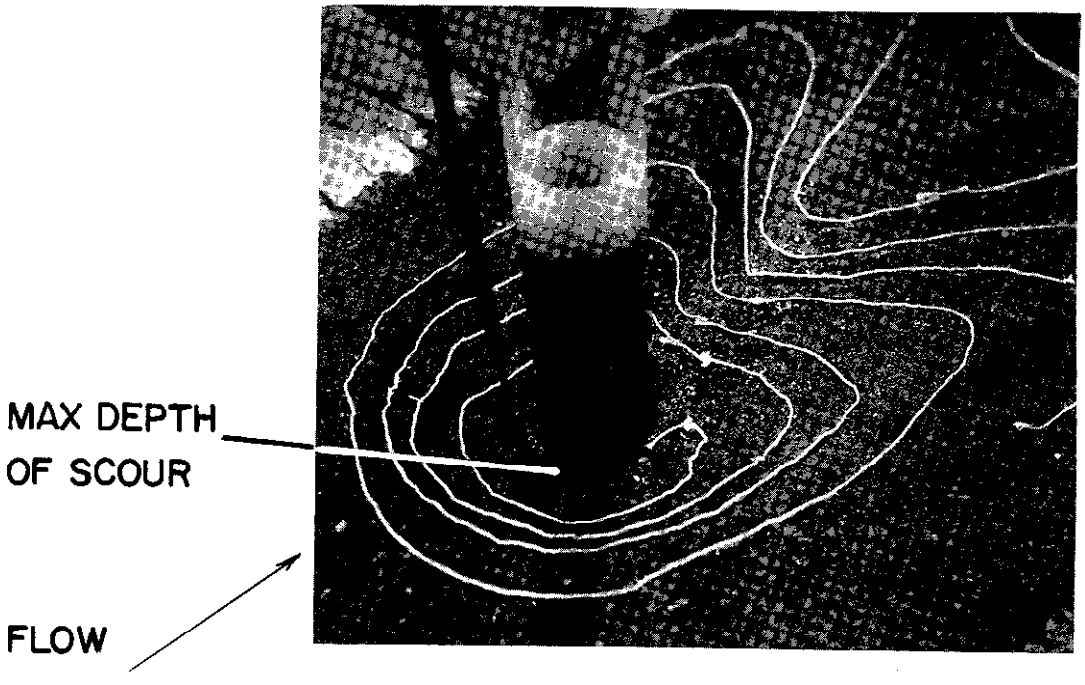


FIGURE 6. — Observed scour pattern at a round-nosed pier.

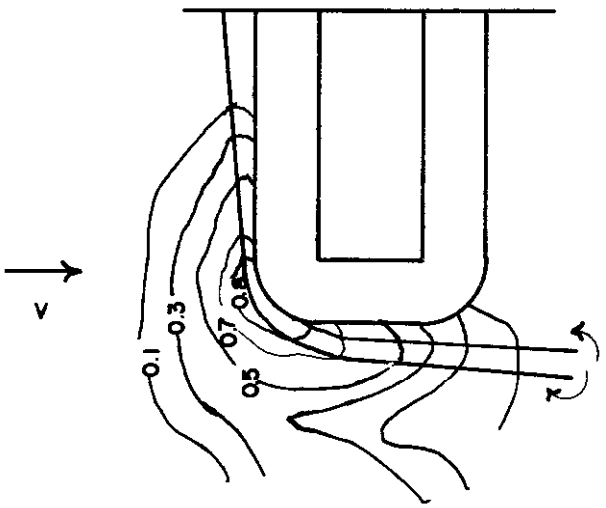


FIGURE 7. — Observed scour at a bridge abutment.

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LABORATORY STUDY OF AN ALLUVIAL STREAM AT CONSTANT DISCHARGE[1]

[Paper No. 37]

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Abstract

A laboratory study in a 60-foot flume was made to determine the hydraulic and sediment transport characters of an alluvial stream when the discharge is constant and the depth varies. These experiments supplement those of Vanoni and Brooks in which the discharge was varied with the depth constant.

For the given unit discharge (0.50 c.f.s./ft.) and bed sand (mean diameter 0.14 mm.) it was found that two different velocities and sediment transport rates are possible for certain values of slope or bed shear stress; however, this multiplicity occurs only in the range of slope or shear stress where major changes in bed configuration and roughness occur.

Stream morphology is discussed in light of these and other laboratory results, with emphasis on which variables are independent and which are dependent.

[1] The experimental research reported herein was performed in 1957-58 at the California Institute of Technology under sponsorship of the Agricultural Research Service, U.S. Department of Agriculture, as part of Contract USDA-12-14-100-995(41). This support is gratefully acknowledged.

[2] The writers would like to express their appreciation to Professor Vito A. Vanoni for the valuable criticism and many helpful suggestions that he offered during the investigation. The writers also wish to extend their gratitude to R. Hugh Taylor, Jr., for his assistance in performing the experiments and reducing the data.

Background and Objectives

In 1955 Brooks (*2*) reported a laboratory study of the mechanics of open-channel flow over a sand bed. The results of this investigation led him to question some widely held concepts about the relation between the sediment transport rate, depth and velocity of flow, and slope of alluvial channels. Brooks' conclusions, as restated by Vanoni and Brooks (*19*), are as follows:

1. In the laboratory flumes it was found that neither the velocity nor the sediment transport rate could be expressed as a single-valued function of the bed shear stress, or any combination of depth and slope, or bed hydraulic radius and slope. This contradicts assumptions which have commonly been held for some years to the effect that knowledge of the slope, channel geometry, and bed material of a stream were sufficient to predetermine its flow and sediment-transporting characteristics.
2. The cause of the non-uniqueness cited in conclusion no. 1, above, is the extreme variation in channel roughness caused by the variable nature of the bed configuration. In general, the flows at low velocities were accompanied by high, rugged dunes, while those at high velocity were associated with flat beds, with sand wave phenomena occurring at intermediate velocities.
3. On the basis of experiments to date, it appears that the depth and velocity may logically be used as independent variables, along with the grain size of the bed material. Knowing these quantities, it is found for the flumes that the slope, bed shear, friction factor, and sediment discharge are all uniquely determined. It was also found possible to consider the water and sediment discharges as independent variables, and from these to determine the depth, velocity, friction factor, and slope.
4. An examination of the results of these experiments yielded the following qualitative relationships:

a. For constant discharge, q, an increase in the sediment discharge, q_s, requires a decrease in the depth, d.

b. For a given slope and discharge, two depths of flow are possible. When q_s is low, the bed is covered with dunes, d is large, and the velocity, U, is small. When q_s is high, the bed is flat, d is small, and U is large.

c. If q is to be increased without changing q_s, then an increase in d is necessary, although this increase is relatively less than in q.

d. For a given q, the largest values of the bed friction factor, f_b, are associated with the lowest values of q_s.

e. When U is increased with d constant, f_b generally decreases, the slope S and the bed shear velocity U_{*b} may either increase or decrease, and the sediment discharge concentration, C, increases until the sand wave stage is reached.

f. When d is increased with U constant, f_b and C both decrease.

g. [3] The bed shear velocity, U_{*b}, changes less than any other quantity. It may be expected, therefore, that U_{*b} is not a good variable from which to determine the flow and sediment transport characteristics.

The discussion of Brooks' paper (*2*) was, for the most part, vigorously skeptical. In his closing discussion (*2*), Brooks presented the results of a new, larger group of experiments [also reported by Vanoni and Brooks (*19*)], some of which were designed to answer specific objections raised by the discussers. The new investigation covered a wider range of velocity at two different constant depths and used three sands with wider size distributions than the two fairly uniform sands he originally used. The results of the new experiments were in complete agreement with Brooks' original conclusions, which were pertinent. Those regarding the variation of sediment transport rate, slope, and friction factor with velocity for a constant discharge were neither proved nor disproved, because of the manner in which the runs were scheduled: the depth was fixed for each series of runs and the discharge was varied.

The objective of the experiments reported here was to verify Brooks' conclusions for the case of constant discharge. The main point to be resolved was whether two (or possibly three) different depths of flow are possible for a given discharge and slope. This question was the subject of considerable controversy in the discussion of Brooks' paper and was not settled by the later experiments (*2, p. 582* and *p. 594*). In the present inquiry, eight runs were conducted at a constant unit discharge of 0.50 c.f.s. per foot. For a more detailed description of these experiments the reader is referred to a report by Kennedy (*10*).

At the end of the paper stream morphology is discussed in the light of these experimental flume results.

Apparatus and Procedure

Only a summary description of the apparatus and procedure will be presented here. Complete details and thorough discussions of the apparatus, the techniques used in measuring the various quantities, and the procedure used in reducing the data have been given by Brooks (*2*), and by Vanoni and Brooks (*19*).

The experiments were performed in a 33.5-inch wide, 60-foot long flume. The flow circuit of the flume is closed, so the water and sediment are continuously recirculated and the mean depth of flow in the open channel is controlled by the amount of water in the system. The entire flume is mounted on a truss that is pivoted near one end and supported by a jack at the other end so the flume slope can be set at any desired value and easily adjusted during the course of a run. A point gage mounted on a carriage that moves along rails on the flume was used to measure the elevations of the water surface and sand bed relative to the flume. Water surface elevations were measured during the run at 4-foot intervals along the flume, and bed elevations were measured after the pump had been stopped and the bed leveled in 4-foot reaches with a special leveler. The discharge

[3] Conclusion 4g is based primarily on the experiments by Vanoni and Brooks (*19*).

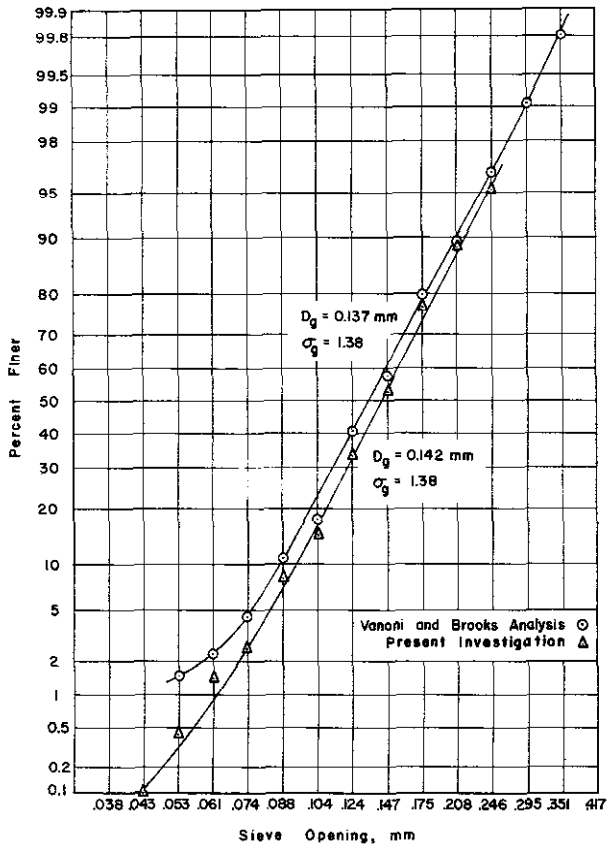


FIGURE 1. — Sieve analyses of the bed material.

was measured with the venturi meter located in the return pipe. To determine the total sediment transport rate, 12 or more samples of 1 liter each were siphoned through special samplers from the vertical section of pipe above the pump in the flow circuit. The individual 1-liter samples were then passed through filter paper, and the sediment retained was dried and weighed. A sieve analysis was performed on the composite of all samples for each run.

Sand Characteristics

The sieve analysis of the bed sand, plotted on logarithmic-probability paper in figure 1, shows that the geometric mean sieve diameter, D_g, was 0.142 mm., and the geometric standard deviation, σ_g, was 1.38. This is compared in figure 1 with the sieve analysis of the same sand when it was being used previously in the same flume by Vanoni and Brooks (*19*). There was a slight increase in D_g, no change in σ_g, and a significant decrease in the amount of fine material. For example, at the conclusion of the present experiments there was only about 76 percent as much material finer than the 170-mesh (0.088 mm.) sieve as during the experiments by Vanoni and Brooks, and for the smaller sieve sizes the losses were even greater. Most of the loss probably occurred when the flume was periodically drained to be refilled with clean water. This coarsening of the bed material will be cited later to explain a discrepancy in the transport rate between the results of the present experiments and those of Vanoni and Brooks.

Experimental Results

The principal measured and computed quantities from the laboratory experiments are summarized in tables 1 and 2. Quantities with subscript b refer to the bed section, as computed from the sidewall correction procedure, described by Vanoni and Brooks (*19*). Table 1 is self-explanatory except for the quantity, f_b/f_b', the friction factor ratio. In this quantity, f_b' is the friction factor predicted by the Moody pipe-friction diagram, using the geometric mean sieve diameter of the bed material, D_g, as the equivalent sand roughness, and $4r_b$, where r_b is the bed hydraulic radius, in place of the pipe diameter for the characteristics length. Thus, f_b' is the friction factor predicted by the Moody diagram for flow of clear water over an immobile flat sand bed. The friction factor ratio, f_b/f_b', is a measure of the effect of dunes or other bed features on the friction factor of the flow and serves as a partial quantitative description of the bed configuration; it should be approximately unity for flow over a flat bed. The use of this friction factor ratio in analyzing the roughness of alluvial channels was first suggested by Vanoni and Brooks (*19*). Detailed discussions of the friction factor ratio and its significance have been presented by Kennedy (*11*) and Taylor and Brooks (*18*). Values of f_b/f_b' obtained here are consistent with previous investigations (*18*).

The various types of bed configuration listed in column 18 of table 1 have been discussed by Brooks (*2*) and by Vanoni and Brooks (*19*). A dune-bed configuration, illustrated in figure 2, occurred for all runs of this investigation with velocity less than 1.5 f.p.s. The average wave lengths and heights of dunes of these runs are listed in table 2. The lengths were obtained by counting the number of dune crests along a measured line segment parallel to the flume. Dune heights were computed from measured elevations of troughs and crests of representative dunes.

Discussion

Variation of Friction Factor and Bed Shear Stress

The variations of slope S, bed shear velocity U_{*b}, and bed friction factor f_b, with changing mean velocity U, are shown in figure 3. These relations were obtained by first fitting a curve to the f_b data, and then computing values of S

TABLE 1.—*Summary of hydraulic and sediment transport data from laboratory experiments*
Sand characteristic: D_g=0.142 mm, σ_g=1.38

Run No.	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	Run No.
	d	U	q	F	r	S	U_*	f	r_b	U_{*b}	f_b	$\frac{f_b}{f_b'}$	T	C	q_s	Analysis of sediment load		Bed configuration	
	Depth	Mean Velocity	Unit discharge	Froude number	Hydraulic radius	Slope	Shear velocity	Friction factor	Bed hydr. radius	Bed shear velocity	Bed friction factor	Friction factor ratio	Water temp.	Discharge concentration	Sed. disch. rate	D_g	σ_g		
	Ft.	*Ft./sec.*	*C.f.s./ft.*		*Ft.*		*Ft./sec.*		*Ft.*	*Ft./sec.*			°C.	*G./l.*	*Lb./min. =ft.*	*Mm.*			
3–1	0.550	0.91	0.500	0.22	0.395	0.00056	0.084	0.068	0.503	0.095	0.086	4.85	19.5	0.014	0.026	0.103	1.60	Dunes	3–1
3–4	.441	1.14	.503	.30	.335	.00145	.125	.096	.414	.139	.119	6.60	18.4	.39	.73	.100	1.46	Dunes	3–4
3–2	.373	1.35	.504	.39	.294	.00206	.140	.086	.352	.153	.102	5.67	18.4	1.42	2.68	.096	1.43	Dunes	3–2
3–7	.345	1.45	.500	.44	.277	.00198	.133	.067	.324	.144	.079	4.34	25.1	1.13	2.12	.097	1.47	Dunes	3–7
3–5D [1]	.340	1.47	.500	.44	.273	.0016	.119	.052	.315	.126	.060	3.30	25.3	.98	1.84	.085	1.45	Dunes (S.W.) [2]	[3]3–5D [1]
3–5F [3]	.254	1.97	.500	.69	.215	.0025	.131	.036	.234	.137	.039	2.05	25.3	1.71	3.20	.102	1.49	Flat (S.W.) [2]	[3]3–5F [3]
3–6a	.235	2.13	.501	.78	.201	.00198	.113	.023	.209	.115	.023	1.22	25.7	1.57	2.95	.127	1.41	Flat	3–6a
3–6	.233	2.15	.501	.78	.200	.00207	.115	.023	.209	.118	.024	1.25	25.2	1.41	2.65	.128	1.38	Flat	3–6
3–6b	.228	2.21	.504	.82	.196	.00221	.118	.023	.205	.121	.024	1.24	25.4	1.74	3.29	.....	.....	Flat	3–6b

[1] Dune section of flow with a long sand wave.
[2] S.W. = Sand wave developed on bed.
[3] Flat section of flow with a long sand wave.

TABLE 2.—*Summary of data from dune measurements*

Run No.	Average length	Average height
	Ft.	*Ft.*
3–1	0.43	0.049
3–4	.46	.064
3–2	.48	.070
3–7	.42	.066
3–5D	.50	.061

and U_{*b} corresponding to points on the f_b curve; thus, the three curves are consistent with each other. Between the points for the run with a sand wave, the curves are shown as dashed lines because there is some question of whether or not uniform equilibrium flows over a sand of this size can exist at these intermediate velocities. For instance, for run 3–5, the amount of water was adjusted to yield a uniform depth of 0.30 ft. with mean velocity of 1.67 f.p.s. over the full length, but instead the sand wave phenomenon produced a depth of 0.34 ft. over the dune section and 0.25 ft. over the flat-bed section.

In figure 3 it is seen that the bed friction factor increases from 0.086 to 0.119 as the velocity increases from 0.91 to 1.14 f.p.s. With further increase in velocity, the bed friction factor decreases to 0.023 at a velocity of about 2.1 f.p.s. An initial increase of bed friction factor with velocity for this sand has also been observed by Vanoni and Brooks (*19*) for runs at a nearly constant depth of about 0.54 ft., but not for runs at a depth of about 0.24 ft. Thus, it appears that for some depths there is a velocity for which the roughness of the dune configuration is a maximum.

An important point in these experiments is the fact that the bed friction factor varied by a factor greater than 5 from 0.119 down to 0.023. The variation in f_b was due primarily to changes in the roughness of the bed as its configuration changed. As the velocity was increased above a certain value (about 1.2 f.p.s. for these experiments), the shape and arrangement of the dunes changed in such a way that the roughness of the bed decreased. When the bed became flat, the only roughness was that of the individual sand grains and was near the irreducible minimum. However, a small further decrease in the friction factor could have occurred at higher velocities with larger sediment concentrations. The suspended sediment apparently modifies the turbulence structure of the flow and thereby reduces the friction factor. This effect has been described by Vanoni and Brooks (*19*) and by Vanoni and Nomicos (*21*).

The large variation in the bed friction factor has an important effect on the relation between U and S, and between U and U_{*b}. As shown in figure 3, for a constant discharge the slope in-

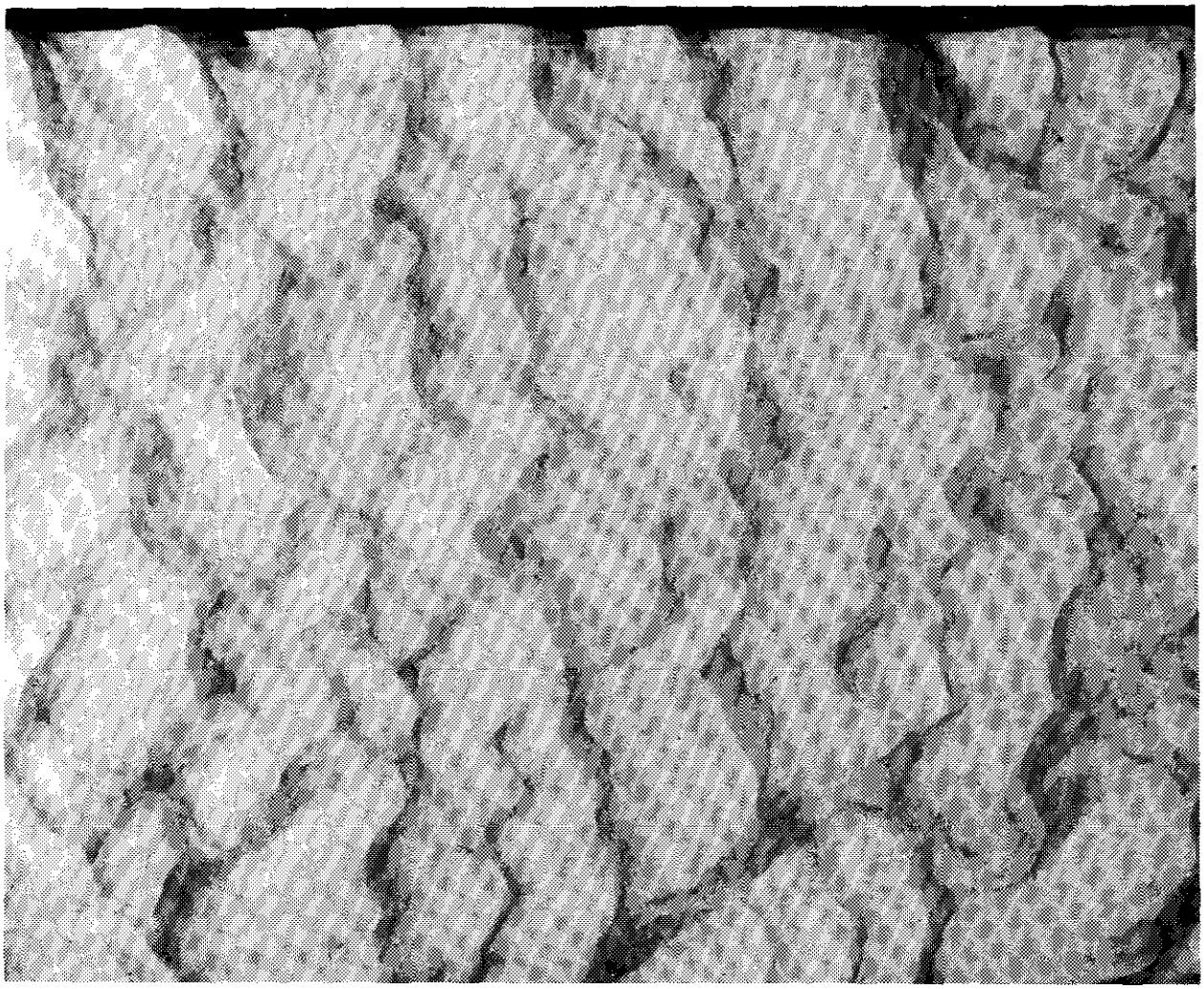


FIGURE 2. — Dune configuration of run 3-2. Flow direction left to right. Full 33.5-inch width of the flume is shown. Bed friction factor, f_b=0.102.

creases with velocity throughout most of the dune regime, then decreases at the upper end of the dune regime. In the flat bed regime, the slope again increases with velocity. Consequently, an identical value of slope, S=0.00198, was observed for two different runs: Run 3–7 (U=1.45 f.p.s., q_s=2.12 lb./min.-ft., dune bed) and run 3–6a (U=2.13 f.p.s., q_s=2.95 lb./min.-ft., flat bed). These two runs confirm, for this sand and this unit discharge, Brooks' conclusion No. 4b that for a given discharge and slope, two different velocities are possible. This type of nonuniqueness is illustrated schematically in figure 4.

Brooks' conclusion No. 4b regarding the multiplicity of velocity as a function of slope for constant discharge is valid only for a certain range of velocity and a corresponding range of slope. Since this multiplicity is a result of major changes in the bed configuration, two or more values of velocity for a given slope can occur only in the range of U and associated range of S from slightly less than that at which the transition from dunes to sand wave occurs to slightly greater than the velocity at which the bed configuration changes from sand wave to flat. For the constant discharge experiments reported here, this range of slope is S=0.0018 to 0.0021. The multiplicity of U as a function of U_{*b} for a constant q noted in the present experiments and shown in figure 3 is also limited to the range of U and corresponding range of U_{*b} where major changes in the bed configuration occur.

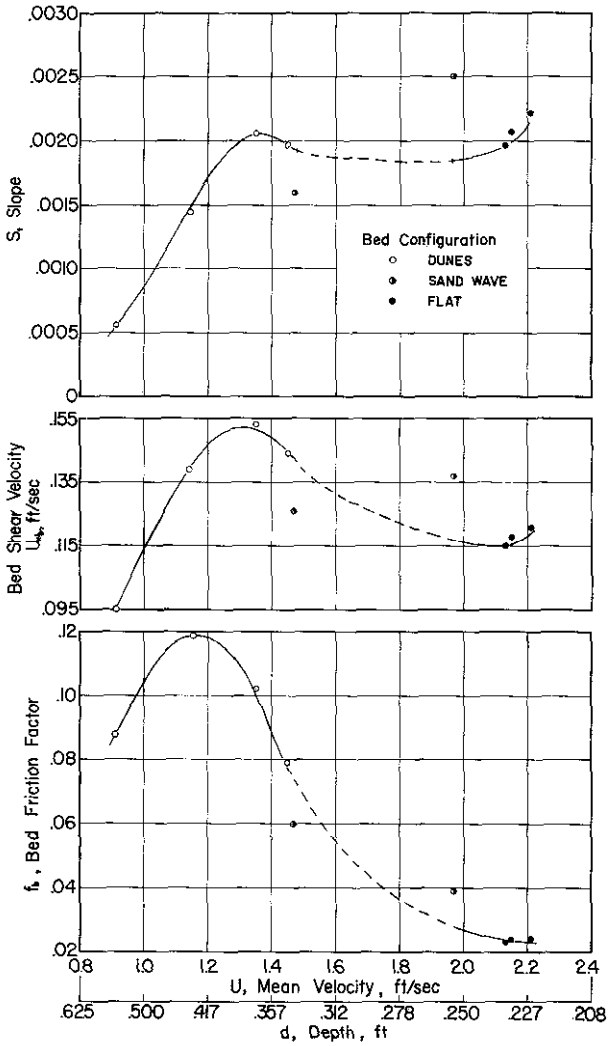


FIGURE 3. — Variation of slope, bed shear velocity, and bed friction factor with mean velocity and depth for constant-discharge experiments ($q = 0.50$ c.f.s. per foot).

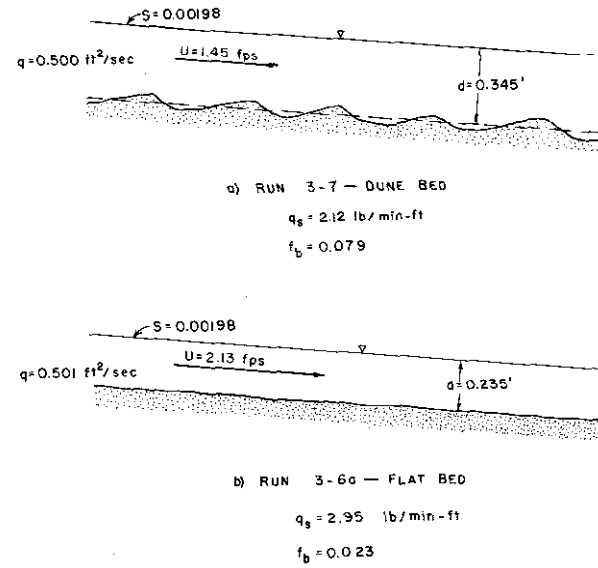


FIGURE 4. — Schematic diagram of two different equilibrium flows with same slope and discharge.

Thus, for the laboratory stream, the velocity for a given discharge cannot, in general, be expressed as a single-valued function of slope or bed shear velocity. However, each of the quantities S, U_{*b}, and f_b can be expressed as a single-valued function of the velocity, or in the case of these constant discharge runs, as a function of the depth. This is consistent with Brooks' conclusion No. 3. However, for velocities between 1.45 and 2.1 f.p.s., the nature of the variations of S, U_{*b}, and f_b is obscured by the sand waves that formed.

The bed shear velocity, U_{*b}, varied from a minimum of 0.095 f.p.s. to a maximum of 0.153 f.p.s., a factor of only 1.61. The maximum f_b was over five times greater than the minimum, and the slope varied by a factor of approximately four in the velocity range investigated. This comparison supports Vanoni and Brooks' conclusion No. 4g that U_{*b} is the least sensitive parameter that can be used to determine the hydraulic and transport characteristics of a flow.

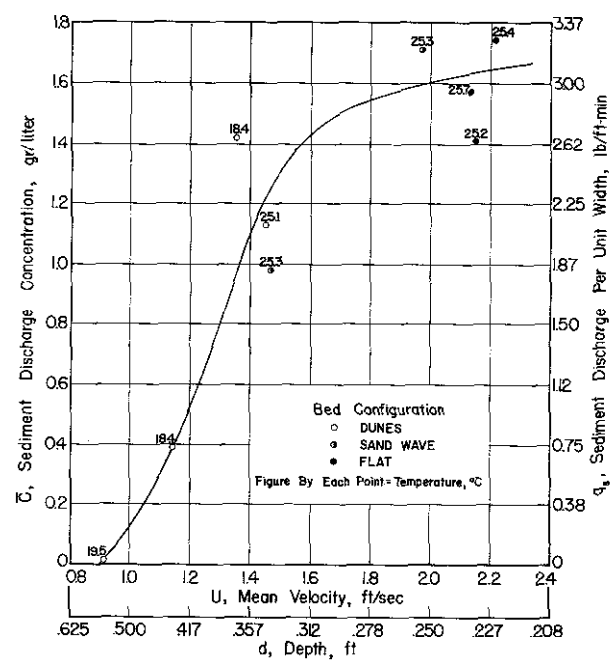


FIGURE 5. — Variation of sediment discharge concentration and sediment transport rate with mean velocity and depth for constant discharge experiments ($q = 0.50$ c.f.s. per foot).

Sediment Transport Rate

In figure 5 values of sediment discharge concentration, $C=q_s/q$, are plotted against mean velocity. Corresponding values of the sediment discharge per unit width, q_s, are shown on the ordinate scale at the right of the figure. The water temperature during the run is noted by each point.

After run 3–4 was completed, a heater was installed on the flume to control the water temperature. For run 3–5 and successive runs the water temperature was maintained at approximately 25° C. Runs 3–1, 3–2, and 3–4, performed before the heater was installed, had somewhat lower temperatures.

This temperature change is no doubt responsible for the apparent discontinuity in the velocity-sediment concentration relation between run 3–2 (U=1.35 f.p.s., C=1.42 gm./l., T=18.4° C.) and run 3–7 (U=1.45 f.p.s., C=1.13 gm./l., T=25.1° C.). The results of experiments with fine sand by Nomicos reported by Vanoni and Brooks (*19*) showed, for a constant discharge, that C and q_s generally decrease with increasing temperature. The drop in C between run 3–2 and run 3–7 is of the magnitude that Nomicos' experiments would indicate. Therefore, to put all transport data on a quantitatively comparable basis, the concentrations and sediment discharges for the runs at the three lowest velocities should be reduced about 25 to 35 percent.

With this adjustment (not shown), figure 5 indicates that C and q_s are single-valued functions of velocity for this constant discharge and that as the velocity is increased, these quantities also increase. This is consistent with Brooks' conclusion No. 4a that for a constant discharge an increase in q_s requires a decrease in d. As implied in Brooks' conclusion No. 4d, the high values of f_b are generally associated with the low values of q_s; however, the *maximum* f_b was not observed for the run with the lowest q_s, because of the slight initial rise of the f_b versus U curve (fig. 3).

The slight decrease in the geometric mean size of the sand that occurred between the time of Vanoni and Brooks' (*19*) experiments and the present investigation (see fig. 1) had a marked effect on the sediment transport rate, as may be illustrated by the comparison of Brooks and Vanoni's run 2–2 and run 3–6a of the present investigation. Table 3 gives a summary of the data for each of these runs, along with the transport rate of the individual sieve fractions and the cumulative transport rates of the material coarser than each sieve size. These two runs had identical velocities and depths, and temperatures that were nearly the same. For the material coarser than the 0.124 mm.-sieve the differences in the transport rates of the individual sieve fractions are insignificant in view of the temperature difference between the two runs, but for the finer sieve fractions the differences are striking. For each of these finer sieve fractions, significantly more material was transported in run 2–2 than in run 3–6a. This was because of the greater availability of fine material in the bed sand of run 2–2, as shown in the comparison of the sieve analyses of the bed materials in figure 1. The geometric mean sieve diameter, D_g, of the transported material of run 2–2 is accordingly much smaller, and the geometric standard deviation σ_g is larger, due to the extension of the frequency distribution toward the smaller sizes.

These two experiments point out the important effect the sand size distribution has on the sediment transport rate of the flow, and thus the necessity of accurately knowing the size distribution of both bed material and the load in analyzing, interpreting, and comparing the transport data for sediment-laden flows.

TABLE 3.—*Transport rate of individual sieve fractions*

Sieve opening (mm.)	Run 2–2 [Vanoni and Brooks (*19*)]: U=2.13 f.p.s.; d=0.233 ft.; D_g=0.095 mm.; σ_g=1.60; f_b=0.024; T=23.5° C.[1]			Run 3–6a U=2.13 f.p.s.; d=0.235 ft.; D_g=0.127 mm.; σ_g=1.41; f_b=0.023; T=25.7° C.[1]		
	Percent retained	Sediment discharge of sieve fraction	Cumulative discharge of coarser material	Percent retained	Sediment discharge of sieve fraction	Cumulative discharge of coarser material
		Lb./min.-ft.	*Lb./min.-ft.*		*Lb./min.-ft.*	*Lb./min.-ft.*
0.246	0.91	0.04	0.04	2.80	0.08	0.08
0.208	2.28	.11	.15	4.91	.14	.22
0.175	4.02	.19	.34	7.61	.22	.44
0.147	10.79	.50	.84	18.76	.55	.99
0.124	11.93	.55	1.39	17.87	.53	1.52
0.104	20.22	.94	2.33	23.15	.68	2.20
0.088	8.04	.37	2.70	10.41	.31	2.51
0.074	15.61	.73	3.43	9.42	.28	2.79
0.061	9.39	.44	3.87	2.30	.07	2.86
0.053	4.47	.21	4.08	1.89	.06	2.92
0.043	6.42	.30	4.38	.67	.02	2.94
Pan	5.92	.28	4.66	.21	.01	2.95
Sum	100.00	4.66	.....	100.00	2.95	.....

[1] U=velocity; d=depth; D_g and σ_g=analysis of sediment load; f_b=bed friction factor; and T=temperature.

Remarks on Stream Morphology

In view of recent laboratory results (*1, 2, 10, 11, 12, 15, 16, 17, 18, 19, 21*), one may well ask which stream variables are functions of which other variables. For simple physical systems there is no confusion. For example, for a simple pendulum the length L may be considered a function of the period T and the acceleration of gravity g; but on the other hand T may just as well be considered a function of L and g. It makes no difference, because the relation is unique either way. However, if in a complex system there are many variables with several interlocking relationships, involving multiple-valued functions, the analysis may easily become confused. Selection of some variables, as independent and others as dependent, then becomes very useful for avoiding confusion and clarifying what functional relations are required.

As used here the term "independent" means that which is imposed on the system externally, and "dependent" means that which results when equilibrium is reached. "Unique" means that there is only one set of values of the dependent variables corresponding to the imposed values of the independent variables, whereas "multiple-valued" means that there may be two or more sets of values of the dependent variables and that there is *no basis for selecting one or the other purely on the basis of the chosen independent variables.*

A frequent error in applying dimensional analysis to sediment transport in streams is the failure to recognize all the variables and all the interdependencies between them simultaneously. For a most elementary alluvial stream flowing with uniform depth and velocity, the pertinent variables are as follows:

Q=discharge of water;
Q_s=discharge of sediment (of sizes found in the bed);
b=width;
d=mean depth;
r=hydraulic radius;
U=mean velocity;
S=slope of energy line=slope of water surface;
f=Darcy friction factor;
ν=kinematic viscosity of fluid (including any change due to suspended clays, etc.);
ρ=mass density of fluid;
ρ_s=mass density of sediment particles;
D_g=geometric mean size of bed material;[4]
σ_g=geometric standard deviation of sizes of bed material;[4]
w=geometric mean settling velocity of particles;[5]
g=acceleration due to gravity.

The above variables are interrelated in several ways, as follows:

1. Continuity:	$Q=Ubd$
2. Definition of f:	$U=\sqrt{\frac{8g}{f}}\sqrt{rS}$
3. Roughness function:	Einstein and Barbarossa (*9*) Shen (*15*)
4. Sediment transport function:	Brown (*3*) Chien (*4*) Vanoni, Brooks, and Kennedy (*20*)
5. Width-depth relation:	Leopold and Maddock (*13*); for flumes, b=constant
6. Hydraulic radius-depth relation:	$r=d$ for wide channels; r/d depends on shape of cross section in general.

The third relation predicts the roughness, including the effect of bed forms. The fourth gives sediment transport in relation to the other variables. Since there are no universally accepted formulations of relations 3 and 4 above, some general references are given instead of explicit equations. In the field, the width and shape of the cross section that govern relations 5 and 6 are self-adjusting to the variables listed above, plus some other geomorphic factors which are not listed above, such as bank resistance, hydrologic flow regime, and geologic history.

Since there are 15 basic variables and 6 dependencies, there are 9 independent and 6 dependent variables. In a laboratory flume (or canal) with fixed walls, there are only 5 dependent variables, because width is predetermined by the channel geometry and does not depend on the flow; i.e, we delete relation 5 above. Therefore, if 9 variables (10 in flumes) are selected or specified, the other 6 variables (5 in flumes) should be physically determined by the system. In flume experiments this is found to be the case, but for rivers the relations are not so clear because it is not possible to control the variables at will for test purposes. In

[4] Logarithmically normal distributions are implied for the diameter and fall velocity of the sediment particles comprising the bed, as is frequently the case for natural streams.

[5] If all particles were spherical, w would be determined by other variables on the list (D_g, ρ_s, ρ, ν, g); for nonspherical particles an indication of the shape is thus given by w and the other variables considered together.

TABLE 4.—*Choices of independent and dependent variables for steady equilibrium flow in alluvial streams*

System	Independent variables: Properties of fluid, sediment, gravity, etc.	Independent variables: Characteristics of flow system (not all combinations listed)	Dependent variables (not all combinations listed)	Functional relations are: Unique	Functional relations are: Multiple-valued for some ranges
Flumes	$\nu,\rho,\rho_s,D_g,\sigma_g,w,g$	Q,d,b	Q_s,U,r,S,f	x	
	do	Q,Q_s,b	d,r,U,S,f	x	
	do	U,d,b	Q,Q_s,r,S,f	x	
	do	d,S,b	Q,Q_s,r,U,f	.....	x
	do	r,S,b	Q,Q_s,d,U,f	.....	x
	do	Q,S,b	Q_s,d,r,U,f	.....	x
Field:	$\nu,\rho,\rho_s,D_g,\sigma_g,w,g$	Q,d	Q_s,b,r,U,S,f	x	
Short term	do	d,S	Q,Q_s,b,r,U,f	.....	x
	do	r,S	Q,Q_s,b,d,U,f	.....	x
	do	Q,S	Q_s,b,d,r,U,f	.....	x
Long term ("graded stream")	ν,ρ,ρ_s,g	Q,Q_s	$b,d,r,U,S,f,D_g,\sigma_g,w$	x	
Very long term	ν,ρ,ρ_s,g, geology	Climate, man-made works	$Q,Q_s,b,d,r,U,S,f,D_g,\sigma_g,w$	x	

table 4 an attempt has been made to summarize experience to date in flumes and to indicate the implications for the field. Various alternate classifications of the variables are found by reading across on any line of the table.

As indicated in table 4 some choices of independent variables do not define a unique result. Thus, for the six cases with "x" in the "multiple-valued" column, the choice of independent variables is not really correct. For example, as shown in the preceding sections, specifying discharge Q and slope S in laboratory flumes will not always define uniquely the flow condition of the stream. Instead there may be several possible flow conditions, including one with low velocity, high depth, high roughness, low transport, and a dune bed, and another with high velocity, low depth, low roughness, high transport, and a flat bed. Each flow can easily be generated in a flume and will remain stable over a long period of time, because there are other constraints such as depth regulation, continuity of sediment transport, and continuity of the energy grade line.

One might be tempted to say that the problem should be formulated in two parts: one for dune bed cases, one for flat bed cases, etc. Since within each regime there will probably then be no multiplicity, one might think that the nonuniqueness problem could thus be avoided. But experience shows that the bed form itself is strongly related to the velocity and cannot be deduced solely from the independent variables listed for "multiple-valued" cases in table 4; thus we are back where we started from with an indeterminant relation.

The slope has very commonly been thought of as an independent variable, but flume experience quickly teaches otherwise. In a typical flume experiment, one sets the flume to some estimated slope and then waits to see if the energy gradient of the established flow matches that of the apparatus; if it does not, then the *apparatus* slope must be adjusted to the *energy* slope. Alternatively, the *energy* slope can sometimes be made to match that of the flume by successive adjustments of the discharge or depth in a closed circuit flume, or the discharge and the sediment feed rate in an open circuit flume.

In the field the identification of independent and dependent variables, or cause and effect, is clouded by long-term intercorrelations naturally established; parameters cannot be varied at will but have certain values produced by the system itself. For example, at a given time and place one may find by sampling that the bed sand for a given stream has a certain mean diameter D_g, which is indeed "imposed" on that stream for any conditions occurring in the near future ("short term"). But from a broader point of view, it must be recognized that the stream itself established the D_g value through years of transport, sorting, and abrasion. Therefore, for the short term the bed sand size D_g is an independent variable, but on the geologic time scale D_g is certainly a dependent variable. Similarly, the slope of an alluvial river changes very slowly, but at any time the slope is indeed a slope that is compatible with the geology and general hydrologic flow regime because these factors over geologic time have established the slope.

The time scale under consideration is also important in classifying Q and Q_s as dependent or independent variables. Streams are seldom, if ever, in a steady state (because of finite time required to change bed forms and depth), and transitory adjustments are accomplished by storage of water and sediment. Water storage is relatively short (hours and days) and occurs simply by a rise of river stage or overbank flooding; sediment storage (plus or minus) occurs by deposition or scour. Over the short term Q_s, the sediment transport rate, in any reach, will depend on Q and the associated value of d re-

quired to give continuity in the water surface and energy grade line at the ends of the reach at any moment. Thus, for the short term, Q_s may be considered a dependent variable, with departures of the sediment inflow from the equilibrium transport rate being absorbed in temporary storage (for perhaps months or years).

However, in the long term the river must assume a profile and other characteristics for which, on the average, the inflows of water and sediment equal the outflows; consequently, for this case (called a "graded stream" by Mackin (*14*)), Q and Q_s are listed in table 4 as the independent variables. In this instance, Q and Q_s need not necessarily be particular single values but also may be considered to represent related sequences of values with fixed statistical characteristics. For the very long term, streams respond to the geology of the region, the climate, and the properties of matter. From this point of view, every characteristic of a river is dependent on something else.

Even for the short term in the field the occurrence of discontinuous rating curves for streams (*5*, *7*) suggests that depth and slope are not always sufficient to define the instantaneous discharge. In fact, the authors are of the opinion that alluvial rivers naturally evolve into the ranges of variables (slope, depth, velocity) where relations are multiple-valued due to changes in bed form. If generally so, this is indeed fortunate, for alluvial streams respond to flood flows by reducing their hydraulic roughness as well as by increasing stage. If the roughness did not change, then far greater increases in stage would be required to allow passage of floods! Similarly, high roughness with low flows is beneficial, since it provides adequate depth for navigation with as little flow as possible.

Until recently sediment transport equations have almost universally been built around the idea that bed shear is an essential parameter. [See summary as of 1950 in (*3*).] Yet table 4 shows that r and S, and hence the shear $\tau = \rho g r S$, do not uniquely determine the flow or the sediment transport rate. The experiments reported in the preceding sections show that the shear velocity changes least of any variables, and thus is certainly a poor starting place for a transport calculation. Colby (*6*) has recently suggested that stream velocity is a far better variable with which to correlate sediment transport.

In this regard, Einstein (*8*) and Laursen (*12*) have recently used τ', the grain resistance, in place of τ in formulas for sediment transport. But τ' is given by

$$\tau' = \frac{f'}{8} \rho U^2$$

where f' is a friction factor expected for flow over a flat bed, and may be taken from the pipe-friction diagram or the Prandtl-Kármán resistance equations (in the ways indicated by Einstein and Laursen). Since f' is not related to the total friction factor f for the actual flow, τ' is not a measure of the total shear but is for all practical purposes a parameter reflecting the velocity. Thus, there has been a subtle shift toward computing sediment transport from velocity rather than from total shear stress.

Summary and Conclusions

The present experiments corroborate, for this sand and unit discharge, Brooks' conclusion No. 4b that for a given slope and discharge two different depths of flow are possible in a laboratory flume. This conclusion is valid, however, only for slopes in the range where major changes in the bed configuration occur. It was also found that for runs at a constant discharge, the velocity cannot in general be expressed as a single-valued function of shear velocity. However, in the range investigated, if the velocity is used as the independent variable, then the slope, bed friction factor, and bed shear velocity are uniquely determined; i.e., they are single-valued functions of velocity.

For these runs at a constant discharge, an increase in the sediment discharge required an increase in the velocity and a corresponding decrease in the depth; this is in agreement with Brooks' conclusion No. 4a. The results of these experiments qualify Brooks' conclusion No. 4d that the largest bed friction factors are associated with the smallest sediment transport rates, because, with increasing velocity, the bed friction factor first increased slightly, then decreased. However, the sediment discharge increased steadily with increasing velocity.

A comparison of the results from the current experiments and the earlier experiments of Vanoni and Brooks indicated that even very slight changes in the size distribution of the bed material can have a significant effect on the sediment discharge rate and the size distribution of the transported material.

Table 4 shows various possible choices of independent and dependent variables for describing alluvial streams in laboratory flumes and in the field, and indicates which relations are unique and which are non-unique. In the field it is necessary to differentiate between long term and short term points of view. Whereas sediment discharge, Q_s, may be considered one of the *independent* variables for a particular reach of river in the long term, it is more likely to be a dependent variable for the short term. A graded river adjusts itself to the flow of water and sediment in the long term, but during short

periods the sediment inflow to a reach can be far out of balance with the equilibrium transport rate, resulting in short term sediment storage.

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SPIRAL MOTION, SEDIMENT TRANSPORT, AND RIVER DEVELOPMENT

[Paper No. 38]

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Abstract

In fluids, spiral motion is common.

Turbulence behind a falling grain arises from three vortex centers, spaced around the perimeter at about 120° intervals, which shed vortices in sequence. Shedding of an individual vortex tends to rotate the grain in a predictable direction, which, in turn, produces a spiral path.

At Froude numbers above one, shallow water passing an obstacle exhibits a vortex trail, having a spiral nature (down on the outside, up on the inside). Outside of each line of vortices, a standing spiral wave may develop (also down on the outside, up on the inside). Two waves make an ogive behind the obstruction. A roughness element on the channel wall creates one wave. Two waves, from opposite walls, cross near mid-channel in the "rooster tail." In shallow water, spiral waves arise at low velocities, and erode the bed, without any vortex trail being present.

A larger spiral overturn (helical, or secondary flow) operates to divert the main stream of water in the direction of the slow (i.e., rising) side. In deep alluvial streams, this process may be responsible for meandering; in wide, shallow streams, it may induce a system of higher order spirals that, taken together, account for the braided pattern.

Introduction

Spiral motion in fluids is common. It appears to be important in a variety of processes in the atmosphere and in the hydrosphere (*16*). It definitely plays a part in the shaping of stream channels and water current beds, and it may be significant in the transportation of sediment.

Above a certain transition zone (commonly defined as near R=3,000), fluid flow is generally turbulent. The rotational motion within this turbulence can be combined with translation, or straight-line movement, to produce a spiral pattern of sorts and, hence, provide a basis for the general statement that spiral motion in fluids is common. It is the purpose of the present paper, however, to present more specific observations of spiral flow.

Single Grain Studies

Individual grains falling through a fluid medium have been the subject of study for a century or more (*2, 7*). Terminal velocities have been established for certain single grains, under specified conditions. More refined experiments have shown that this terminal velocity depends greatly on the shape and surface of the grain. Rarely has it been observed, however, that even with a single smooth spherical grain, settling through a stilled medium, it is difficult to obtain straight-line motion.

Investigation of behavior of falling grains can be carried out conveniently by means of "scaled up" models, which are large enough to permit detailed examination of flow patterns. For example, we may wish to use a model, 4 cm. in diameter, to represent a quartz grain 0.4 mm. in diameter. The model ratio of length is thus specified as L=100. Inasmuch as the model and the prototype both operate in the earth's gravitational field, the acceleration of gravity is the same for both and the model ratio of gravitational acceleration is unity. This, however, specifies a rigid relation between the dimensions length and time, inasmuch as acceleration is a length per time squared. Substituting 100 for L, and solving the equation (LT^{-2}=1), we determine a model ratio of time as T=10. The scaled-up model must be 100 times as wide as the prototype and must take 10 times as long to settle an equal distance. Settling velocities can be controlled by altering the mass of the model. In the experiments described here a Ping-pong ball nearly full of water has been used as the model (fig. 1).

One additional fact must be stated: the Reynolds regime for the model must be similar to that of the prototype. Inasmuch as length has been specified (arbitrarily), time has been derived from length and mass derived from velocity; only fluid viscosity remains to be determined. The experimenter is free to vary fluid viscosity as he wills, provided he does not alter settling velocities beyond his ability to correct by means of adjusting mass. A wide range of fluid viscosities, from 1,600 cp. to 1 cp. (centipoises), has been used.

The model that meets all of these requirements has interesting behavior indeed. For Reynolds regimes in the range of turbulence, no straight-line fall has been produced. Straight-line fall has been obtained only with very heavy metals (i.e., steel) moving through very viscous liquids (i.e., 1,000–1,600 cp.). Reynolds numbers for these models have been R=1 or less. Laminar motion of fluid elements has been established by use of dye patches on the surface of the model grain.

At Reynolds numbers in the neighborhood of R=10, turbulence appears behind the falling model. (If we wish to maintain the fiction that R=3,000 is an unbreakable limit, we must introduce a factor of roughly 1,000, which we might identify as a "grain fudge factor," into the Reynolds number definition.) This turbulence can be studied by dye tracer methods. Turbulence has occurred in all models that have been run in the range of R=10 to R=1,000.

Three vortex centers commonly appear on the trailing side of the falling model. These are spaced around the periphery of the model at about 120° intervals. Vortices are shed, in sequence, from each of the three centers. Addition of roughness elements to the model surface tends to hasten the development and increase the size of the vortices.

The Kármán vortex trail (*7*), commonly observed in the wake of a boulder protruding above the surface of a rapidly flowing stream, arises from two vortex centers that exist in the plane of the water surface. The turbulence pattern behind the falling model is analogous to the water-surface vortex trail, except that it is three-dimensional (and has three centers) instead of two-dimensional (and having two centers).

The motion in the vortices is such that a filament of fluid directly behind the falling model tends to move in the same direction as the model. In the water-surface Kármán vortex trail, behind an obstruction to flow, this filament actually may move in an upstream direction. Development and detachment of a single vortex,

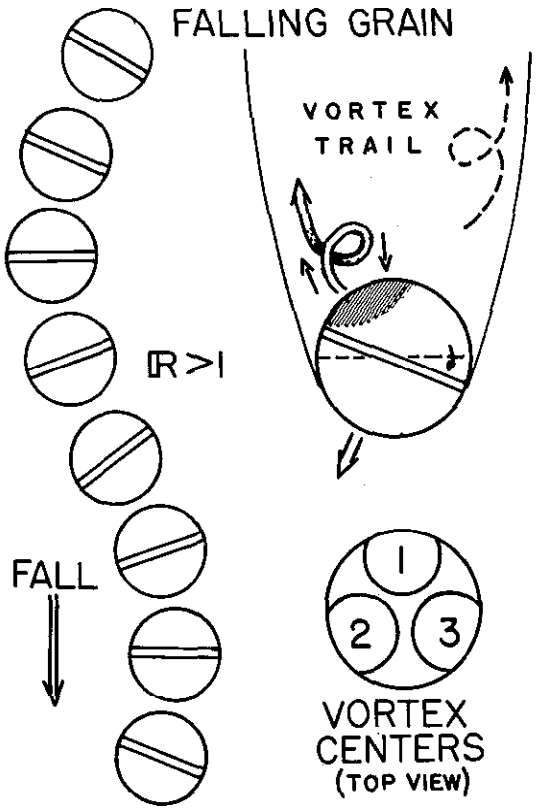


FIGURE 1. — Settling velocities and regimes.

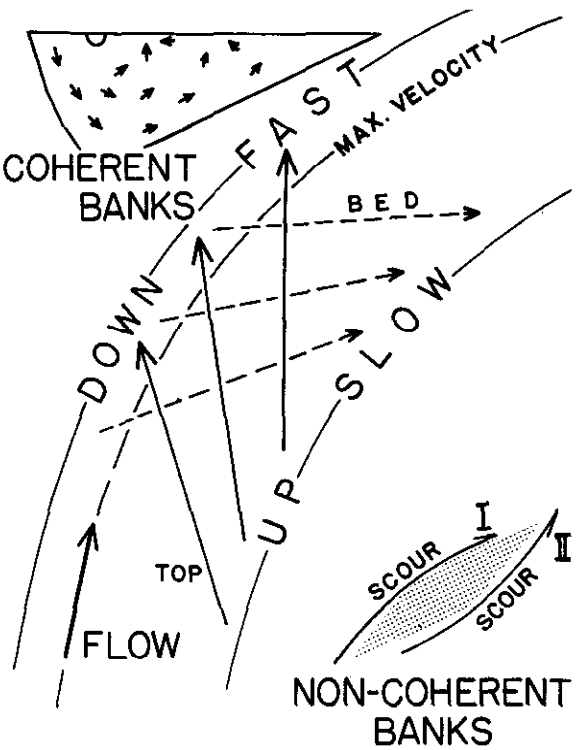


FIGURE 3. — Directions of scour patterns.

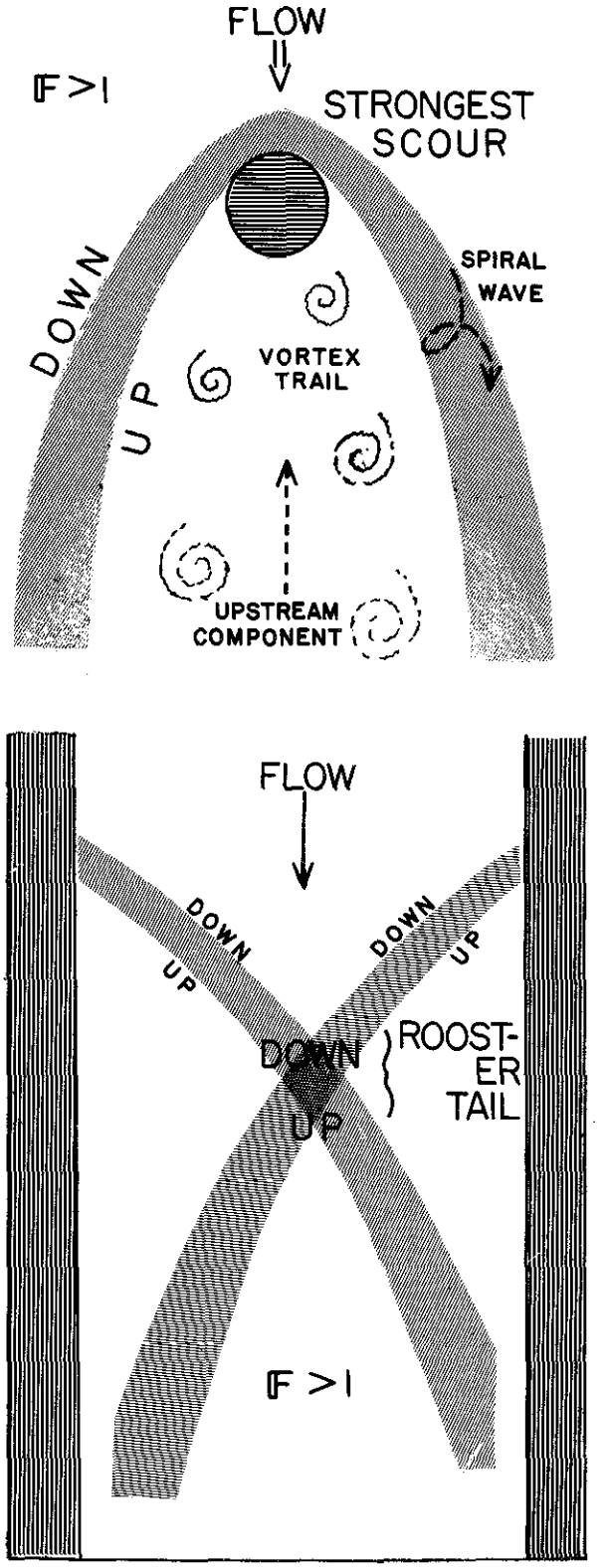


FIGURE 2. — Vortex and wave patterns of streamflow for Froude numbers above 1.

located to one side of the downward-moving particle, tends to rotate the model and to modify its direction of fall. When a second vortex develops, the rotation and fall direction are modified again. Continuous shedding of vortices combines these modifications into smooth movements, which are seen by an observer as a spinning effect and a spiraling effect.

The spiral can be noted from the side, but is most obvious when seen from above. The spin can be made clear by painting a bright equatorial band on the model grain.

Coarse to medium sand-sized particles, if spherical and smooth, settle with Reynolds numbers in the neighborhood of $R=10$ or $R=100$ and spiral and spin. Dye studies show turbulence in their wakes. If the grain shape is altered from a sphere and the surface is roughened suitably, the spiral path can be obtained with fine or very fine sand. Straight-line, or laminar-flow, settling appears to be limited under natural conditions to silt-sized particles.

Because the two settling mechanisms are so vastly different, a single settling velocity curve cannot be constructed for all common sediment sizes. Settling velocities of coarser particles are approximated by the impact law; for finer particles, by Stokes' law, as is well known.

It is possible that this difference between the two size ranges is important in understanding the relative ease with which the coarser grains are placed in motion. Turbulence at the bottom, with attendant momentary increased velocities, might be significant in the scour mechanism, in a way analogous to the spinning and spiraling of the falling model. Bed velocities that would be needed would be approximately 1 cm./sec. for coarse sand sizes and 10 cm./sec. for fine sand sizes. Bed velocities in this range should be fairly common.

The Spiral Wave

At Froude numbers above one, shallow water flowing past an obstacle exhibits the Kármán vortex trail. Motion within individual vortices is both down and downstream near the boundary between the vortex trail and the main flow, and up and upstream near the boundary between the two sets of vortex elements. This pattern is responsible for the pointed deposits of sand which accumulate, in some cases, behind cobbles and boulders (fig. 2).

Outside of the vortex trail, a standing spiral wave may develop. In a plan view, the wave is gently convex. Two standing waves that arise from a single obstruction combine to make an ogive pattern with its tip near the upstream edge of the boulder. The motion in each wave is down on the outside (convex side), up on the inside (concave side). Because of the general downstream motion, flow of a single parcel of water follows a spiral path. Maximum local velocity should be found in the strip of down flow. Scour can be expected, in favorable instances, along this strip (*17*). Maximum scour should develop near the apex of the ogive, immediately upstream from the obstruction. Scour of this type has been observed in natural high-gradient streams and in flumes and small models. In some cases, the spiral wave develops without the benefit of any obvious obstruction; the precise mechanism, in this instance, is not clear. In shallow water, it is not necessary that a vortex trail be present. The latter is a Reynolds regime effect; the spiral wave a Froude regime manifestation. Vortex trails without spiral standing waves appear when the Froude number is less than 1, but the Reynolds number is on the order of $R=100$ to $R=1{,}000$.

A roughness element on a channel wall may generate a single-spiral standing wave, which forms half of the ogive pattern, and vanishes in the downstream direction. Two roughness elements, on opposite channel walls, may give rise to two spiral waves that cross in midstream. At the junction of these waves, a spout, or breaking wave or "rooster tail," may appear (*4*). In the models that have been studied to date, the geometry of the rooster tail appears to have been related to the width-depth ratio. The characteristic width that is used is not necessarily the channel width, inasmuch as spiral waves may be produced by adjacent roughness elements on the channel bed (i.e., two adjacent cobbles or boulders).

The rooster tail is commonly associated, over sand beds, with violently breaking antidunes and patches of turbulent suspended sand. Individual grains pass through these patches with momentary delays.

In water 1 cm. deep, a velocity of about 30 cm./sec. is enough to initiate a spiral wave, which may, in turn, erode the bed. The critical point at which both vortices and the spiral wave may appear is roughly defined by $F=1$ and $R=10$; this occurs in water where the characteristic length is about 0.35 cm. and the velocity is about 50 cm./sec.

The Larger Spiral

Spiral flow on a larger scale, at Froude numbers less than $F=1$, and without benefit of a standing wave, is common in moving fluids. Retardation of part of a moving parcel of air or water, perhaps because of roughness, results in veering and overturning (*1*). The current tends to bend toward the decelerated side. The slower part of the fluid rises, and the faster part sinks [also note Shen (*9*)]. A general spiral develops, which is damped out in a map distance equal to some tens of times the water depth (*14*). The velocity gradient, from the

bed toward the thread of maximum current motion, is steep on the fast, descending, outer side, and is gentle on the slow, ascending, inner side (*5*).

The various effects that have been listed here (deceleration and acceleration, veering, overturning) appear to arise simultaneously. They appear in initially straight streams when the Froude number is less than $F=1$ and the Reynolds number is greater than $R=100$; it is possible that they also appear at somewhat lower Reynolds numbers. These effects do *not* depend on the presence or absence of mobile sediments of any kind (*10*).

Inasmuch as the descending part of the current is significantly faster than the ascending part and inasmuch as the velocity gradient is much steeper where the water descends, active scour occurs here. This scour is directed both downward, toward the channel bed, and laterally, toward the channel wall (fig 3).

Where a spiral cell begins to dissipate, the current is *not* flowing directly down the gravity slope. This fact predisposes the next ensuing spiral cell to veer and overturn, in such a way that direct downslope motion will be established. In other words, spiral flow alternates in direction. This oscillation accounts for the offset pattern of scour holes in a straight channel.

Lateral scour, at each scour site, is the crux of the meandering process. Ultimately a "most efficient" curvature will develop, after which the meander geometry stays within certain limits but "slides" down the length of the stream valley.

Actual circulation within a stream channel is much more complex than stated above. Secondary and higher order cells complicate the picture greatly (*3, 5*). However, the basic flow pattern is as indicated, and it has been confirmed over a period of many years by various workers under a variety of conditions. The chief remaining question is, which comes first, the spiral cell or the meander loop? This question must be answered in favor of the spiral cell, which is frequently observed in straight channels.

Where streambanks are composed of more-or-less coherent materials (such as sand with a small admixture of clay or organic matter), a deep channel is maintained and meandering arises unless velocities are too low. Where streambanks are composed of noncoherent materials (loose sand; gravel), a deep channel cannot be maintained, and the width-depth ratio of the water becomes fairly great. Under these conditions a single spiral cell occupies only a relatively small part of the total channel width and induces second and higher order cells in nearby parts of the current (*6, 11, 12*). Any two adjacent cells have opposite senses of spiral and curvature.

If two cells face each other (concave facing concave), an elliptical, or diamond-shaped, area of rather slow, rising water appears between them (*13*). This area is a site of potential deposition; in it, a diamond-shaped bar can be built. If two adjacent cells are back-to-back (convex against convex), a narrow, swift, descending current appears, resulting in scouring of the streambed. Because of the time lag in the process of inducing higher order cells, adjacent spirals are generally staggered rather than being located precisely face-to-face or back-to-back. This staggering of the diamond-shaped bar pattern produces the phenomenon known as braiding.

Other Spirals

Spiral flow has been observed in connection with dust devils, sand dunes, tidal flat ripple marks (*15*), and possibly sand "waves" on the sea floor. It also occurs under a variety of artificial conditions, including Reiner's "teapot effect" (*8, p. 26*), the bath tub vortex, and in surface tension and other models. Some of these effects appear to be significant in the transportation of sediment and the shaping of land forms, but are not of interest in connection with river studies.

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CHANNEL DESIGN FOR MODIFIED SEDIMENT REGIME CONDITIONS ON THE ARKANSAS RIVER

[Paper No. 39]

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Synopsis

The Arkansas River in Arkansas and eastern Oklahoma is being developed for navigation, flood control, hydroelectric power generation, and other uses by means of a series of navigation locks and dams and upstream storage reservoirs. Entrapment of sediment in the upstream storage reservoirs and in the larger ones of the main stem navigation pools will reduce the present 100-million-ton-per-year sediment load on the lower river by about 90 percent and will induce extensive degradation of the streambed downstream from the dams. The project plan provides for taking advantage of the degradation by spacing out and reducing the number of low-lift navigation locks and dams from that which would be required to match the natural river profile. The navigation channel in the upstream ends of the pools, to be developed initially by dredging and contraction work in advance of natural degradation, was designed to conform to the modified regime conditions by application of relations correlating the channel geometry and slope, streambed composition, water discharge, sediment load capacity of the channel, and sediment supply.

Description of Project

The Arkansas River in Arkansas and eastern Oklahoma is being developed for navigation, flood control, hydroelectric power generation and other uses. The plan of development for the multiple-purpose project is shown on figure 1. Included in the project are a number of large storage reservoirs that serve or will serve variously for flood control, hydroelectric power, water supply for navigation, domestic and industrial water supply, and sediment detention.

The navigation feature of the project provides for a channel following the Verdigris River from Catoosa, Okla. (near Tulsa), downstream to the Arkansas River, thence along the Arkansas River downstream to the vicinity of Arkansas Post, Ark., from which point the route will follow a canal to the White River, then down the White River to the Mississippi River. The authorized minimum navigable depth is 9 feet over minimum widths of 250 feet in the Arkansas and White Rivers and 150 feet in the Verdigris. A depth of 12 feet is authorized for dredging in canals and on limiting shoals. The waterway will be canalized by a series of navigation locks and dams. Four of the navigation dams on the main stem will have moderately high heads ranging from 30 to 54 feet. Two of these will include provisions for hydroelectric power generation initially, and the other two will be so designed as to permit possible future addition of power facilities. The remaining navigation locks and dams will be relatively low-lift structures (generally 20 feet or less) with pools confined within the river channel. The low-lift dams will be gated and the gate sills will be at approximate streambed level, with the result that essentially open river conditions will prevail at the higher flows in excess of approximately half-bankfull capacity.

Stabilization and rectification of the rapidly shifting channel of the Arkansas River by means of stone and pile revetments and dikes is an additional major feature of the project. The stabilization and rectification work will

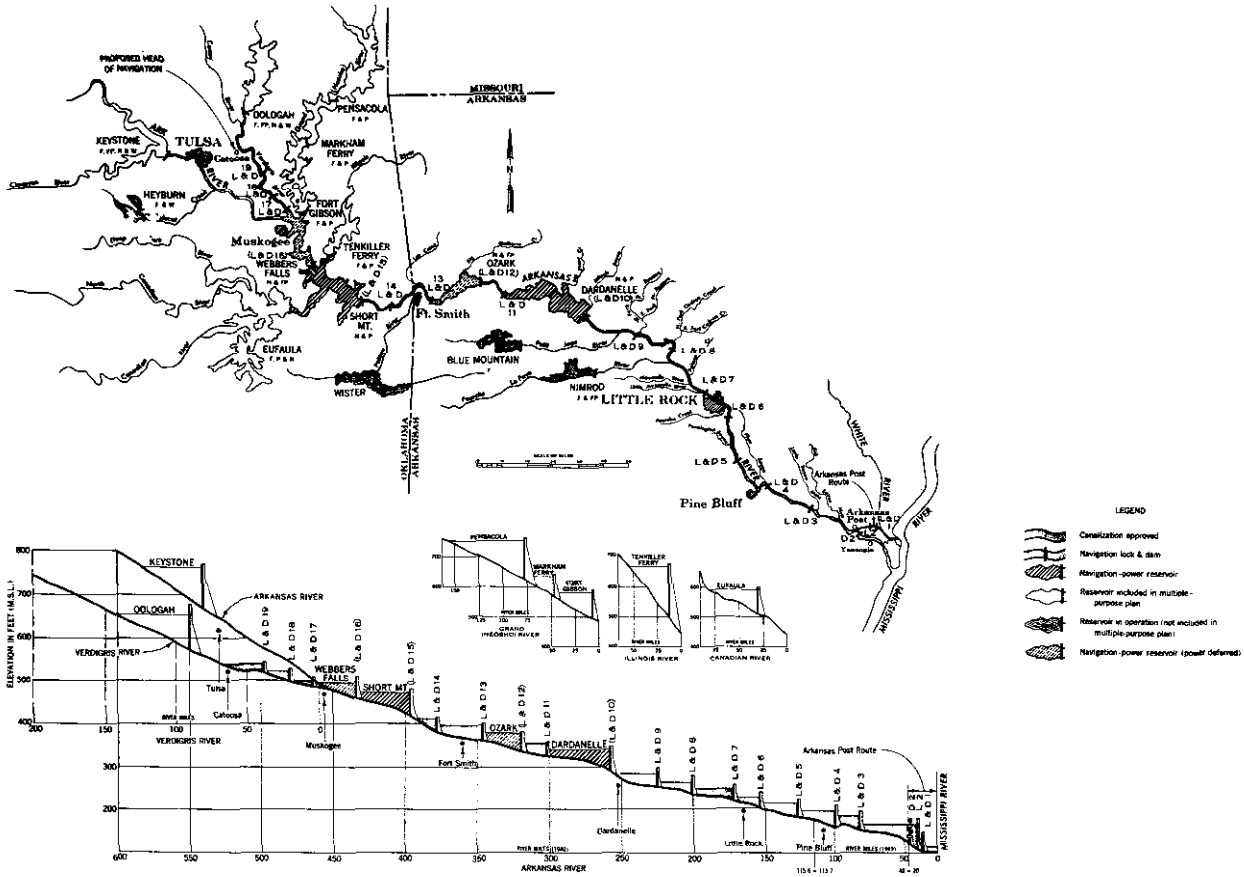


FIGURE 1. — General plan and profile of the multiple-purpose plan for the Arkansas River and tributaries in Arkansas and eastern Oklahoma.

extend from Short Mountain Lock and Dam to the vicinity of Arkansas Post and will be continuous throughout except in the larger main stem pools and in a number of short naturally stable reaches. The project design contemplates taking advantage of the expected degradation of the channel downstream from the larger main stem dams and provides for special measures for dredging and channel contraction in the upper ends of the navigation pools in the interest of increasing depths, so as to permit spacing dams farther apart than could otherwise be done. A description of the design procedures used to determine the degree of contraction and amount of dredging required to produce and maintain a channel of the required navigable depth and to conform to the modified regime conditions is the principal subject of this paper.

Construction Schedule

The entire Arkansas River multiple-purpose project is scheduled for completion in 1970. Construction of the low-lift navigation locks and dams, which will begin in 1964, will be carried on generally in upstream order so as to provide a completed navigation channel successively to Pine Bluff, Little Rock and Fort Smith, Ark., and the vicinity of Tulsa, Okla. Oologah, Pensacola, Fort Gibson, Tenkiller Ferry, Heyburn, Wister, Blue Mountain, and Nimrod Reservoirs are existing. Dardanelle, Keystone, Eufaula, and Markham Ferry Reservoirs are under construction, with filling of the pools expected to begin in 1964. The bank stabilization and channel rectification work is about 50 percent complete. The final phases of the contraction and dredging in the head of each pool will be completed in coordination with the completion of the next upstream navigation lock and dam. As will be brought out later, the sequence of construction of the reservoirs and the locks and dams is a major factor in the design for the contraction and dredging work.

Stream Characteristics

Streamflow

Average flows in the reach of the main stem of the Arkansas River to be traversed by the navigation route range from about 20,000 c.f.s. at Muskogee, Okla., to 42,000 at Little Rock. Representative maximum unregulated peak flows are 700,000 c.f.s. at Muskogee, 875,000 in the vicinity of Fort Smith, Ark., 720,000 at Dardanelle, and 700,000 at Little Rock. Regulation by the storage reservoirs will reduce these peaks to 560,000, 516,000, 492,000, and 448,000 c.f.s., respectively, the reductions ranging from 25 to 41 percent of the unregulated peaks. Minimum monthly unregulated flows downstream from the Canadian River range from 519 c.f.s. at Muskogee to 874 at Little Rock. Corresponding minimum monthly regulated flows will range from about 3,700 to 4,600 c.f.s. Bankfull channel capacities are about 160,000 c.f.s. in the vicinity of Fort Smith and 250,000 at and downstream from the vicinity of Dardanelle.

Slopes

Natural river slopes average about 1 foot per mile between Muskogee and Fort Smith, 0.9 between Fort Smith and Dardanelle, 0.8 between Dardanelle and Little Rock, 0.7 between Little Rock and Pine Bluff, and 0.6 between Pine Bluff and the mouth of the river.

Channel Dimensions

Natural channel widths in the pertinent part of the Arkansas River between Short Mountain Dam site and Arkansas Post range from about 800 feet at some localized naturally stable areas to 3,000 feet in a number of unstable "wild" reaches. Bank heights range from about 20 to 30 feet above mean low water. Depths in pools are generally from 5 to 20 feet below mean low water, and limiting depths on many crossing shoals are less than 3 feet at mean low water and 1 foot or less at extremely low stages.

Sediment

The large sediment load carried by the Arkansas River presents the major problem in the planning and design of the project. The natural suspended sediment load downstream from the mouth of the Canadian River ranges from 95 million tons per year at Short Mountain Dam site to 105 at Little Rock. About 24 percent of this is suspended bed-material load consisting of sand sizes contained in the streambed in appreciable quantities. About 50 percent of the suspended load is contributed by the Canadian River, 25 by the Arkansas River upstream from Tulsa, and the other 25 by the other tributaries in eastern Oklahoma and Arkansas. Bedload, or the material rolling or sliding generally in contact with the bed, is estimated, on the basis of calculations by bedload formulas and from some rough measurements, to be about 5 percent of the suspended bed material load. The streambed is composed predominantly of fine to medium sand, with smaller percentages of very fine sand and of material coarser than medium sand, including appreciable percentages of gravel ranging up to 1 or 2 inches in size.

Effect of Reservoirs on Sediment

As part of the design studies for the project, computations were made to estimate the amount of deposition in each of the reservoirs and in the pools of the moderate-head main stem dams and the sediment outflow from the dams. In effect, the loads were routed downstream through the pools and the open river reaches between pools, taking into account the loads contributed by the tributaries and scoured from the streambed and unstabilized banks of the open river reaches. This routing of flows was carried as far downstream as Dardanelle Dam and covered a period of 50 years. The computations indicated a sediment outflow of about 10 million tons per year from Dardanelle Dam, which is one-tenth the natural load transported past this site. About 8 million tons per year of the sediment outflow would consist of the silt clay fractions and 2 million tons per year would be sand. The details of the computation procedures are beyond the scope of this paper. However, in summary, the sediment deposition in and outflow from the large run-of-river mainstem pools were estimated from curves of relation between the detention time in the reservoirs and the ratio of sediment deposition to sediment inflow. These relationship curves were developed from data of existing reservoirs.

Channel Degradation

With such a large reduction in sediment load as that indicated by the studies described in the preceding paragraph, extensive degrading of the streambed can be expected to take place downstream from Dardanelle, Ozark, and Short Mountain Dams as the river attempts to replenish its sediment load by scour of the streambed. As the degradation proceeds downstream from each of these dams, the stream slope will flatten and the material in the streambed will coarsen, with the result that the transport capacity will reduce until a condition of equilibrium is established between the transport capacity of the channel and the reduced sediment outflow from the dams. Additional lowering of the streambed in some places will result from the shortening of the length of the river by the develop-

ment of artificial cutoffs and bend easing as part of the bank stabilization and channel rectification work. This shortening will amount to 23 miles between Arkansas Post and Dardanelle Dam and 7 miles between the head of Ozark Reservoir and Short Mountain Dam. The degradation not only will make available a large increase in head for generation of power at the main stem power dams but also will make it possible to increase the spacing between and reduce the number of the low-lift navigation dams from those that would be required to match the natural river profile. Natural degradation will progress slowly downstream and would be retarded and eventually arrested by the formation of an armouring layer of gravel on the bed surface, such as has been observed in degrading streams elsewhere and in sediment transport flume studies. Therefore, dredging and contraction work will be performed in the heads of the navigation pools in advance of the natural degradation, in order to provide a channel of the required navigable depth by the time the navigation locks and dams are completed in the separate sections of the river. The dredging will prevent the gravel armouring from inhibiting degradation. Any armouring that occurs after dredging will be beneficial in stabilizing the dredged channel and in preventing excessive degradation which could be detrimental to the locks and dams.

Channel Design Procedure

General

In establishing the damsites and pool elevations, it was necessary to compute the dimensions and bottom grade elevations of dredged channels in the upstream ends of the pools that would have the required navigable depth below the flat pool level and which could be maintained in equilibrium by a reasonable amount of contraction works. The start of navigation will be the most critical time for obtaining navigable depths. Therefore, the sediment load as modified by the reservoirs at that time will be the controlling equilibrium condition for channel design. Continued degradation after the initial dredging will improve navigable depths. The term "equilibrium" defines the condition under which, for a given discharge or sequence of discharges, the sediment transport capacity of a channel is equal to the sediment supply and neither scouring nor filling of the channel bottom will take place as long as that discharge or sequence of discharges and corresponding sediment supply remains the same.

The channel design procedures, therefore, required the development and application of functional relations correlating the geometry, slope, and roughness of stream channels; the grain-size composition of the bed; water discharge; sediment transport capacity; and sediment supply. Inasmuch as the silt clay fractions are not contained in the streambed in appreciable quantities and as these sizes can be expected to pass on through the open river channel and small navigation pools as wash load without appreciable deposition, the design procedures considered only the bed-material load or sand fraction. The relations and their application with particular reference to the channel reach between Arkansas Post and Dardanelle Dam are described.

In brief, the procedure involved the selection of the critical sediment supply that the design channel must transport with each of several water discharges; the trial assumption of a contracted channel width; the determination of the depth and slope required to transport the given discharge and sediment supply; and the corresponding channel bottom grade elevation. If the resulting channel bottom elevation did not provide the required navigable depth below the flat pool level, a new channel width was assumed and the process was repeated until the navigable depth was attained. If the required contracted channel width was less than approximately 1,000 feet, the pool elevation and dam site were revised as necessary.

Other major factors governing the pool elevations and locations for the dams were the effects of the pools on surface drainage and ground water levels on adjacent lands, the effect of channel alinement on navigation conditions in the approaches to the locks, availability of sufficient channel width to provide space for the lock and an adequate spillway, and, to a lesser extent, the foundation and abutment conditions. Provision of adequate harbor facilities was a factor at some localities.

Natural Sediment Load

A rating curve of suspended bed-material load of various sand sizes versus discharge at Little Rock for natural conditions is shown on figure 2. This curve was based on numerous measurements over a period of several years at a bridge at Little Rock. This curve was used to determine the sediment supply in the part of the river downstream from Little Rock to which it was assumed that load reduction effects of the reservoirs would not have progressed by the time of start of navigation.

Development of Relation Between Channel Depth and Slope and Sediment Load

For flexibility in the channel design studies, it was considered desirable to express the relation between stream and channel characteristics, discharge, and bed-material sediment load in generalized terms, so that the effects of

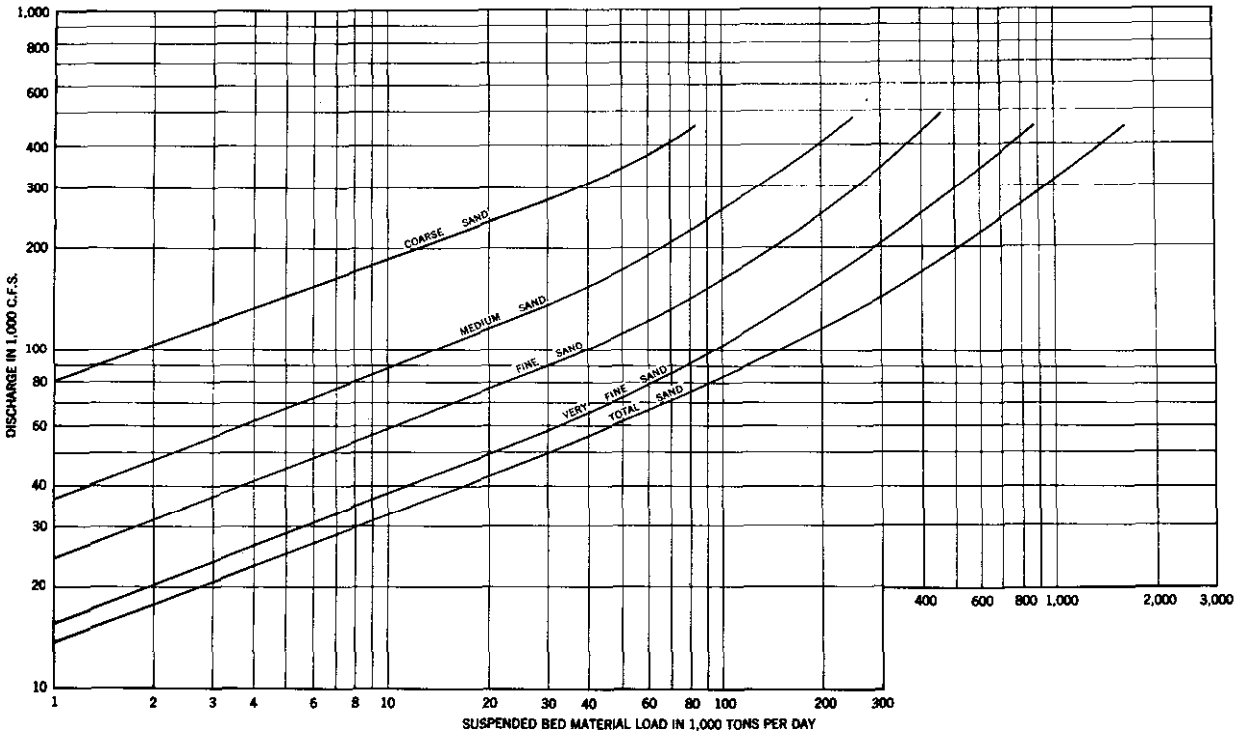


FIGURE 2. — Rating curve of suspended bed-material sediment load versus discharge for the Arkansas River at Little Rock, Ark.

changes in the various parameters involved could be valuated. A functional relation developed empirically by Emmett M. Laursen [1] was used as the framework for developing a generalized working curve for use in the Arkansas River channel studies. Laursen's relationship was adopted for use, because it is expressed in terms that permit separating readily the effects of the various parameters which are generally considered to govern the relation between the bed-material sediment load, the hydraulic characteristics of the streamflow, and the characteristics of the bed material. In addition, being empirical, the Laursen relationship is susceptible of being adjusted to fit Arkansas River observations in a manner similar to the adjustment to fit Missouri River data described by D. C. Bondurant.[2] An average relationship curve, based on data collected in three sets of detailed sediment load and velocity measurements made on the Arkansas River in the vicinity of Dardanelle and Morrilton and plotted in the form used by Laursen, is shown on figure 3 together with the curve from Laursen's paper. As the Arkansas River sediment load measurements included the suspended load only, the plotted data have been adjusted to approximate the total bed material load by adding in values read from the curve designated "bedload" on Laursen's graph. The bedload, or load moving in contact with the bed, represents only a very small part of the total bed material load. The curve based on Arkansas River data falls in the general range of data presented by Mr. Bondurant for the Missouri River at Omaha. The curves in figure 3 are plotted as a relation between the expressions $\frac{\sqrt{\tau_o/\rho}}{w}$ and $\frac{c}{\left(\frac{d}{D}\right)^{7/6}\left(\frac{\tau_o'-1}{\tau_c}\right)}$, where $\sqrt{\tau_o/\rho}$ is the shear velocity at the streambed in feet per second, τ_o is the boundary shear or tractive force at the streambed in pounds per square feet, τ_o' is the boundary shear associated with the sediment particles in the streambed, τ_c is the critical tractive force for beginning of movement of the sediment particles, ρ is the mass density of fluid (1.94 for water), w is the fall velocity of sediment in feet per second, c is the concentration of sediment in percent by weight, d is the diameter of sediment

[1] LAURSEN, E. M. THE TOTAL SEDIMENT LOAD OF STREAMS. Amer. Soc. Civil Engin. Proc. 84, HY1 (Paper 1530). 1958.

[2] BONDURANT, D. C. DISCUSSION OF REFERENCE LISTED IN FOOTNOTE 1. Amer. Soc. Civil Engin. Proc. 84, HY6: 64-74.

particle (mean diameter of fractional size range) in feet, and D is the depth of flow in feet.

Additional pertinent equations are as follows:

$$\tau_o = \gamma DS = 28.25 \frac{n^2 V^2}{D^{1/3}}$$

$$\tau_o' = \frac{V}{30}\left(\frac{d_m}{D}\right)^{1/3}$$

$$\tau_c = 4d$$

$$q_s = 27\, qc$$

where γ is the specific weight of water (62.4 lb./cu. ft.), S is the energy gradient in feet/feet, n is the Manning roughness coefficient, V is the mean velocity (q/D), d_m is the median size of sediment particles in streambed in feet (considered representative of the grain roughness of the bed), q is the discharge per foot width in cubic feet per second per foot, and q_s is the sediment load in tons per day per foot width. Data on grain sizes and corresponding fall velocities for the various size classes considered in this study are given in table 1.

TABLE 1.—*Grain sizes and fall velocities*

Grain-size class	Range	Geometric mean		Fall velocity (w) at 65° F.
	Mm.	*Mm.*	*Ft.*	*Ft./sec.*
Very fine sand	0.062–0.125	0.088	0.000285	0.021
Fine sand	.125– .250	.1767	.000580	.068
Medium sand	.250– .500	.3533	.001158	.15
Coarse sand	.500– 1.00	.707	.00232	.26

For use in the channel design studies discussed in this paper, the Laursen sediment load relationship as modified by the Arkansas River data was converted into a more directly usable general relationship between bed material sediment load per foot width (q_s) and the boundary shear (τ_o) or equally applicable product of depth and slope (DS). This relationship is shown on figure 4,*B*, a separate curve being given for each grain-size class range from very fine sand to coarse sand, inclusive. This relationship was developed for each grain-size class if various combinations of depth and velocity are assumed; corresponding values of $(\frac{d}{D})^{7/6}$, $\left(\frac{\tau_o'}{\tau_c}-1\right)$, $\frac{\sqrt{\tau_o/\rho}}{w}$, and τ_o or DS are computed; the corresponding value of c is obtained from the modified Laursen relationship on figure 3; and the corresponding value of q_s is computed. The values of $\sqrt{\tau_o/\rho}$ and τ_o or DS were computed from the assumed depths and velocities by means of the equation given above when τ_o is expressed in terms of Manning's flow formula. With a constant Manning's n, the curves of load versus DS or τ_o shown on figure 4,*B* for each grain size were uniquely defined for all combinations of depth and velocity or equivalent combinations of depth and slope. The sediment load versus DS curves were computed initially by the use of an arbitrarily assumed Manning's n value of 0.023 in the determination of τ_o and DS. Similar sets of curves were computed for various other n values. By comparison of the various sets of curves, a relationship giving the relative variation of load with variation in Manning's n was developed and is shown as a solid line in the graph on figure 4,*A*.

Subsequently, it was found that for the channel studies discussed in this paper the load curves of figure 4,*A* would correlate somewhat better with backwater computations and data from the total discharge versus total bed material load relation of figure 2 if a basic n value of 0.028 were assumed. The value of 0.028 also agreed more closely with n values determined for the Arkansas River data used in modifying the Laursen relationship. An adjusted curve of relation between relative load and n values for a basic n of 0.028 is shown as a dashed line in figure 4,*A*. The curve agrees closely with independently determined load correction factors obtained by correlating average DS and n size for the Arkansas River downstream from values from figure 2. The independently determined load correction factors are plotted for comparison as circled points on figure 4,*A*. In effect, the correction factors serve to establish the position of curves of load per foot width versus DS for any n value parallel to the basic curves for each grain size.

The curves for the various grain-size classes shown on figure 4,*B* were computed in such a way that each curve gives the load per foot width that would result for a streambed composed entirely of each of the individual size classes. In determining the load for a bed composed of a sand mixture, the load value for each size class is weighted by the percentage factor by which that size class is represented in the bed mixture, and the weighted load values for the various sizes are then summed to obtain the total bed material load. A bed material size (d_m) of 0.25 mm. was used as the representative size for grain roughness in the bed in computing the curves of figure 4,*B*. This size was approximately the average median bed material values from backwater computations with load Dardanelle Dam under present conditions. Other sets of curves based on other median bed material sizes can be developed for general use at other locations having different bed composition and for use in such studies as degradation studies, which require consideration of varying bed grain roughness and varying bed

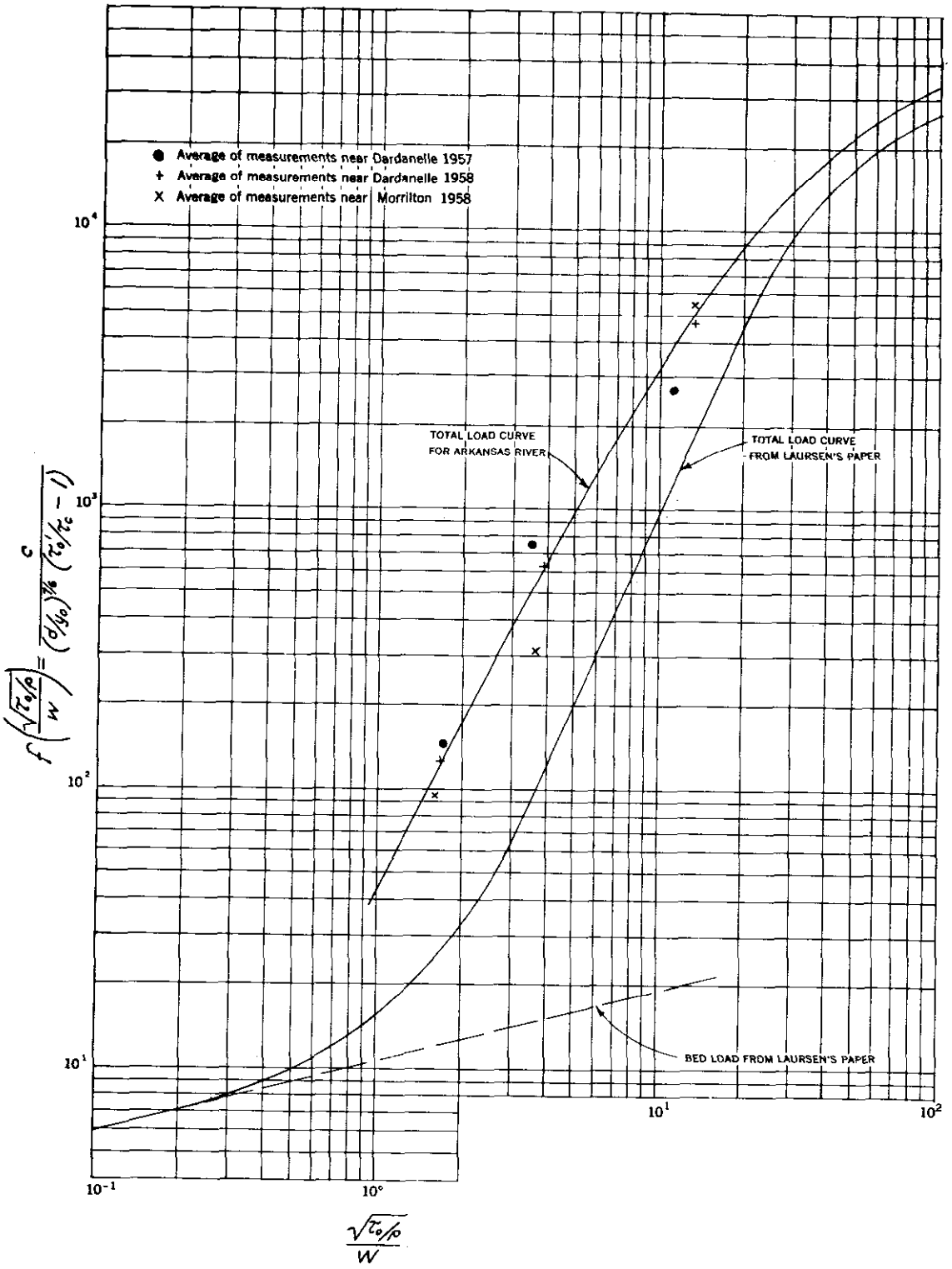


FIGURE 3. — Laursen sediment load function.

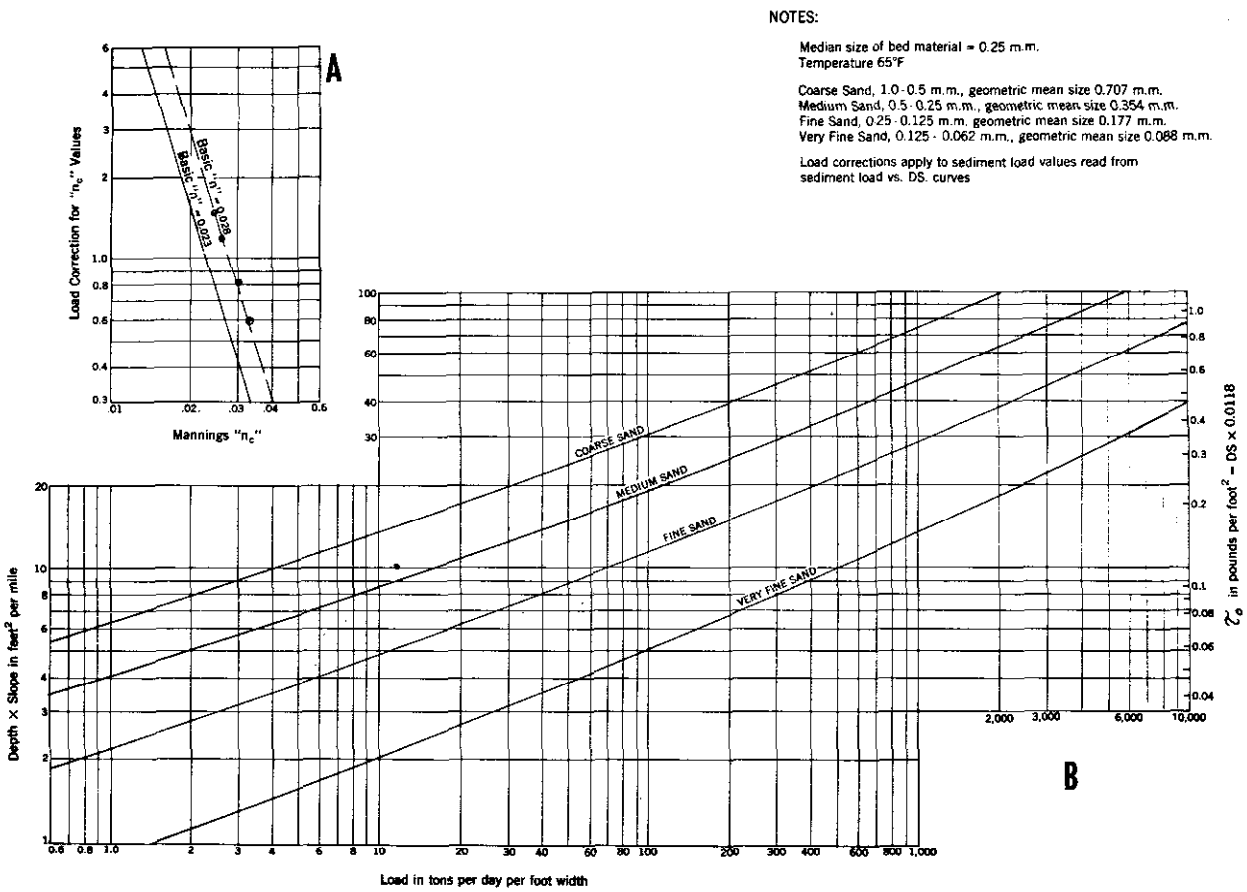


FIGURE 4. — *A*, Load correction factors for variations in Mannings "n" (main river channel); *B*, relation between bed tractive force, τ_o, or product of depth and slope, *DS*, and bed-material sediment load per foot width.

grain size distributions. However, in the computation of the equilibrium bed profile for dredging in the studies covered by this paper, the bed material has been assumed to be constant at the present average composition. In addition, it was found that with the present average bed composition, the fine sand load curve is approximately representative of the total bed material load. The fine sand load curve of figure 4,*B* has, therefore, been adopted as the basic working curve for these studies.

Modified Bed Material Load

The modified sediment outflow from Dardanelle was estimated for a 50-year period as mentioned previously. The computed bed material load outflow ranges from 2.4 million tons per year at start of operation to 1 million tons per year about 12 years after all upstream reservoirs have been in operation. Thereafter it increased again to about 2 million tons per year as the storage capacity in the upstream reservoirs are depleted after 50 years. From a practical standpoint, these changes in sediment outflow during the 50-year period are negligible. Although a greater percentage of the sand load would pass through the spillway as the reservoir capacity was depleted, the inflowing sand to Ozark and Dardanelle Reservoirs will be reduced in a compensating manner with coarsening of the upstream riverbed, which will be the main source of supply. The sediment load outflow for any discharge was available during the 50-year period for use in the channel studies.

In separate studies for estimating the amount and time sequence of degradation downstream from Dardanelle Dam, the increase in bed material load downstream from Dardanelle was estimated by reaches for a distance of about 40 miles, or about halfway to Little Rock, during the 25-year period after impoundment of the reservoir. Of particular interest herein is the modified bed material load at the time of start of navigation. The minimum depths for navigation should prevail at that time because, with the degrading tendency under reduced sediment

load, the bed level should be progressively lowered thereafter with the passage of time. For the purpose of studying the navigation channel, the sediment load 6 years after closure of Dardanelle Dam has been assumed to conform to start of navigation. The total bed material load plotted versus distance downstream from Dardanelle in the sixth year is shown in figure 5 for two discharges for purpose of illustration. The load of sand progressively increases downstream to the vicinity of Little Rock, where it is estimated that essentially complete recovery of the original or natural load will have occurred in the sixth year. This means that at the time, degradation of the channel will be in progress between Dardanelle and Little Rock, but little significant degrading could be occurring downstream from there, since the river would be carrying a capacity load of bed material. The use of the natural and modified sediment load curves in estimating the future channel is described subsequently.

Manning's n Values

The channel studies required separation of the channel and overbank discharges both for purpose of making backwater computations and in estimating the movement of the sediment load. Highwater marks and crest stages have been obtained at various times in the past, which, in conjunction with stages at gaging stations along the river, permitted drawing of water surface profiles for a range in discharges. When backwater for rises were computed near bankfull stages, main channel (n_c) values were determined, and by use of those values the overbank discharges could be computed for rises exceeding bankfull. Work of this nature has been done at various times since 1943. These data were reviewed and additional estimates of main channel n values were made by means of trial backwater computations to check observed profiles based on more recent floods. As few high-water soundings are available, the cross section elements used to compute the n values were based largely on low-water surveys. The channel (n_c) values adopted increased as the discharge decreased from 0.025 for 435,000 c.f.s. to 0.033 at 50,000 c.f.s., as shown on figure 6. Manning's n values determined for use in the studies described herein are somewhat higher than those customarily used for streams of this type, owing to the fact that weighted average or

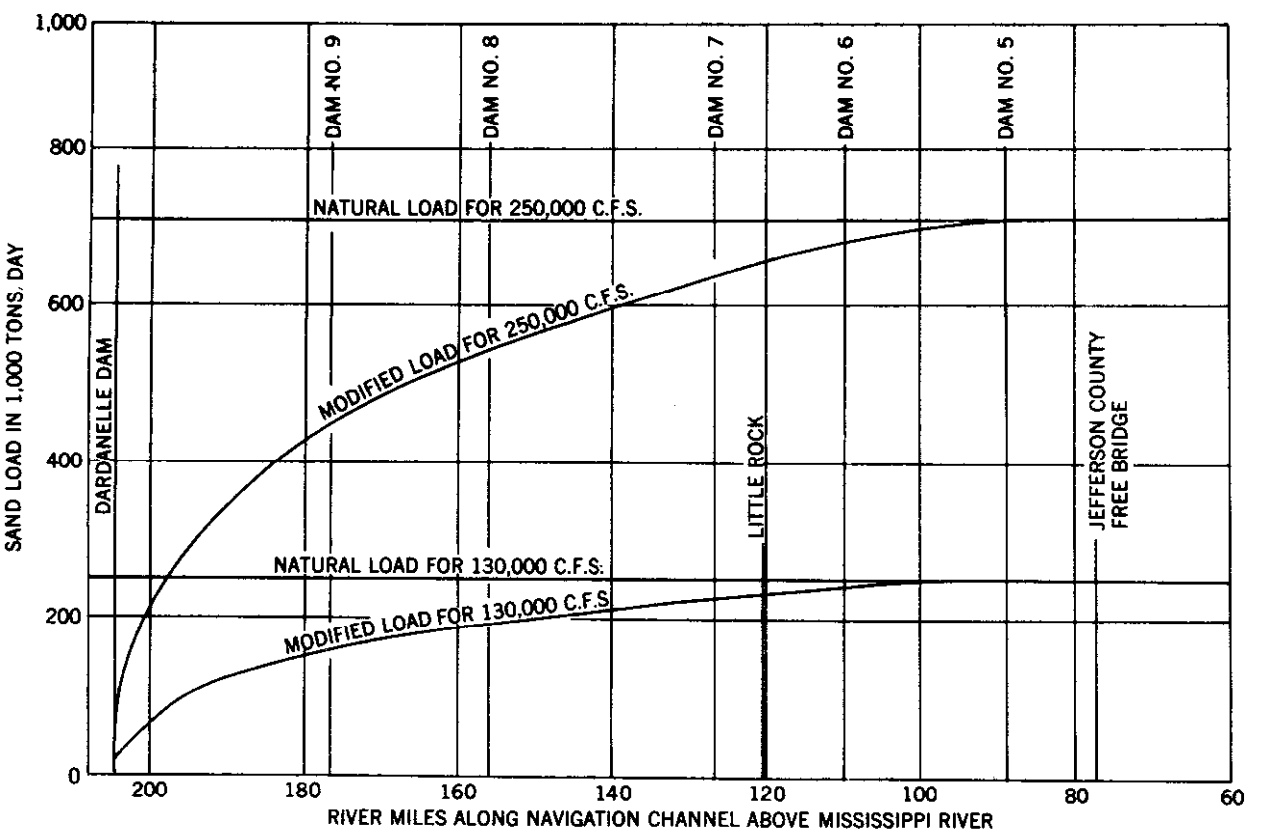


FIGURE 5. — Estimated variation of modified sand load with distance downstream from Dardanelle Dam.

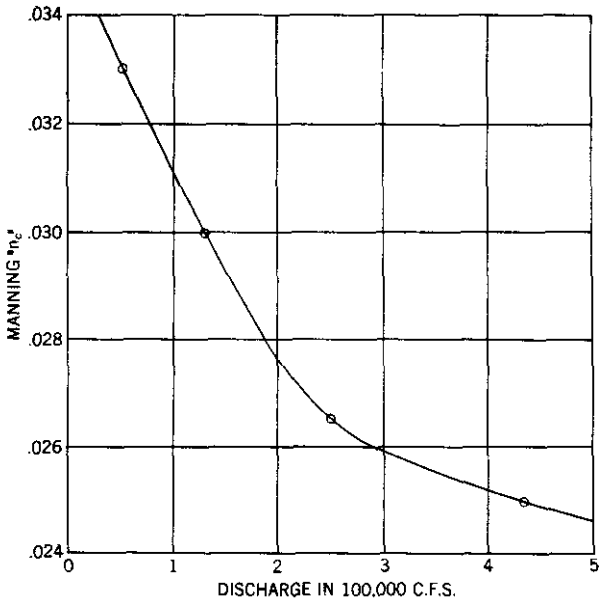


FIGURE 6. — Variation of the main channel Mannings "n" values with total discharge on the Arkansas River in the vicinity of Little Rock.

effective depths and corresponding effective widths have been used in these studies as discussed later. The corresponding cross section conveyance factor $\Sigma\ (AD^{2/3})$ is somewhat larger than the conventional $AD^{2/3}$ values computed from the total area and mean depth or hydraulic radius. Higher n values are naturally associated with larger conveyance factors in Manning's formula.

The overbank was divided into areas that differed in ground cover or land use, and overbank (n_o) values were assigned for these areas by a study of the latest aerial mosaics and published data on roughness coefficients for flood plain areas.[3 4 5] Typical n_o values were in the range of 0.04 for sandbars with scattered low brush, 0.06 for pastureland with low grass and scattered timber, and 0.13 for moderately thick willow growth. A high degree of accuracy in the overbank (n_o) values was not essential since the percentage of total flow carried by the overbank is small. For example, under present conditions between Little Rock and Pine Bluff, 93 percent of a total flow of 250,000 c.f.s. and 89 percent of a total flow of 435,000 c.f.s. are carried by the main channel.

[3] BRADLEY, J. N. HYDRAULICS OF BRIDGE WATERWAYS. U.S. Dept. Commerce, Bur. Pub. Roads, Hydraul. Design Ser. 1, 34 pp. 1960.

[4] RAMSER, C. E., and BARTZ, H. J. THE FLOW OF WATER IN THE MAIN DIVERSION FLOODWAY OF THE LITTLE RIVER DRAINAGE DISTRICT IN SOUTHEAST MISSOURI. U.S. Dept. Agr. Bur. Pub. Roads, Mimeo. Paper. 1924.

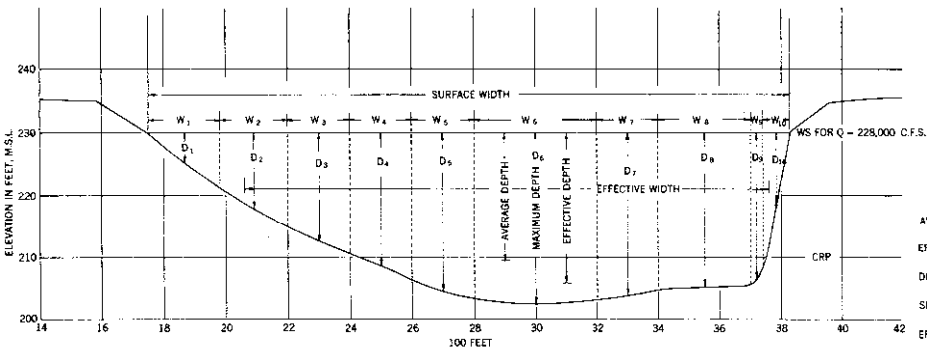
[5] RAMSER, C. E. FLOW OF WATER IN DRAINAGE CHANNELS. U.S. Dept. Agr. Tech. Bul. 129. 1929.

Adjustment of Cross Sections to Stabilized and Realined River Channel Conditions

The ultimate objective of the Arkansas River channel studies was to estimate the cross section elements of the river at the time of start of navigation with the channel realinement of all dams and stabilization works in place. However, it was necessary to proceed a step at a time, starting with the available cross sections of the river. The first step was to adjust tentatively available cross sections of the existing channel for realinement of the river with the normal bank stabilization works in place. The 186-mile reach of the Arkansas River between Dardanelle and Arkansas Post was divided into about 200 subreaches approximately one-half to 2 miles in length, and a typical cross section was developed for each subreach and the conveythese cross sections were of three general types: Crossing sections, sections representative of the upstream part of a bend, and sections representative of the downstream part of a bend. Where recent surveyed ranges were available, an average section for surveyed conditions was developed for each subreach and the conveyance factors of the cross section for a range of flows was computed by $\Sigma\ (AD^{2/3})$ where A is the area and D is the average depth in subsections of the cross section. Figure 7 illustrates how a typical cross section was divided into subsections at the major breakpoints.

These average cross sections for surveyed conditions were then modified to reflect the changes that could be expected as a result of stabilization and rectification work fixing the banks, modifying the radius of curvature of bends, contracting the channel to concentrate the flow in wide and shallow, or braided, sections into a single deep channel, and modifying the slope of the river by the construction of cutoffs. These modifications were based on studies of characteristics of existing stable reaches of the stream, their shape, width, maximum depth, and sediment carrying capacity. Design curves used as a guide for maximum depths in bends as a function of radius of curvature and depths of crossings as a function of width are shown on figure 8. The plane of reference for the depths on this figure is the construction reference plane (C.R.P.) used in the construction of bank stabilization structures. The C.R.P. corresponds to the water surface profile for a flow of approximately 10,000 c.f.s.

As a first approximation it was assumed that the subreaches would have approximately the same conveyance factor for various discharges after stabilization and rectification as at the



SUB-SECTION	WIDTH	DEPTH	AREA	$D^{5/3}$	$AD^{5/3}$	$DAD^{5/3}$
1	230	4.9	1,130	2.88	3,250	15,900
2	220	12.2	2,680	5.30	14,200	173,200
3	200	17.4	3,480	6.72	23,390	407,000
4	200	21.3	4,260	7.68	32,720	696,900
5	200	25.4	5,080	8.64	43,890	1,114,800
6	400	27.6	11,040	9.13	100,800	2,782,100
7	200	26.3	5,260	8.84	46,500	1,223,000
8	300	24.9	7,470	8.53	63,720	1,586,600
9	40	23.6	940	8.23	7,740	182,700
10	90	11.5	1,040	5.10	5,300	61,000
TOTAL	2080		42,380		341,510	8,243,200

AVERAGE DEPTH = $\frac{\text{TOTAL AREA}}{\text{SURFACE WIDTH}} = \frac{42,380}{2,080} = 20.4$ FT.

EFFECTIVE DEPTH = $\frac{\text{TOTAL } DAD^{5/3}}{\text{TOTAL } AD^{5/3}} = \frac{8,243,200}{341,510} = 24.1$ FT.

DEPTH RATIO = $\frac{\text{AVERAGE DEPTH}}{\text{EFFECTIVE DEPTH}} = \frac{20.4}{24.1} = .846$

SHAPE FACTOR = $\frac{\text{AVERAGE DEPTH}}{\text{MAXIMUM DEPTH}} = \frac{20.4}{27.6} = .739$

EFFECTIVE WIDTH = $\frac{\text{TOTAL } AD^{5/3}}{\text{EFFECTIVE } D^{5/3}} = \frac{341,510}{24.1^{5/3}} = \frac{341,510}{201.1} = 1,700$ FT

WIDTH RATIO = $\frac{\text{SURFACE WIDTH}}{\text{EFFECTIVE WIDTH}} = \frac{2,080}{1,700} = 1.224$

FIGURE 7. — Subdivision of a main channel cross section and determination of the "effective mean depth" and "effective width."

present time; therefore, the value of $\Sigma\ (AD^{2/3})$ was kept approximately the same when the cross section was adjusted for improved channel conditions. For example, where the concave bank in a bend was stabilized and an appreciable change was made in the radius of curvature of the stream, the cross section was modified to conform generally to an existing stable section of comparable radius of curvature and the depths were refined so that $\Sigma\ (AD^{2/3})$ remained essentially constant. In subreaches where the channel had changed significantly since the latest survey, hypothetical sections were developed from the latest aerial mosaics, earlier surveys, and the characteristics of similar stable sections.

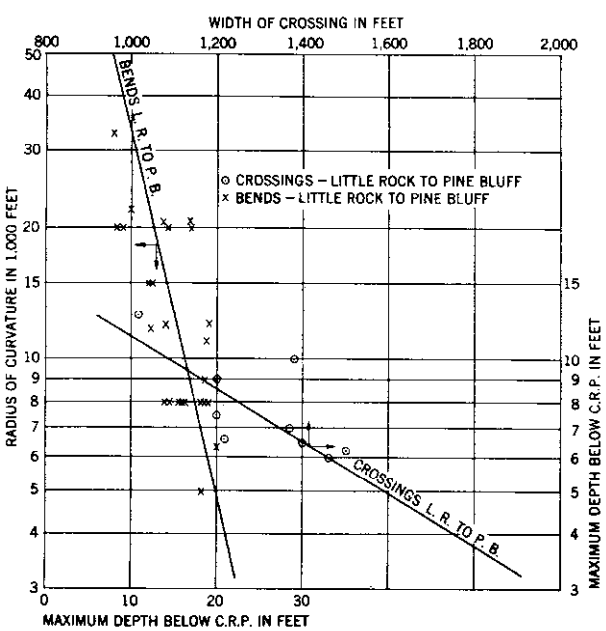


FIGURE 8. — Relations between radius and curvature and maximum depth in bends and between channel width and maximum depth in crossings.

Backwater Computations

Backwater computations were made for a range of discharges up to the maximum modified discharge of record by the use of the tentative cross sections described above. Separate channel and overbank distances were determined as required for reaches represented by the respective cross sections. The Manning formula is applicable in the form shown below. Plots of water surface elevation versus the applicable ratios in the formula were prepared for convenience in making the computations.

$$\text{Fall} = \left[\frac{\frac{Q_t\ L_c^{1/2}}{1.49}}{\frac{A_c D_c^{2/3}}{n_c} + \frac{A_o D_o^{2/3}}{n_o}\left(\frac{L_c}{L_o}\right)^{1/2}} \right]^2$$

Where Q is the discharge, L is the length of reach, A is the cross-sectional area, D is the effective mean depth, and n is the friction factor. The subscript t is total, c is the movable bed part of the cross section, o is the nonmovable bed part.

The primary departure from the usual method of computing backwater is the use of the "effective mean depth" and "effective width." As described subsequently herein, the shape of the cross section of the channel must be defined in terms of the water surface width and mean depth. The channel shape varies widely from near-rectangular on crossings to triangular in sharp bends. Therefore, the depths and widths were weighted according to the concentration of discharge across the section or by the conveyance factor $(AD^{2/3})$ of subareas to obtain effective mean depths and widths. As illustrated in figure 7, each cross section was divided into subsections at major breakpoints in the bottom and the overbank and channel were also separated.

The factor $AD^{2/3}$ was computed for each subsection and multiplied by the depth in the subsection ($D\ AD^{2/3}$). The effective depth is then computed as shown in the tables in figure 7 as:

$$\text{Effective } D = \frac{\Sigma\ (D\ A\ D^{2/3})}{\Sigma\ (A\ D^{2/3})}$$

and the effective width $= \dfrac{\Sigma\ (AD^{2/3})}{D^{5/3}}$

For a perfectly rectangular channel the conventionally determined actual mean depths and actual surface widths are the same as the effective values. The effective mean depth and effective width in effect define an equivalent rectangular section and the nonrectangular sections.

Adjustment of Cross Sections for Continuity of Natural Sediment Load

In an alluvial stream like the Arkansas River, the channel cross section at any location is developed by the flow pattern under the prevailing discharge, the bed material load, and the type of material comprising the bed. Most of the available cross sections were obtained from surveys made at low stages, and the riverbed configuration reflected the accumulated effect of the random flow conditions that existed immediately before the survey was made. Surveys made at the same locations have indicated that the bed elevation may vary as much as 20 feet because of differences in antecedent flow conditions. Also, the riverbed surveyed at a given high stage may range widely, because the discharge and sediment load were not constant for a sufficient period to develop a typical shape for that stage. Therefore, to afford an adequate and systematic basis for study of the channel, it was considered necessary to adjust further the channel depths of the realined channel for uniform flow and constant natural sand load conditions for each of the several discharges ranging up to about the maximum modified flood of record. The method used involved the combination of the basic relation of bed material load per foot width of channel, as described previously, with the Manning flow formula.

The Manning formula was arranged in the following form:

$$D = \left(\frac{5{,}280 \times q_c^2 \times n_c^2}{2.21\ DS}\right)^{3/7}$$

Where D is the effective mean depth of the movable bed part of the main channel, q_c is the discharge per foot of width of main channel, or movable bed channel, S is the hydraulic slope (assumed to be the same or the water surface slope), n_c is the channel roughness factor for the applicable river discharge, and DS is the product of the mean depth and slope. The constant 5,280 is introduced in the equation to permit use of the slope S in feet per mile and DS in feet squared per mile.

An example for adjusting the mean depth at a cross section to equilibrium conditions for a uniform load is shown below.

Given:

Total river discharge, c.f.s.	250,000
Main channel discharge, c.f.s.	241,000
Effective width of main channel, feet	1,385
q_c, c.f.s.	174
Total natural sand load for $n =$ 0.0264 from figure 2, tons/day	710,000
q_s, sand load per foot of main channel width, assuming all sand carried by main channel, tons/day	512
q_s for n_c of $0.028 = 512 \times 0.865 =$ $512 \times 1/1.157$, where the load correction factor 1.157 is obtained from the circled point for $n =$ 0.0264 on figure 4,*A*	443
DS from fine sand curve on figure 4,*B*, sq. ft./mile.	20.5

Effective mean depth adjusted,

$$D = \left(\frac{5{,}280 \times (174)^2 \times (0.0264)^2}{2.21 \times 20.5}\right)^{3/7} = 28.4 \text{ feet}$$

The step for computing q_s for $n_c = 0.028$ outlined above is equivalent to and substitutes for making use of a separate load curve for an n value of 0.0264 to determine DS. As discussed previously, the separate load curve, if shown, would be parallel to the basic fine sand curve for $n = 0.028$ on figure 4,*A*, and its position is defined relative to the basic curve by the load correction factor 1.157 for an n value of 0.0264. In effect, the basic curve is used only to establish the slope of the functional relationship between load per foot width and DS.

If the main channel is contracted, as with dikes, the sand load per foot of width is increased, and from figure 4,*B* it results in an increase in DS, which appears in the denominator of the equation. However, the discharge per foot of width of the movable bed (q), which appears in the numerator, will increase more, so that the depth is always increased for a contraction of the channel. Furthermore, a reduction in sand load per foot width results in a reduction in DS (fig. 4,*B*), which also results in an increase in the depth. Attention is invited to the assumption that all the sand load is carried by the main channel. Although the finer silt and clay particles are uniformly dispersed, the heavier sand load is largely concentrated in the deeper channel. As stated previously, about 90 percent of all the flow is carried by the main channel. Computations for a typical cross section indicate that more than

99½ percent of the sand load is carried in the main channel.

The effective mean depth adjusted for uniform flow of water and sand was determined for each cross section used in the backwater computations. However, to compute the bed elevation, an actual rather than effective depth must be subtracted from the water surface. The actual mean channel depth was known for the basic plotted cross sections by dividing the cross-sectional area by the surface width. The effective mean depths were also computed for the basic cross section. The ratio of the actual and effective mean depths determined from the basic cross sections times the *adjusted* effective mean depths gave the adjusted actual mean depths.

Subtraction of the actual mean depth from the water surface profiles obtained by the backwater computations would afford a separate mean bed profile for each of the discharges. However, the bed elevation of the deeper part of the river comprising the navigation channel is needed rather than the mean bed level across the entire main channel. The highest bed elevation in the lowest 250 feet width of channel was adopted for measuring navigable depths, since this is about the minimum width for navigation. As the shape of the cross section depends on the direction of flow and the flow pattern, the depth in the 250-foot width cannot be computed analytically. Therefore, empirical relationships were developed between mean depth and the depth in the deeper 250-foot width, based on the surveyed cross sections, as shown on figure 9. Figure 9, left, shows the mean depth versus the depth for 250 feet with channel width as parameter for a near-rectangular-shaped cross section. The actual adjusted mean depth and the actual adjusted surface width were used to enter the figure. The correction factors for shapes other than rectangular ones were obtained from figure 9, right.

With the final actual depths for the 250-foot width of realined channel under uniform natural sediment load condition obtained, those depths were subtracted from the water surface profiles estimated by backwater computations to obtain the bed elevations. The water surface profiles and corresponding bed elevations for 50,000 and 250,000 c.f.s. are shown for illustration for pool No. 7 on figure 10. The effect of discharge on the bed profile depends on the shape of the cross section. In general, the larger discharge tended to lower the computed bed profile in bendways or where the overbank flow is relatively small and to raise the bed level on crossings, which is in accordance with observations of the Arkansas River and other alluvial streams.

Water Surface Slope

In the above solution no changes or corrections were made in the water surface slopes or elevations from those computed in the backwater computations. However, for continuity of sediment load with the depth (D) computed

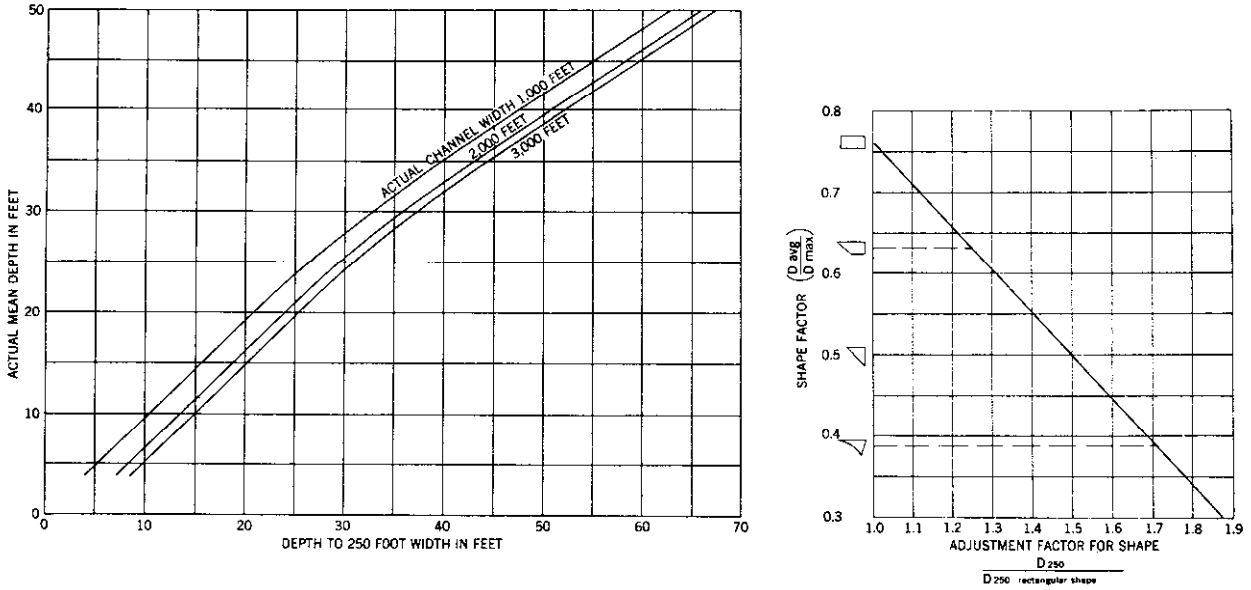


FIGURE 9.—Left, relation between mean channel depth, width, and depth to the highest point in the lowest 250-foot width of near-rectangular cross sections; right, adjustment factors for determining depth to the lowest 250-foot width for cross sections of shapes other than near-rectangular.

and DS known, a new slope can be obtained as:

$$\text{Slope} = \frac{DS}{D}$$

If an increase in depth is determined, the slope will always be reduced, provided the channel friction factor n_c remains unchanged, and a correction in slope is then necessary that will in turn change the water surface elevations from those obtained by the backwater computations. The bed elevations are shown on figure 10. However, the channel n_c values were assumed to increase over those to date, because of the extensive revetment and dike system provided under the normal bank stabilization plan.

Estimates made indicated that an increase in n_c value of about 5 percent would compensate for the computed flattening of the slope and raise the water surface profiles back up to those determined by backwater computations. The channel model study at the Waterways Experiment Station, as subsequently described, indicated an increase in n_c values and an increase in slope for high flows as a result of channel contraction. Therefore, no correction was made in the slopes where contraction works alone were involved in channel deepening as is the case for the data described above. Note that an assumption of flatter slopes and the related lower water surface profiles would have resulted in lower bed elevations than shown, which would not have been conservative with respect to establishing the normal pool elevations for navigation. However, the falls through the reaches were computed by the ratio DS/D for use in a subsequent step.

Adjustment of Cross Section Elements for Modified Sediment Load with Dams in Place

With the channel elements determined for natural load conditions for the improved channel as described above, the next step was to make further channel adjustments to obtain the controlling riverbed profiles under modified sediment load conditions at the time of start of navigation (6 years after closure of Dardanelle) and with the navigation locks and dams in place. As described previously, the load would be reduced downstream to the vicinity of Little Rock. First, locations for the dams were assumed, then the new equilibrium bed elevation was computed in the 250-foot width of channel

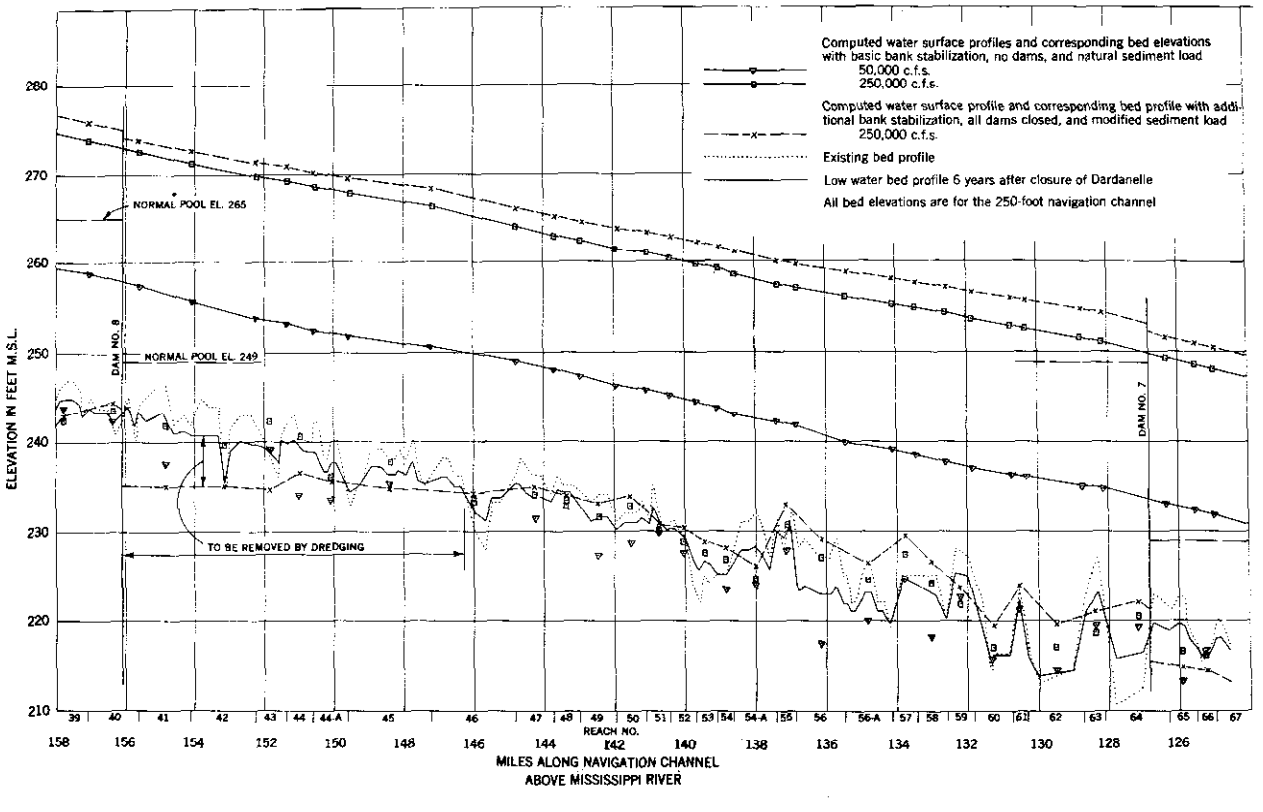


FIGURE 10. — Streambed and water surface profiles in Arkansas River Pool No. 7 for basic conditions and computed modified regime conditions.

under the modified sediment loads at time of start of navigation. The basic method used was the same as described above. A variation from the previous computations was the adjustment of the fall through each reach for the flattening of the water surface slope, owing to reduced sediment load. This was done by computing the fall through each subreach by the equation $S = \frac{DS}{D}$, using the DS value corresponding to the reduced load, subtracting this value from the fall computed from the same formula as described in the preceding paragraph, and subtracting this difference from the fall determined by backwater computations for the basic cross sections. This gave the adjusted fall in each reach for the reduced sediment load and the accumulation of falls gave the adjusted water surface profile. With the new water surface known and the depth computed, the bottom elevation under modified sediment load for 250,000 c.f.s. was computed.

While the spillway gates would be fully open for the discharge of 250,000 c.f.s., the constriction caused by the piers, lock, and embankments would create a swellhead at each dam. The higher elevations as indicated on figure 10 with the dams in place are caused by this swellhead.

Adjustment of Cross Section Elements for Special Contraction Dikes

As mentioned previously, in addition to the regular bank stabilization and realinement works, special contraction dikes are planned for several miles downstream from each dam to increase the concentration of flow and cause deepening of the channel. The special dikes are similar to the regular dikes except that on crossings a trace width as small as 1,000 feet was provided, as compared to about 1,400 feet for the regular training dikes.

The last step in the computations was to determine the depths and riverbed elevations for the additional contraction of the main channel. With the reduced widths selected, the resulting increased depths and lower bed elevations were computed by the procedure used before. As before, no flattening of the slope was used in those computations where channel deepening is caused by contraction alone. The depths were subtracted from the water surface determined previously for the reduced sediment load conditions. It is apparent that the effect of the reduced sediment load on channel depths and water surface slopes had to be computed before the effect of the additional channel contraction could be determined, rather than combining the computations into a single step. The final equilibrium bed elevation is shown as a dashed line on figure 10.

As previously indicated, the equilibrium bed is the nonscouring or filling configuration created by a certain flow pattern or direction of flow under a specified discharge, load, and bed material size. As the equilibrium bed varies with discharge, computations were required for several discharges to determine which created the highest or controlling bed elevation at the respective cross sections. The equilibrium bed also depends on the bed material sizes, whereas the computations do not provide for any change in the material forming the bed. Any natural degradation would tend to coarsen the bed material. However, since it is planned to dredge in the head of each pool, thus removing part of the bed surface, the bed material at the new bed elevation under equilibrium status would not depart greatly from that for the original cross sections. The effect of the coarsening of the bed where no dredging was performed was taken into account in computing the reduced bed material load, as previously described.

Dredging

Dredging in the head of pools where depths would be critical for navigation is required to attain the equilibrium bed at time of start of navigation, because of the slow natural degradation process and also as armouring by the coarser material would theoretically prevent ever fully reaching the prescribed depth by natural scour. A dredge can operate more efficiently by excavating a deeper and narrower cut than would normally conform to the relatively shallow depth of material above the equilibrium bed level. Therefore, the proposed dredging would remove in a relatively narrow, deep cut, a volume of material equivalent to that between the cross sections of the bed at time of start of navigation and the equilibrium bed cross section, so that the river could readily mold it to the estimated equilibrium shape. A typical dredge cut is indicated on the cross section on figure 11. The bed profile at time of start of navigation without dredging and the final equilibrium bed profile are shown on figure 10, the difference in the profiles indicating the material to be dredged as noted. Ample space for spoiling of the dredged material is available in dike systems and on the relatively high convex bars within the river channel. Placement of the spoil in this manner will provide further contraction to the main channel flow and thus further encourage deepening of the channel by scour of the bed. Since dredging would be undertaken during low water the bed profile at time of start of navigation is shown for a small discharge or low water, and since the crossings that are the shallowest part of the river would tend to be highest during high discharges, the equilibrium bed profile is com-

puted for a high discharge. For purposes of illustration, the equilibrium bed is shown for the discharge of 250,000 c.f.s.

Regulation of the Pools

Another factor related to computing the riverbed level is the plan of regulation of the pools and the backwater created by the locks and dams. No detailed plan of regulation has been studied for each of the pools, but most of the bed material load will be carried by the higher flows when all spillway gates will be fully open, resulting in essentially open river conditions. However, the locks and dams will cause some backwater even with all gates open and a higher computed water surface will tend to raise the bed level correspondingly, since the computed depths are subtracted from the water surface to obtain the bed elevations. In other words, backwater would eventually cause sufficient deposition to attain a higher equilibrium bed than otherwise and the mechanics of the method followed simulated that situation. The effect of backwater at the dams is shown in the example on figure 10.

When pooled stages prevail in conjunction with the smaller discharges there will be a tendency for deposition to occur, but not only is the natural bed material carried at such times small, but the supply from upstream will be cut off at Dardanelle and the relatively slackwater condition in the navigation pools will reduce the tractive force on the bed so that only a relatively small load can be initially entrained from the riverbed to create a deposition problem farther downstream. Therefore, the smaller flows under pooled conditions are expected to have little influence in forming the riverbed.

Channel Model Tests at Waterways Experiment Station

A generalized model study was conducted at the Waterways Experiment Station of a 10-mile reach of the Arkansas River between Little Rock and Pine Bluff to investigate the effects of various types of control works on channel stability and sediment transport with normal sediment load. The model was constructed to a horizontal scale of 1:150 and a vertical scale of 1:36, resulting in a geometric distortion of 4.17. To provide satisfactory sediment transport in the model, a supplementary slope of 0.27 ft. per mile was used, giving a slope scale of 5.7:1. The overbank areas were molded in concrete, and the movable bed portion was originally molded to October 1957 prototype conditions, using sand with a median diameter of 0.2 mm. The time scale was such that a prototype day was reproduced in 10 minutes in the model; the rate of introducing bed material and the discharge scale were varied with stage.

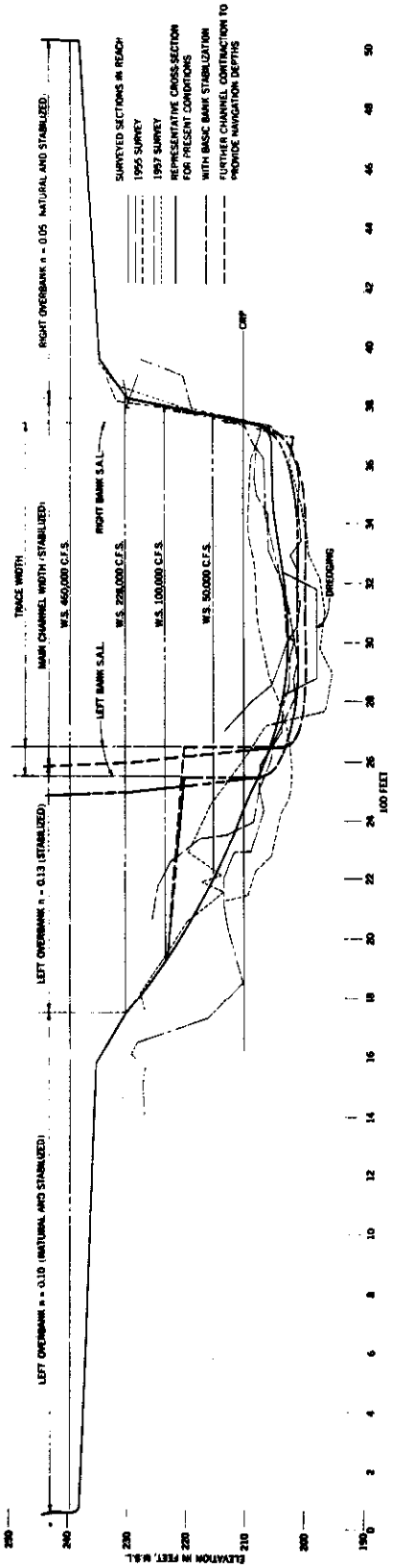


FIGURE 11. — Typical cross section illustrating comparison between present conditions and conditions with channel contraction.

After adjustment of the model, a base test was made with the channel rectified and stabilized according to the normal bank stabilization plan for the river; that is, the concave banks were revetted and the crossings contracted to 1,200 or 1,400 feet. The efficacy of plans of supplemental improvement were then compared to conditions in this base test. Several runs were made for each plan in an attempt to obtain equilibrium sediment load conditions, each run being one reproduction of the experienced 1957 discharge hydrograph.

Because of time limitations most plans were not tested to stability, and the effects of the various features tested were not fully developed. Certain arbitrary assumptions were also made, such as the use of a specific discharge hydrograph for testing and the lower tailwater control to reflect the effect of a dredged channel used in some runs. However the results do indicate that by proper local contraction additional depths can be attained, and the tests provide general indications of the type of effects produced by the various regulative measures investigated as follows:

(1) Installation of a large dredge cut without an appreciable reduction in sediment load and construction of control structures would not be effective, as the dredged cut would soon refill.

(2) Regulating structures, which increase the radius of curvature of a bend, provide a more uniform channel cross section and improve the alinement of the channel over the crossings.

(3) In bends, longitudinal dikes provide a more efficient channel than spur dikes, improve flow conditions and channel cross section, and reduce the amount and extent of shoaling in the crossings.

(4) Extending training works on a concave bank downstream into the crossing improves the alinement of the channel through the crossing, reduces shoaling in the crossing, and improves flow conditions in the next bend downstream by reducing the tendency for a point bar to develop there.

(5) Dredged spoil and willow growth on convex bars improve the alinement, depth, and shape of channel through the bend. Such spoil banks require only minor protection except at the upstream end.

(6) Contracting the channel to 1,000 feet through the crossings increases navigation depths and improves channel alinement. This degree of contraction and the placing of dredged spoil and willow growth on convex bars simultaneously increased water surface slopes at high flows significantly.

(7) Backwater computation for model test conditions indicated that channel n values were greater in reaches with spur dikes than in uncontrolled or revetted reaches, but that with longitudinal structures along the channelward ends of spur dikes the effect of the dikes on the channel roughness would be minimized. Backwater computations indicated that constricting the crossings to 1,000 feet with channel-line structures in bends and simulating accretion and willow growth on convex bars increased n values to approximately those which obtained for spur dikes. Low-level longitudinal groins did not produce any discernible increase in n value.

(8) Spur levees were tested along with other factors, so their effect was not isolated. However, they would be effective only during infrequent extremely high flows and would not supplement other structures in maintaining a navigable channel during normal flow periods.

(9) Low-level spur groins in crossings to provide additional contraction to 600 feet at low flows increased the depths over the crossings, but as some of them were transverse to the direction of flow they produced considerable backwater and undesirable shoaling upstream.

(10) Low-level longitudinal groins in crossings to provide additional contraction to 600 feet at low flows were effective in fixing the location and alinement of the crossings and produced some lowering of the controlling elevations over the crossings. As they were parallel to the flow, they offered practically no resistance.

Complete details of this model study are given in U.S. Army Engineer Waterways Experiment Station Technical Report 2–608, August 1962, entitled "Development and Maintenance of Navigation Channel, Arkansas River, Arkansas and Oklahoma."

A similar model study of the reach of river in the vicinity of a specific damsite on the lower Arkansas River is currently underway at the Waterways Experiment Station in Vicksburg, Miss., using a movable bed of granulated coal. Results of this study to date confirm the feasibility of developing and maintaining a deepend channel by means of a reasonable amount of contraction by dikes and revetments.

Flume Tests on the Armouring Effect of Gravel at St. Anthony Falls Hydraulic Laboratory

As mentioned previously, when the bed material load now carried by the river is reduced by the storage projects, the degradation process will be retarded by formation of an armouring layer of the coarser sized material forming the bed. This not only affects the riverbed elevation resulting from natural scour, but also is a

primary factor in the modified bed material load that is used to compute the equilibrium riverbed level as previously described. While an effort was made to account for the armouring effect of gravel in determining the loads, there is no generally accepted engineering analysis for the problem. Therefore, early in 1959, controlled flume tests were undertaken at the St. Anthony Falls Hydraulic Laboratory.

A sand-gravel mixture comparable to that on the lower Arkansas River was used in the flume studies. All tests were made by introducing clear water into the flume, or for zero sediment inflow. The studies were of two types: Small-scale exploratory tests and full-scale tests. In the small-scale tests observations were made of the degradation process on a bed initially composed of a homogeneous sand-gravel mixture. For each run the initial tractive force was greater than critical for all material in the bed, and as degradation proceeded, the slope flattened and the tractive force decreased to that of the critical value for the top layer of material that had accumulated on the bed when stability was reached. The initial tractive force was increased for each succeeding run, and the armour layer, approximately 1 grain-diameter thick, progressively coarsened.

In the full-scale tests an 18-inch sand bed was covered with an armour layer of 20-mm. gravel 3-grain-diameter thick, and the tests were largely devoted to observations of the natural development and healing of a scour hole in the bed, of phenomena at flow having a tractive force about critical for the armour material, and of the effect of an obstruction at the boundary on the gravel layer. The tests indicated that if excessive tractive force is localized and the mean value is less than critical for the armour, a scour hole will stabilize and armour itself; that if the mean tractive force is about critical for the armour material some of the fines will be leached out as the gravel is transported, the bed will be lowered, but the gravel blanket will remain essentially intact; and that if the tractive force is appreciably above critical for the armour, the gravel blanket will be disrupted and destroyed.

Results of Channel Studies

The estimated bed profiles for the 250-foot minimum width of navigation channel between Arkansas Post and Dardanella Dam under present conditions and for the computed equilibrium bed conditions with all works in place for the adopted plan of development are shown on figure 12. It will be noted that the equilibrium bed provides a minimum depth of 12 feet below the normal pools. The 8 low-lift locks and dams shown in the reach of the river between Dardanelle Dam and the vicinity of Arkansas Post compare with an earlier plan considered which included 13 low-lift locks and dams in the same reach with locations and pool elevations matched to the natural profile of the river. Thus, taking advantage of the degrading tendencies of the river after construction of the large dams made it possible to eliminate five locks and dams in this reach of river. Similarly, two locks and dams are being eliminated in the reach between Short Mountain Dam and the head of Ozark Reservoir. By a relocation of Ozark Dam site in conjunction with dredging and contraction, it now appears possible to eliminate a low-lift lock and dam previously considered between Ozark Dam and the head of Dardanelle Reservoir. Thus, a total of 8 low-lift navigation locks and dams out of a previous total of 18 are being eliminated by taking advantage of degradation with dredging, contraction works, and adjustment of damsites. As the cost of the dredging and contraction work is far less than the cost of locks and dams, a considerable overall reduction in project costs will be realized with added benefits through an increase in head for hydroelectric power generation at Dardanelle, Ozark, and Short Mountain Dams.

It is to be expected that a considerable amount of maintenance dredging will be required in the early years of the project, because of local shifting of material into the dredged channel from bars and from the undredged part of the main river channel. However, the maintenance dredging can be kept to a minimum and dependable navigation can be assured by dredging the initial channel to sufficient depth and width to provide space for deposition of the shifting bed material and by use of advance maintenance dredging. The greater is the extent of initial dredging, the less is the amount of material available for shoaling. As indicated before, navigable depths will improve and maintenance dredging requirements will diminish as time goes on as the effects of reduction in load progress downstream, as degradation continues, as a stabilizing armouring of the bed with coarser material develops, and as the supply of material in the streambed diminishes.

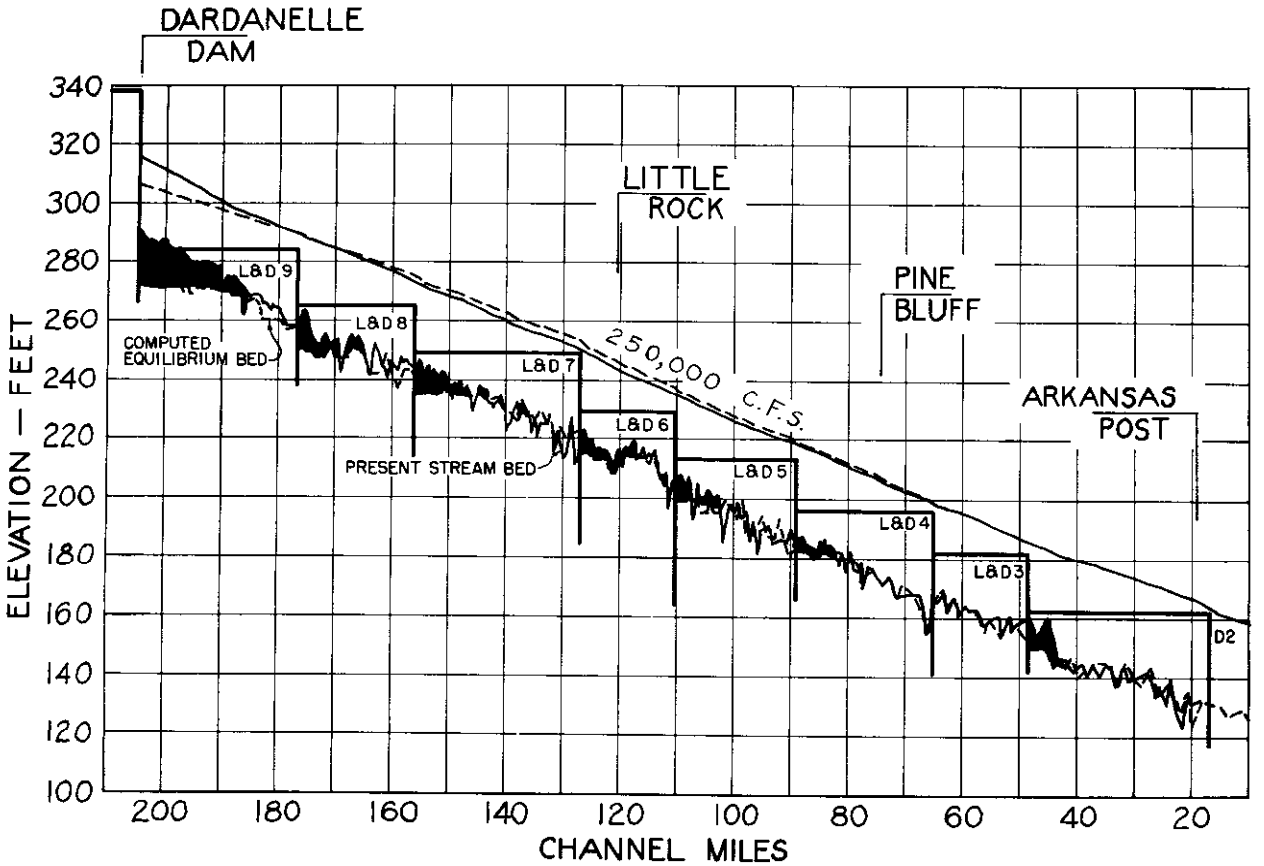


FIGURE 12. — Profiles for existing conditions and computed modified regime conditions between Arkansas Post and Dardanelle Dam.

CHANNEL RECTIFICATION STRUCTURES

[Paper No. 40]

By D. C. BONDURANT, *engineer, U.S. Army Engineer Division, Missouri River, Omaha*

Abstract

Channel rectification structures are utilized to prevent erosion of the banks, guide the flow along a desired alinement, restrict the flow to an effective waterway, or various combinations thereof. In most instances, these structures involve solid or permeable revetments or guide structures of varied types of materials and construction. There are few, if any, rational methods for design of such projects, and it is usually necessary to plan the layout wholly on the basis of experience and judgment. There is no attempt herein to present design criteria; rather, the areas of influence of various components are noted, the deficiencies in knowledge are pointed out, and the desirability of a comprehensive investigation to develop rational design is suggested.

Rectification Structures

Structures for channel rectification have been constructed of numerous types of materials and in a number of different ways. In general, each of these types of materials, varying from broken concrete and other trash — old auto bodies, rock, woven mattresses, piling or varied structural units — to solid paving or solid guide walls, should be adequate if properly applied at the right places. However, if improperly applied, they may be of no benefit or may even increase the damage they are meant to prevent. On the other hand, if applied properly but in excessive quantity or in locations where not actually needed, they result in an excessive waste of construction funds.

Rectification structures may be normally classified as either revetment or guide structures, although there may be a considerable overlap in their functions. Structures utilized

for stabilization of the bed of a channel will not be considered herein.

Revetments

Revetments may be placed either as an armoured blanket on a channel bank or as a guide structure to provide a smooth alinement and induce sedimentation to fill the bank line. The latter case will be considered with guide structures.

The purpose of a blanket revetment is to protect a channel bank against erosion. For this purpose, it should ideally possess the following characteristics:

1. It should be placed on a bank that has been properly alined and graded to a stable slope.
2. It should have adequate strength to resist movement by the flow or breakage by ice or debris.
3. It should be sufficiently tight to prevent leaching of the underlying soil materials.
4. It should be sufficiently flexible during construction to conform to any irregularities in the bank.
5. It should be mobile, in that it should settle into any areas of breakage or subsidence in a manner to be self-healing.
6. It should be permanent.
7. It should be susceptible to placement by reasonable methods.
8. Under certain circumstances, a hydraulically rough surface is preferable to one that is smooth.

The criterion for grading and alinement is most essential for larger streams, particularly those with easily erodible banks, but is beneficial in any case. The grading primarily provides a slope that can be more readily stabilized. A proper alinement eliminates eddies and deflections of the current. On smaller streams grading and alinement may not be economically feasible, and, if the banks are reasonably resistant, may not be necessary. The planner who will probably have noted instances of ungraded and unalined banks being adequately stabilized by trash dumps will have no guide lines except his own opinion of the resistant character of the banks.

Adequate strength is required in all cases. In the case of rock or similar materials, transport formulas, adjusted for bank slope factor, can be utilized to determine the size of the material required. A further adjustment might be used to reflect the keying action inherent in a well graded mixture of material sizes; however, there are no criteria available, and the uncertainties involved in transport computations make such adjustment inadvisable. Adequate strength of other materials, in place as revetment, is almost wholly a matter of judgment.

Leaching of the soils through the interstices of a revetment is a very common cause of failure, particularly if these soils are primarily sandy. Once a pocket has progressed to the extent that a local failure of the blanket occurs, the failure is apt to spread rapidly. Leaching can be inhibited or prevented by a tight blanket, a filter layer, or by a well graded mixture of rocks such that all voids are filled.

Most materials utilized for a blanket revetment permit sufficient flexibility in placement to conform to bank irregularities; however, any failure to do so will almost certainly result in ultimate failure of the blanket.

The question of mobility to the extent that a blanket will be self-healing will, if adhered to rigidly, eliminate many materials that might actually be adequate under the proper circumstances. As a general rule, however, it is an important criterion. Continuous pavement of concrete or asphaltic materals, for example, might be the most satisfactory armor under conditions where damage by ice or debris is not a factor, but once a break does occur at a location exposed to high velocity flow the armor is more apt than not to peel off in large strips. Hand-placed riprap, in addition to requiring expensive hand labor, tends to bridge over a failure until it becomes too extensive to be readily repaired. The most satisfactory material in this respect is dumped rock or even miscellaneous trash.

Permanence is not a problem with rock, broken concrete, and similar materials. Lumber and brush are relatively permanent where continuously submerged, but deteriorate rapidly where subjected alternately to wetting and drying. In locations where vegetation can be established once the bank is stabilized, the vegetation may adequately replace the deteriorated part of the revetment.

A reasonable method of placement would be defined largely in terms of the magnitude of the project.

The benefits of a hydraulically rough surface are not universal, and, in any event, are of value only within certain limits. They derive from the difference in velocity distribution as between rough and smooth boundaries; i.e., velocities immediately adjacent to the rough boundary will be less than those adjacent to the smooth boundary. In the Missouri River, for example, the high velocity flow normally occurring along the concave bank of a bend moves about 50 feet riverward after rock revetment is placed. As a concurrent feature, the deepest scour in the section occurs riverward of the toe of the revetment rather than at the toe. There are numerous examples of depths in bends of 15 to 20 feet below the toe elevation

of the revetment and several examples where this excess is greater than 30 feet. In none of these locations, however, has there been any undercutting or failure of the revetment.

An approximation of the range in which this effect would be of value can be made by computations of the two-dimensional velocity profile, but, since the rough boundary is generally sloping, the three-dimensional effect is more complicated. As the horizontal effect on the Missouri River is about 50 feet (with the revetment toe normally about 9 feet below the low water profile), it is probable that the benefits of the rough blanket surface would be limited to cases where the toe depth is not more than 15 feet below the low water profile. The greater turbulence generated at the rough surface increases the tendency for leaching; thus, a tight blanket becomes more important.

There are no universal design rules for blanket revetment. Normally the revetment should extend to the top of a low bank or to a level 8 or 10 feet above the low water profile; however, this latter value would vary with the size of the stream. Woven or other mattress is usually extended to the thalweg of the stream, but other types of revetment are usually stopped at the intersection of the bank with the streambed. If the project is of sufficient magnitude, it is advisable to either excavate a trench to extend the mattress several feet below the normal bed or to add excess toe material that can settle to heal undercutting. In locations where the flow hits the bank at an angle greater than about 30°, a mattress extending to the thalweg is advisable, regardless of the type of upper bank blanket.

Permeable Training Structures

Permeable revetments and permeable training structures are similar in purpose and construction, their primary function being to guide the flow to a desired alinement and, concurrently, to cause sedimentation in a desired location landward of the structure. They have been constructed of piling, fencing, steel or timber cross frames (jacks), brush, and other available material. There are two schools of thought insofar as the design principles of such structures are concerned: (1) that the units are considered as roughness elements that cause a reduction of the velocity of the flow through the structure and (2) that the structure should primarily guide the flow while concurrently permitting a part of the flow to pass through into a relatively quiescent area to deposit its sediment load. Both are correct, for the two functions are interrelated, and the one that predominates is a matter of the overall layout.

The most rational analysis of the action of a permeable structure is, perhaps, that presented by Einstein[1]—"When the division takes place between the water masses which move along the jacks and those which pass through the line of jacks, all the water has entered a local zone near the structures of slightly higher pressure. In entering this zone, all water particles lose, according to momentum theory, a given constant part of their velocity." Einstein computed that if the jacks were fully effective it would require 140 jacks (composed of 18 foot timbers) to turn a flow of 50,000 c.f.s., flowing at a velocity of 5.7 feet per second, around a 90° bend. He estimated, however, that since the jacks are only partially effective due to their splitting action, the actual requirement would be in excess of 500 units. He concluded that secondary effects, particularly those of the sand deposits behind the jacks, were a major part of their function.

For a different attack, Carlson and Enger[2] measured the head loss in a model. Actually, it can be observed in the field that a permeable structure which is essentially parallel to the flow acts as a permeable rough boundary and that as the angle to the flow increases, this effect decreases and the structure acts more as a roughness element in the line of the flow. In either event, there appears to be a pressure gradient as discussed by Einstein.

The preceding discussion cannot be said to solve anything in particular except to point out the fact that the designer knows very little about the tool with which he is working. The lack of knowledge may well result in either an unsuccessful project or one in which a necessarily conservative approach generates excessive costs. The average project utilizing permeable structures will include guide lines to assist in turning the flow and to cause a reduction in the velocity of the flow passing through it. Between this frontal line and the existing bank there will be tielines, either solid or permeable, which will be expected to reduce further the velocity of the flow in the sedimentation area, to prevent the buildup of a secondary flow system, and to return flow from the sedimentation area to the channel. In projects that have continued over a period, experience will have developed criteria for the optimum angles of the guide lines, the size and spacing of the units therein, and the number and locations of the tielines required, but this experience will not be too helpful if transferred to a system on an entirely different scale.

[1] EINSTEIN, H. A. REPORT ON THE INVESTIGATION OF THE FUNDAMENTALS OF THE ACTION OF RIVER TRAINING STRUCTURES. Calif. Univ. [Rpt. not published.] 1950.

[2] CARLSON, E. J., and ENGER, P. F. USE OF STEEL JETTIES FOR BANK PROTECTION AND CHANNELIZATION IN RIVERS. Presented at Amer. Soc. Civil Engin., Hydraul. Div. Mtg., August 1956, Madison, Wis.

Groins

Groins are generally structures placed normal to the current or at moderate upstream or downstream angles thereto, and may be likened to some extent to the tiebacks in a permeable system without the frontal lines. As such, they are not considered to be desirable if constructed as short segments along a straight or concave bank, but there have been some recent indications that a series of groins, or tieback, structures with a downstream extension (L-head) may permit deletion of a part of the frontal line. In other words, instead of being continuous the frontal line might extend only one-half or two-thirds of the distance from one tieline to the other.

Any structure extending into and normal to the flow will have several generally undesirable effects. The primary such effect is the formation of a relatively deep scour hole immediately at and downstream from the outer end of the structure. A second such effect is the ever-present danger of destruction of the root end of the structure, and a third is the fact that a system of groins along a straight or concave bank can increase the overall roughness coefficient of the reach by as much as 25 percent. When a downstream extension is placed at the outer end of the structure, however, the structure begins to approach the characteristics of a frontal system with tiebacks. Some field trials under way indicate that, if the downstream leg extends to one-half to two-thirds the distance to the next structure and if the structures are properly placed in a straight reach or flat bend, both the deep scour and the excessive roughness will be partially or perhaps wholly eliminated. This means only that adequate guidance might be provided by a discontinuous rather than a continuous frontal line; however, the evidence to date is inadequate to provide any rational proportions or justify use except on an experimental basis. There is little doubt that even a short L-head improves the action of a groin, as such.

Research Needs

An observer noting numerous examples of successful revetment and training structures might have reason to consider the foregoing discussion unnecessarily pessimistic. He might also recognize that the ultimate success of a permeable system is dependent upon auxiliary factors such as the effect of the sediment deposit itself and the vegetation that becomes established thereon, and, so recognizing, discount any need for refinement in the initial structure. He might even be right on both counts.

The planner who must prepare the project, however, finds that there is a general similarity in the layout of such projects, but beyond that point he is largely dependent upon intuition. This is hardly a proper concept of engineering; therefore, several areas in which some investigation would be beneficial are suggested.

In terms of blanket revetment, the most fruitful research would appear to be in the area of the required strength; i.e., for example, the size of rock required, with and without grading, on various bank slopes, to resist a given flow velocity. The required thickness of blanket also needs analysis.

There are uncertainties as to the proper depth to which a revetment should extend. The "toe trench" rock revetment utilized on the Missouri River for 20 years without a failure of consequence is also used successfully on the Arkansas River, yet the writer has been informed that on the latter stream there will frequently be a slip in the original toe rock and that a second step to add additional rock is utilized to obtain a permanent structure. To date, the reason for this difference has not been established. The general dimensions are similar in both cases.

It has also been noted in a small model that the existence of large dune formations leads to excessive scour, owing to deep eddy action at the toe of the dune. This question is probably of academic interest only, since dunes of the relative size noted are not apt to occur in natural streams, but it should be investigated further.

Investigation of the basic action of permeable guide structures without regard for secondary influences are needed, for these secondary influences cannot be effective until engendered by the basic structure. Such an investigation would be required to measure the varied effects of the structure in all positions from normal to the flow to parallel to the flow, and would need to consider the size and spacing of units in relation to flow area as well as frictional effects. A complete velocity pattern should be developed for analysis by momentum or other hydraulic theory; the ultimate purpose is to permit a theoretical analysis to determine the most economical and effective layout and spacing of the frontal line and the various tieback lines required in the deposition area.

The transport function should be investigated further, to determine the proportions of sediment diverted under varied arrangements of the frontal line and varied conditions of transport or of sediment load in the stream.

There should be a simultaneous empirical analysis, since this approach would probably yield the most immediately useful results. The relative effect of the secondary influences would be a necessary study here, as the designer

would, in most cases, desire to reduce the theoretical structure somewhat in accordance with anticipated secondary actions.

In one recent instance,[3] a model study was conducted for an individual project design. Considering the fact that the initial cost of structure in the field can vary from a minimum of perhaps $5 per lineal foot of structure to well over $30 per lineal foot, it seems obvious that the elimination of a single tieline or of a hundred feet or so of a frontal line would more than pay the cost of the model. It is surprising that more such analyses have not been made.

Mention has been made of field investigations of an L-head structure. Actually, the L-heads were added to a series of perpendicular dikes in an effort to improve their action and eliminate some of their deficiencies. In the locations tested; i.e., the concave bank of a flat bend, the improvement has been impressive. The cost of prototype testing, however, will eliminate extension beyond the point of satisfactory action in a particular location. The questions of optimum height and relative length for general application will remain unanswered insofar as the field installation is concerned, but they could be resolved in a model at moderate cost.

Unquestionably many other facets of the problem could be discussed; however, if the preceding material develops some thought and engenders some realistic analyses, the writer's purpose will have been served.

THE RIO GRANDE COMPREHENSIVE PLAN IN NEW MEXICO AND ITS EFFECTS ON THE RIVER REGIME THROUGH THE MIDDLE VALLEY

[Paper No. 41]

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Synopsis

The Rio Grande comprehensive plan in New Mexico consists of a system of reservoirs on the main stream and tributaries. The reservoirs are located upstream from the head of a reach of about 180 miles of rehabilitated floodway through the Middle Valley.

Sediment detention and regulation of floodflows by the reservoirs are expected to prevent further aggradation and inhibit excessive degradation through the rehabilitated floodway.

The comprehensive plan in New Mexico consists of a system of reservoirs (Abiquiu, Jemez, Cochiti, Galisteo) on the Rio Grande and tributaries at and near the head of the Middle Valley and the rehabilitation of the Rio Grande Floodway constructed by the Middle Rio Grande Conservancy District in 1935. The Rio Grande Floodway confines the river throughout most of the Middle Valley, which comprises a reach approximately 180 miles long. Locations of the elements of the plan are shown on figure 1.

The reservoirs are for flood and sediment control. The levee improvements and channel stabilization are necessary in the interest of flood control, major drainage, and water salvage.

In the Cochiti-Rio Puerco reach of the floodway, the Rio Grande occupies a wide shallow channel between the levees of the floodway. The channel has no banks, and the average level is generally at or above the level of the areas behind the levees.

The levees constructed as a part of the floodway by the conservancy district were subject to attacks by the river. Early attempts to protect them by means of pile jetties demonstrated the need for a more effective system of levee protection. Deterioration of the floodway due to aggradation and the high losses of water which were occurring indicated the need for regulation of floodflows, sediment retention, and channel stabilization.

In 1948 the Corps of Engineers and the Bureau of Reclamation jointly recommended the comprehensive plan of improvement for the Rio Grande in New Mexico. The dams would be constructed by the Corps of Engineers and the floodway rehabilitation would be constructed jointly by the Corps of Engineers and the Bureau of Reclamation. The plan was authorized for construction in 1948, except for a main stream reservoir that was to provide control of floods originating principally in Colorado.

A restudy of the problem of main stream control was made by the Corps of Engineers in 1958 and two additional reservoirs were authorized by the Flood Control Act of 1960, one on the main stem at Cochiti at the head of the Middle Valley and the other on Rio Galisteo.

The Jemez and Abiquiu Reservoirs and the floodway improvements in the Cochiti to Rio Puerco reach are constructed, and the Cochiti and Galisteo Reservoirs are in the preconstruction planning stage. The Cochiti to Rio Puerco

[3] CARLSON, E. J., and DODGE, E. A. CONTROL OF ALLUVIAL RIVERS BY STEEL JETTIES. Amer. Soc. Civil Engin. Proc., Waterways and Harbors Div. Jour. 88 (WW4): 53-81. Nov. 1962.

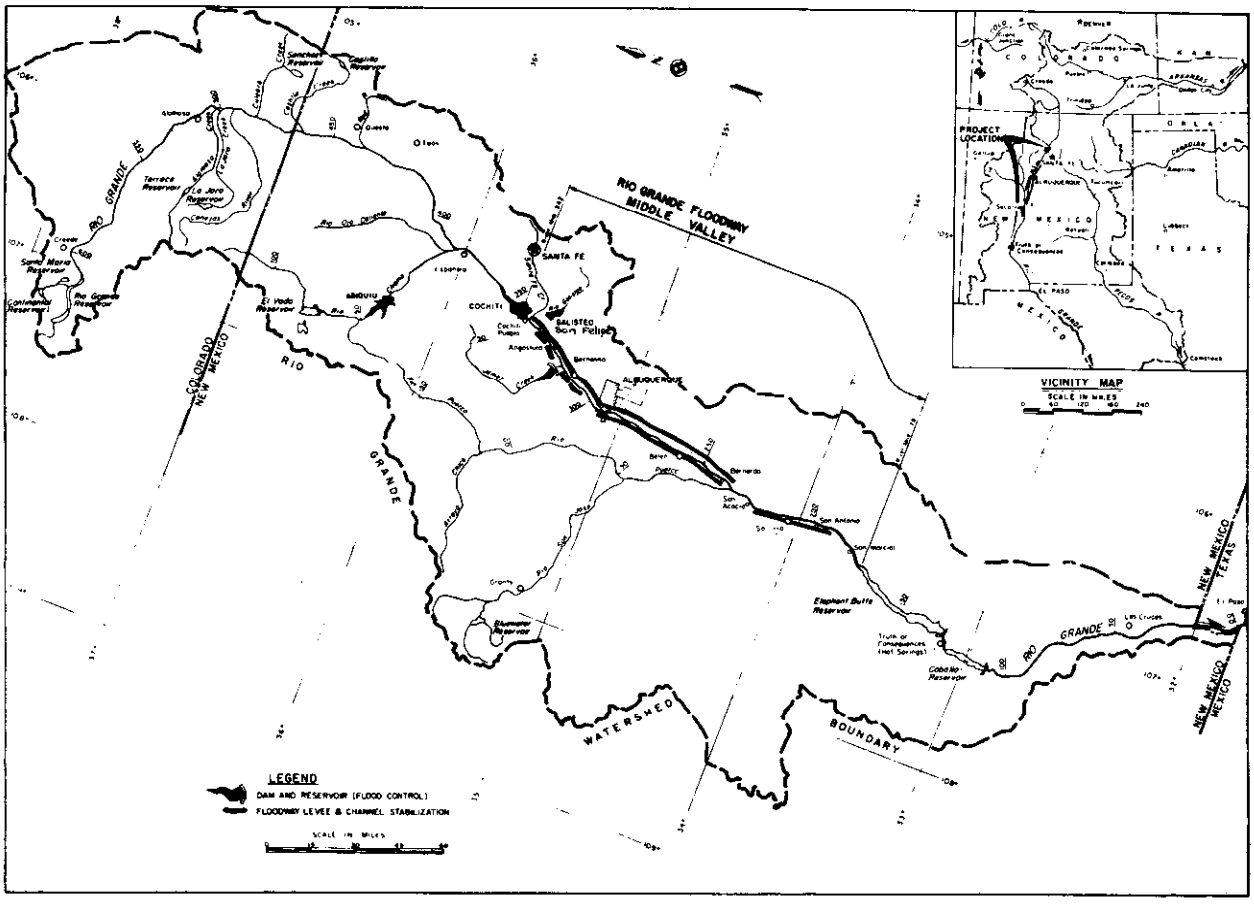


FIGURE 1. — Rio Grande floodway, Middle Valley, N. Mex.

reach comprises about 105 miles of the total floodway length. This discussion primarily concerns the plan of development and its effects in this reach.

The Middle Rio Grande Conservancy District was organized in 1925 under the laws of the State of New Mexico for the primary purposes to improve drainage, irrigation, and flood control for about 128,000 acres of land, including urban population centers within the Middle Valley. The district was subdivided into four divisions: Cochiti, Albuquerque, Belen, and Socorro. The principal works were begun in 1930 and completed in 1935. Because of the inability of the district to finance a flood control system involving a number of reservoirs, the basic flood control element adopted for the Middle Valley was a floodway. Figure 1 indicated the extent of the floodway through the Middle Valley.

In general, the levees were located parallel to the then existing meander pattern of the river. Because the bends in the river are moderate, no significant attempts were made to straighten the river by cutoffs. The width of the floodway between levees is about 1,500 feet and the average height of the levees is about 8 feet. The adopted design flow for this reach was 40,000 cubic feet per second except at the city of Albuquerque where extra height of levee was provided for a design flow of 75,000 cubic feet per second.

Below the Rio Puerco the levees were constructed only on the west side of the river to the village of San Antonio, a distance of about 50 river miles. The design capacity of the floodway in this reach was 50,000 cubic feet per second.

The method of computing aggradation or degradation consisted of determining the volumetric changes in the valley between surveys of the range lines from 1936 to 1953, inclusive. The computations were made for the channel area, the flood plain area, and the combined channel and flood plain area. The channel area consists of that part of the floodway that does not contain vegetation but is characterized by

sandbars and low flow channels. The flood plain area is designated as the remaining areas between levees or bluffs subject to overflow only during periods of high flows. This area usually contains growths of trees, shrubs, grass, and weeds of varying density.

Volumes were computed by multiplying the average depth of ground surface below an established base plane at each range by the area between ranges. The difference between volumes of any two surveys represents the change in volume by aggradation or degradation that occurred in the time interval between the surveys. Some adjustments were made to the surveys prior to the 1952–53 surveys because of differences in methods used and in range numbering. A summary of the results of the aggradation and degradation study for the greatest period (1936–53) between surveys is shown in table 1.

TABLE 1.—*Aggradation and degradation, Cochiti to San Antonio, N. Mex., 1936–53*

River and division		Plane area	Net change[1]	Rate of change[1]
		Acres	*Acre-ft.*	*Ft./yr.*
Cochiti (22.5 river miles)	Channel	2,057	−1,515	−0.043
	Flood plain	2,049	+1,810	+ .052
	Channel and flood plain	4,106	+ 295	+ .004
Albuquerque (39.9 river miles)	Channel	4,286	+1,153	+ .017
	Flood plain	4,183	+4,582	+ .068
	Channel and flood plain	8,470	+5,735	+ .042
Belen (52.6 river miles)	Channel	7,704	−2,575	− .020
	Flood plain	5,575	+3,284	+ .035
	Channel and flood plain	3,279	+ 709	+ .031
Socorro[2] (29.0 river miles	Channel	4,203	+2,614	+ .037
	Flood plain	4,264	+1,520	+ .021
	Channel and flood plain	8,467	+4,134	+ .029

[1] + = aggradation; − = degradation.
[2] Terminates at San Antonio.

The period used in table 1 begins shortly after the completion of the floodway and the diversion dams constructed by the Middle Rio Grande Conservancy District and reflects the full effects of these structures. It also includes the high flow years of 1941 and 1942.

From table 1 it can be seen that the Rio Grande within the Middle Valley is still an aggrading stream, and construction of the floodway has not halted aggradation. Aggradation has occurred in amounts varying from practically zero in the Cochiti Division at the upstream end of the valley to a maximum of about 0.04 of a foot per year in the Albuquerque Division. The indicated channel degradation that is occurring in the Belen Division is not believed indicative of a long-term trend. The Rio Puerco enters the Middle Valley at the lower end of this division. The Rio Puerco is the largest sediment-contributing tributary of the Rio Grande in New Mexico and contributes well over 50 percent of sediment load entering the valley. The indicated channel degradation is believed to be the result of the river cutting through a large sediment plain deposited in the Rio Grande by the Rio Puerco below the mouth during the major flood of 1929. It is known that during this flood a large part of the flow came from the Rio Puerco, together with an enormous flow of sediment. The 1929 flood occurred before the beginning of the period of measurements reflected in table 1. In the event of another flood similar to the flood of 1929, which can be reasonably expected, it is believed this trend toward degradation would be sharply reversed. The river channel at the present time is aggrading above the junction of the Rio Puerco about the same rate as it is degrading below.

In all, about 190 miles of levees were constructed, together with a small number of levee protection works. The levees, together with the natural bluffs along the river, confined the river within the floodway throughout the Middle Valley. In addition to the flood control works, four major diversion dams and several hundred miles of drains, canals, and related works also were constructed.

The work of the conservancy district did much to improve distribution of irrigation water and reduce waterlogging of lands within the district. However, after the major flood year of 1941, it was clearly evident that the floodway would not afford the protection that it was designed to provide. The mean daily peak of this flood was only 22,500 cubic feet per second; however, this long duration flood covered a period of about 2 months with flows about 20,000 cubic feet per second for 5 days and above 10,000 cubic feet per second for 43 days. The levees were breached in 25 places and active flood fighting was necessary at 80 different locations along the river. Extensive flood damage was experienced.

Subsequent to this flood, the Corps of Engineers made a comprehensive investigation for flood control along the Rio Grande in New Mexico with a view to developing a more adequate plan for flood and sediment control. Because of the effects of the river regime on the agricultural economy and the need for further improvement of the irrigation works of the conservancy district, the U.S. Bureau of Reclamation began cooperative studies with the Corps of Engineers. Emphasis was placed on determination of aggradation or degradation that had occurred in the Middle Valley after the construction of the works of the conservancy district. Considerable speculation had existed as to whether or not the trend toward aggradation was continuing and to what extent it had taken place.

Fortunately in 1936, shortly after the Rio Grande floodway was placed in operation by the Middle Rio Grande Conservancy District, the Soil Conservation Service of the U.S. Department of Agriculture established and surveyed river cross sections or ranges across the floodway within the entire Middle Valley. The ranges were to be resurveyed at later dates so that changes in riverbed elevations could be determined. These ranges were located approximately at 2-mile intervals. Additional ranges were established by the Corps of Engineers in 1944, which reduced the interval between ranges to one-half mile. The ranges were surveyed in 1936, 1940, 1941, 1942, 1944, and 1952-53. A cooperative study of aggradation and degradation based on these surveys was made by Bureau of Reclamation and the Corps of Engineers.

The rate of change for the period covered by the surveys within the Albuquerque division is illustrated in figure 2. The high runoff years influenced the rate of change in both the channel and the flood plain. The occurrence of a high runoff year like 1941 was instrumental in keeping the 1936–53 aggradation rate in the channel to a low amount while increasing aggradation rate in the flood plain. Another factor that is partially shown on figure 2 is the influence of the relative order or cycle of flows upon the rate of change. An example is the change that resulted from the 1942 runoff, which was a high flow year after the still higher flow in 1941. The aggradation rate in the flood plain was about the same for 1941 and 1942, whereas the channel change was practically zero. It is doubtful that the change in 1942 would have been the same if it had followed a series of low runoff years. The conclusion reached, from the plotting of figure 2, is that within the limits of the Middle Valley there is a definite relation between the annual runoff and the aggradation or

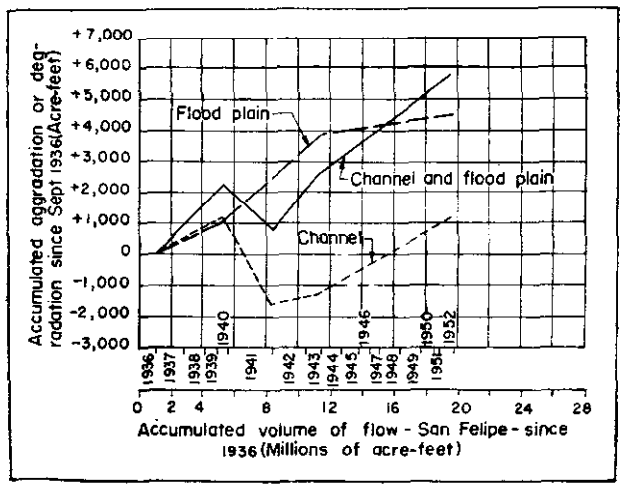


FIGURE 2. — Rate of change — Albuquerque Division (1936-53).

degradation in the channel and flood plain. A further fact revealed by the surveys, but not shown in table 1, is that there has been no material net change in the relation of areas of channel and flood plain since 1936.

Simultaneously with the survey of the river ranges in 1936, the Soil Conservation Service began an extensive program of sediment sampling that has been carried on through the cooperation of other Federal agencies, including the Corps of Engineers and the Bureau of Reclamation. The purpose of the sediment sampling program was to determine the origin, quantities, and characteristics of sediments transported by the Rio Grande. When this program was begun, except for the station at San Marcial, less than 20 samples of water had been analyzed for sediment content. To date about 70 thousand integrated samples from the main stem of the river and tributaries have been collected and analyzed. The majority have been collected within the Middle Valley.

Flows are very turbulent which makes suspended sediment samples from the Rio Grande more nearly indicative of the total load than similar samples from other rivers. The suspended sediment samples contain a considerable amount of sand, which increases with increases in rate of flow. For example, the sand load in the samples at Cochiti averages about 3 percent of the total load from flows less than 500 cubic feet per second, and 34 percent in 32 samples from flows between 2,000 and 3,000 cubic feet per second. The amounts at Bernalillo are 17 percent in 29 samples from flows less than 500 cubic feet per second, and 29 percent in 16 sam-

ples from flows between 2,000 and 3,000 cubic feet per second. Table 2 shows sediment flows through 1954 at the principal stations within the Middle Valley.

TABLE 2.—*Discharge and sediment summary, 1936–54*

Division and station		Drainage area	Average annual discharge	Annual sediment discharge
		Sq. mi.	*Acre-ft.*	*Acre-ft.*
Cochiti division	Cochiti	11,440	1,212,400	2,930
Albuquerque division	Angostura	12,941	1,165,900	3,342
	Bernalillo	14,059	1,089,000	3,921
	Albuquerque	14,502	1,054,000	3,764
Belen division	Isleta	14,737	1,127,000	3,925
	Belen	15,291	1,039,800	3,616
	Bernardo	15,929	1,017,800	3,446
	San Acacia	23,623	1,092,150	10,182
	San Antonio	24,460	1,058,900	9,645
Socorro division	San Marcial	24,558	1,061,500	7,626
	Head Elephant Butte Reservoir	24,600	1,063,900	5,838

The significant increases in sediment flow from Angostura to Bernalillo are due to the inflow from Jemez Creek, which enters between these two stations. The large increase between Bernardo and San Acacia is due to the Rio Puerco, which enters between these stations. The sediment inflow at Cochiti is derived principally from the Rio Chama.

The general trend toward aggradation indicated by the surveys of river ranges also is confirmed by the sediment discharge compilation in table 2. For example, in the reach between Bernalillo and Bernardo, which is between two of the major sediment contributors to the Middle Valley, Jemez Creek and Rio Puerco, the annual inflow of sediment is 3,923 acre-feet and the outflow is 3,446 acre-feet, despite an approximate increase in drainage area of 1,900 square miles.

The plan for rehabilitation of the floodway follows in general the original levee alinements. However, the design capacities for the floodway were materially reduced except in the reach below the Rio Puerco. This is due to the high degree of control provided by the reservoirs above the Rio Puerco. The design capacity above the Rio Puerco is 20,000 cubic feet per second except at Albuquerque, where the capacity is 42,000 cubic feet per second. Below the Rio Puerco, the design capacity is 60,000 cubic feet per second.

The most noteworthy improvement of the floodway is the provision of a rectified channel within the floodway from Cochiti to the Rio Puerco. The purpose of this channel is to provide protection to the levees and decrease the existing channel area in the interest of reducing water losses and improving water and sediment transport capacity. The original river channel, which averaged about 800 feet in width, was narrowed to a uniform width of about 550 feet. This was accomplished by an arrangement of jetties projecting from the levees on both sides of the river to the channel and lines of jetties paralleling the channel.

An interesting feature of the channel rectification work from a sedimentation standpoint is the use of the Kellner jetty system. In the past many types of levee protection and channel rectification works have been used on the Rio Grande. These have been generally of the more rigid types, utilizing timber piling in combination with woven wire or wire baskets of brush or rockfills. Jetties employing piling have proved generally unsatisfactory for conditions prevailing on the river. Channel scour often undermines the piles and destroys their retarding ability. The Kellner jetty system is particularly adapted to heavy silt-laden streams subject to considerable variation in channel scour during periods of high riverflow. These steel jetties are composed of permeable and extremely flexible multiple-units that readily conform to channel scour. They permit sediment-laden flows to penetrate the jetty where velocity and energy are reduced. Sediment is deposited behind the jetty, thereby building up an area or bank behind the jetty. Vegetation grows rapidly on the accumulated sediment, which further increases the retarding effects. The system is particularly adapted to the protection of levees, riverbanks, and bridge abutments.

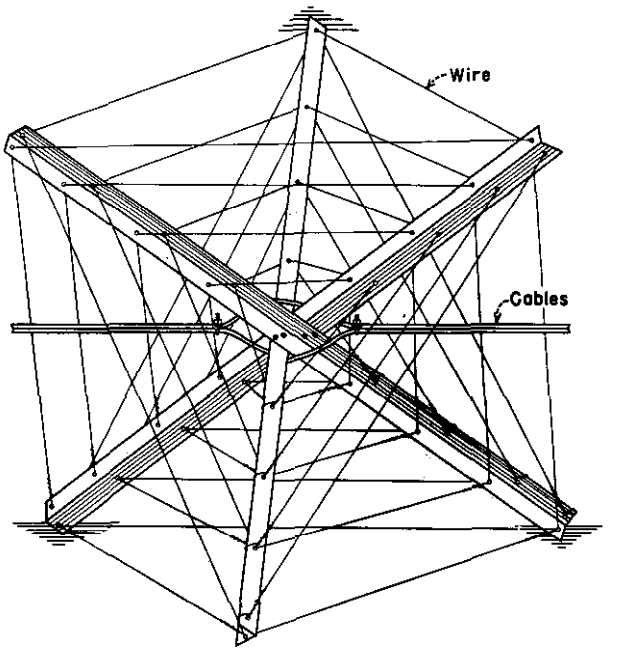
The single or standard unit of the system is called a "Jack" and connected together they form a jetty. A series of lines of jetties form a jetty field. The layout of a jetty field involves two types of lines of jetties, the diversion lines and the tiebacks, or retard lines. The diversion lines, usually two in number, are placed roughly parallel to the levee or bank of the river. The retard lines extend from the diversion line back to the levee, or bank. They are usually placed at an angle of roughly 67.5° to the diversion line

at any desired spacing between lines. As installed on the Rio Grande, the spacing of retard lines at river bends is usually about 250 feet. Single diversion lines and wider spacing of retard lines are used in less critical areas.

The Kellner Jetties Corp. pioneered the development of the system. General principles usually followed in the layout of a jetty field and the design at curves are based on model studies of the retarding effects of the jetties and of experience. More detailed information is contained in the Corps of Engineers' "Report on Measures for Bank Protection, Use of Kellner Jetties on Alluvial Streams," June 1953, revised August 1962, and the Bureau of Reclamation's publication, "Use of Steel Jetties for Bank Protection and Channelization in Rivers," by Enos J. Carlson and Phillip F. Enger, Hydraulic Engineers. This type of jetty is comparable in cost to the more rigid types, but the effective life is much longer. There are installations with service records of 30 years with a reasonable life expectancy of 50 years. Figures 3 and 4 illustrate the standard unit and its use in a typical jetty field utilized by the Corps of Engineers and the Bureau of Reclamation on the Rio Grande floodway. Figure 5 shows the progressive effectiveness of a jetty field in causing deposition and growth of vegetation to change the regime of the river.

Studies were made by the Bureau of Reclamation to estimate the combined effects of construction of the flood and sediment control reservoirs and the channel rectification work on the river regime within the Middle Valley. The effects on floodflows were readily determined by routing studies. The effects on channel aggradation and degradation were more complicated; however, the data collected from the sediment studies and survey of the river ranges were believed sufficient for a reasonable estimate of anticipated effects. The procedure followed in general is as follows. The river was divided into convenient reaches that were treated as units with similar characteristics within the reach, such as slope, width, and riverbed material. From the suspended sediment sampling and flow records, the suspended load was determined. Bedload, or the load not measured by sampling, and the total load were then determined by the use of the modified Einstein method developed by the U.S. Geological Survey and the Bureau of Reclamation. This was the basic condition without dams or rectified channel.

If the same reaches but with modified condition of rates of flow and sediment inflow due to the construction of the dams and the rectification work are used, the capacity of the reach to transport sediment was determined by application of the Lane-Kalinske formula for suspended sediment load and the average of the Kalinske and Schoklitsch formulas for the unmeasured load. The results were then adjusted by a factor to conform to the results obtained by the application of the modified Einstein method. This factor was determined by comparing the results obtained with the use of these formulas in the unmodified conditions with the results in the same reaches obtained by the modified Einstein method. Samples of streambed material were obtained at the river ranges and from these, composition of the riverbed material in each reach was estimated for several divisions of grain sizes, each of which is subject to transport at different rates of flow. The amounts of sediment entering the reach were subdivided in a similar fashion. Assumptions were made as to depths of scour of streambed material. The sizes smaller than sand were assumed to pass through the reach. With the ability of the stream to transport the various sizes and the total amount of the various sizes in the riverbed and entering the reach available for transport known, outflow from the reach was determined. This process was repeated for each successive year and the net differences in inflow



NOTES:
(1) Unit is usually 16'x 4"x 4"x $\frac{1}{4}$" angle iron laced with No. 6 wire.
(2) Cables are usually $\frac{3}{4}$" ϕ.
(3) Above unit is placed 12$\frac{1}{2}$' on ctrs.

FIGURE 3. — Standard unit Kellner jack.

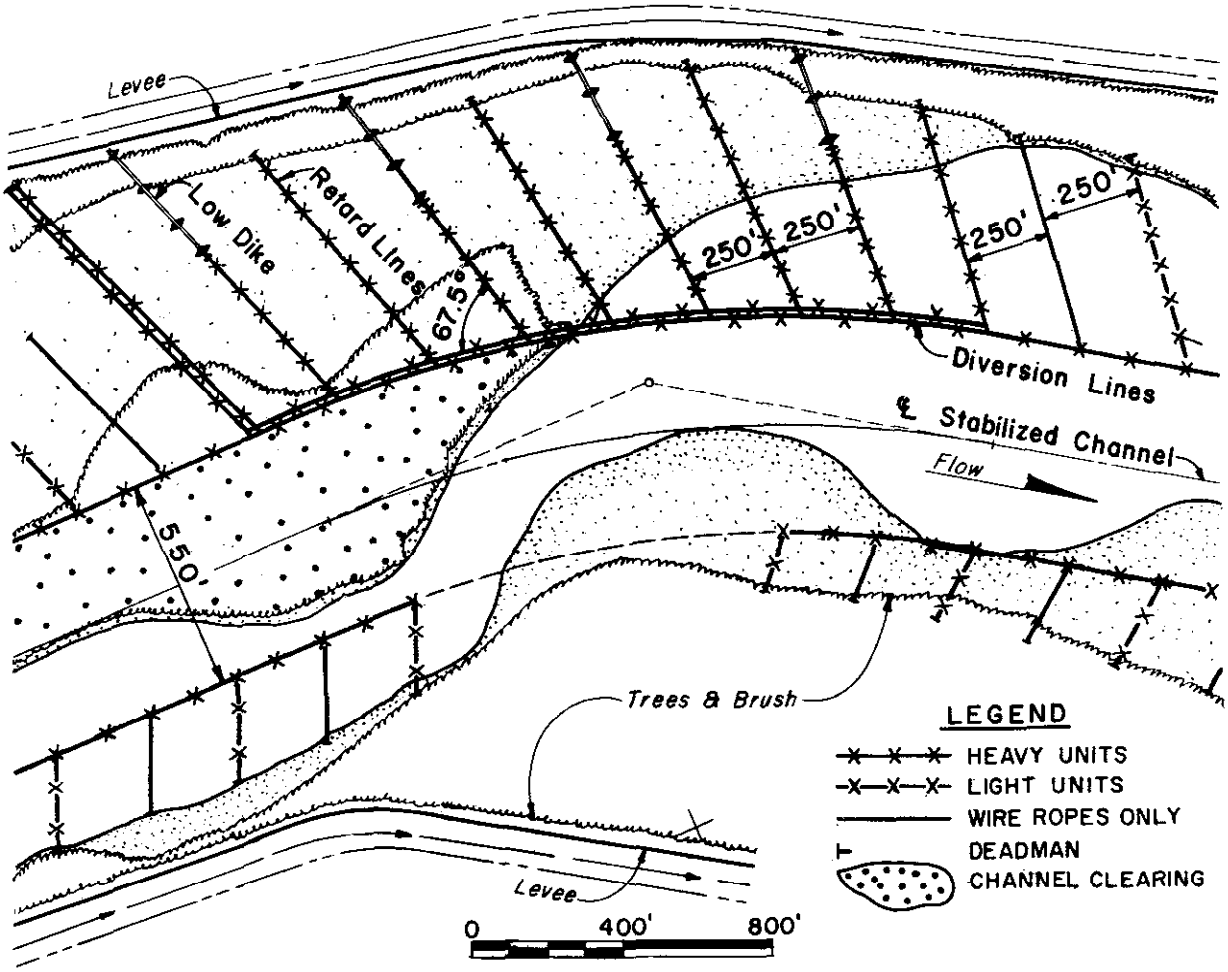


FIGURE 4. — Typical Kellner jetty field—Rio Grande, N. Mex.

and outflow of sediment indicated aggradation or degradation for the reach.

The results indicate that the sediment inflow at Bernalillo will be reduced by the reservoirs about 75 percent after about 20 years and that the present trend of aggradation will be reversed. Indications are that degradation will begin at Cochiti and will extend downstream at least to the Rio Puerco. Progressive coarsening of the streambed will inhibit degradation. It appears unlikely that the stream will degrade more than about 3 feet in the Bernalillo to Albuquerque reach. It is probable during the first 10 to 15 years after the reservoirs are all completed and in operation that considerably more degradation will be apparent in certain locations. This will be due to the confinement of the clear water releases by existing bars, etc., while the 550-foot channel is being flushed out. After this flushing has taken place, the energy of the clear releases will be more evenly distributed over the full width of the channel; flows will have less sediment transportability. As a result, it is anticipated by the authors that this will tend to inhibit excessive degradation at any one place.

The Corps of Engineers and the Bureau of Reclamation have a cooperative agreement to share the costs of periodic photogrammetric resurveys of the entire Middle Rio Grande Valley. The initial condition survey was made during the spring of 1962. Low level contact prints, controlled aerial mosaics, and cross sections to a scale of 1 inch equals 500 feet at approximately 500-foot intervals will be obtained. These data will be analyzed to check the theoretical estimates of sediment movement, including location of aggradation and degradation. Also, the photographs will show the progressive changes in vegetal growth.

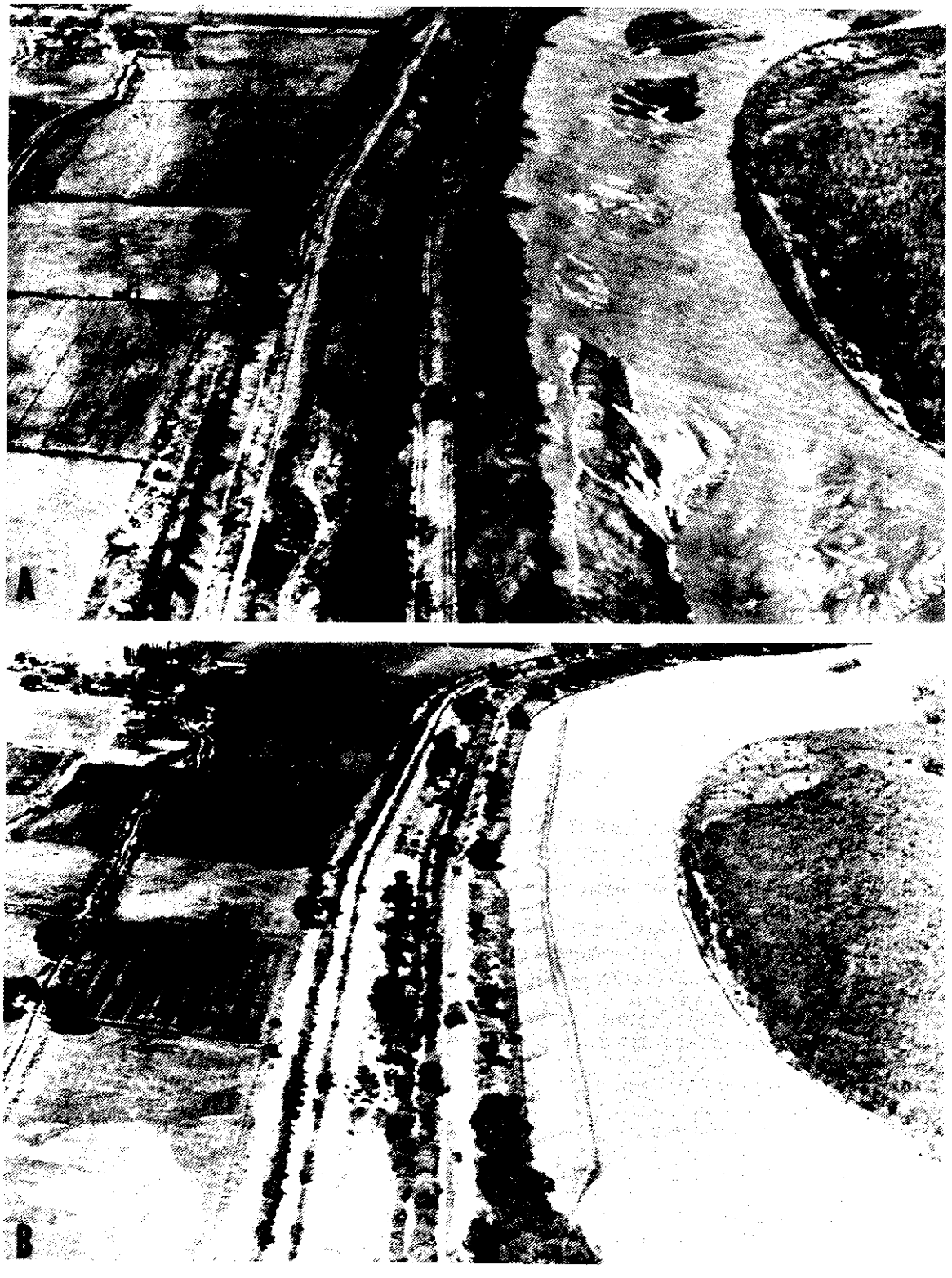
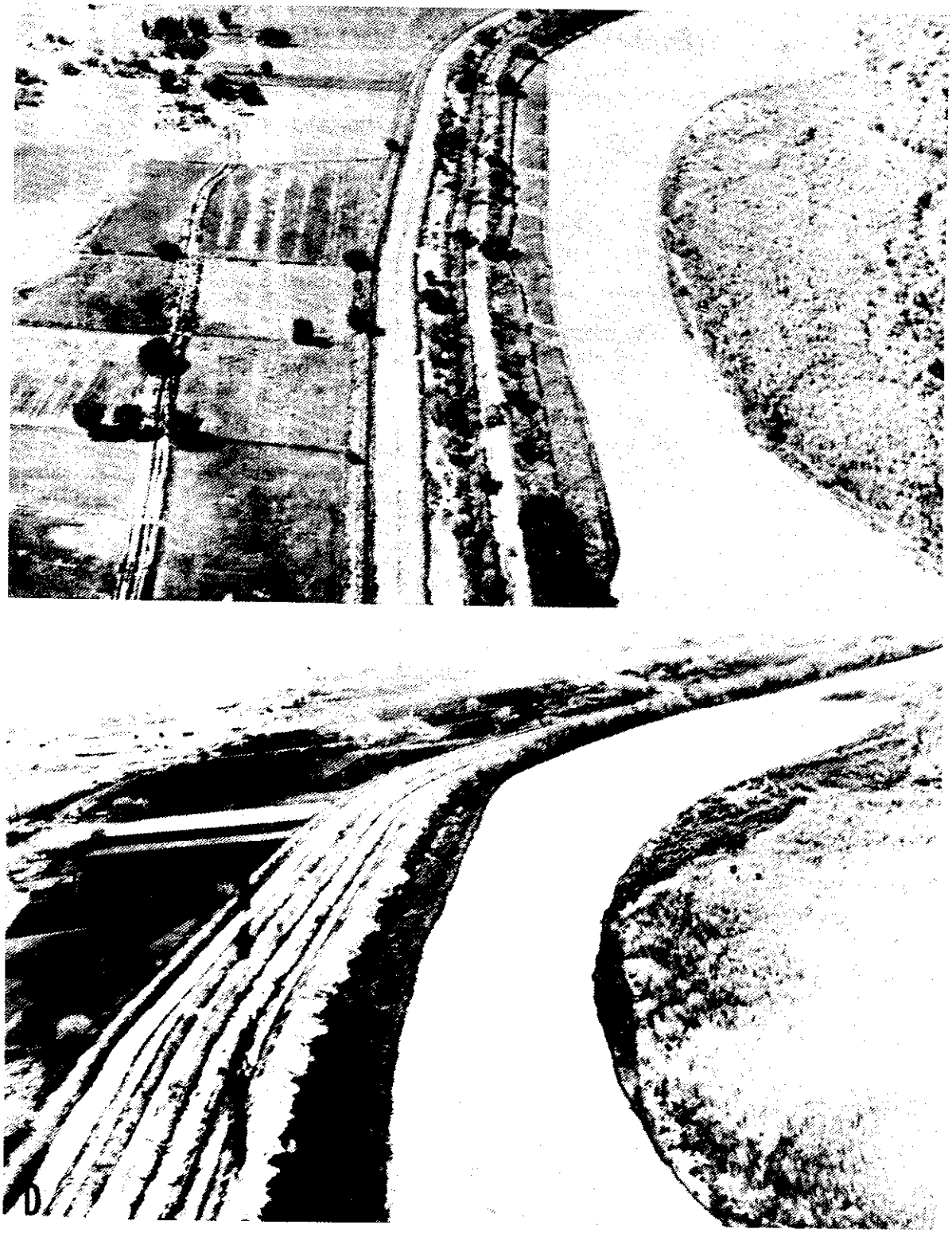


FIGURE 5. — Emergency flood control work on Rio Grande near Bernalillo: *A*, December 1952, looking downstream prior to installation of bank protection works; *B*, August 1953, at the same location after installation of Kellner



jetty field in June 1953; *C*, September 1955, about 2 years after completion of emergency work; *D*, April 1958, about 5 years after completion of bank protection works.

METHODS AND CRITERIA FOR BANK PROTECTION ON THE LOWER COLORADO RIVER

[Paper No. 42]

By FRANK K. ILLK, *hydraulic engineer, Bureau of Reclamation*

Introduction

Bank and bed stabilization along the Lower Colorado River has a history extending back over 50 years. The Bureau of Reclamation first became actively engaged in the problem in connection with the revetment of the levee system protecting the Yuma project near Yuma, Ariz. This work was accomplished in the early years of this century.

Other individuals and agencies, notably the Palo Verde Irrigation District, were also engaged in bank stabilization activities at an early date.

The closure of Hoover Dam in 1935, with its high degree of control over the flow of the Lower Colorado River, instituted a new river regime in which channel stabilization has become increasingly important (fig. 1). The necessity for dealing with the problems inherent to the new regime led to the creation of the Colorado River Front Work and Levee System by Public Law 469—79th Congress, Second Session. The basic responsibilities of the Bureau are set forth in the following quotation from the Act:

(a) operating and maintaining the Colorado River Front Work and Levee System in Arizona, Nevada, and California; (b) contructing, improving, extending, operating, and maintaining protection and drainage works and systems along the Colorado; (c) controlling said river, and improving, modifying, straightening, and rectifying the channel thereof; and (d) conducting investigations and studies in connection therewith.

Under the authority of this Act the Bureau, through Region 3, has engaged in a variety of activities planned to produce eventually a stable channel from Hoover Dam to the Mexican Boundary.

River Characteristics

Historically, the Lower Colorado River carried enormous loads of sediment. Practically all of the sediment originated in the upper basin above the present site of Hoover Dam. The lower basin was apparently actively aggrading, with the sediments of the river gradually building up the bed and banks of the stream and raising the floor of the canyons and valleys.

This natural regime was abruptly terminated by the construction of Hoover Dam and the formation of Lake Mead. This lake has ample capacity to store the sediments originating in the upper basin and as a result, the release from Hoover Dam is almost completely sediment free. The introduction of this clear water into a channel which had been graded and molded by flows containing sediment concentrations of up to 50,000 parts per million caused a rapid readjustment in the channel characteristics.

A factor that causes the Lower Colorado River and its problems to be somewhat unique is its location within one of the most arid regions in the United States. In the entire 440 miles of river from Hoover Dam to the Gulf of California there are only two perennial streams, the Bill Williams and the Gila, and neither of these normally contribute an appreciable amount of either water or sediment inflow. As the result of no inflow and the diversion of water for irrigation at many points along the river, the flow diminishes downstream from Hoover Dam until, more often than not, it dries up completely at or just below the southerly International Boundary.

The sediment pattern is somewhat more complex. Since there is essentially no sediment inflow to the river, all of the sediment that moves through the Lower Basin is derived by scour from the alluvium of the bed and banks. Adding to the complexity is the effect of the various dams along the lower river. Below Hoover Dam, the river is divided into a series of reaches, or divisions, by Davis, Parker, Headgate Rock, Palo Verde, Imperial, Laguna, and Morelos Dams. These dams affect the sediment pattern more or less according to their functions. Storage dams such as Davis and Parker intercept any sediment picked up by the river and reintroduces clear water into the river downstream. The diversion dams mark the points of depletion of the flow and in some cases increase the sediment load downstream by diverting larger portions of water than of sediment or by returning sediment to the river. Each of the reaches between these dams differ somewhat in the character of the channel and in the control problems associated with these conditions.

The unique circumstance that practically all of the sediment load of the Lower Colorado River is derived from the bed and banks of the stream itself, gives an added urgency to the channel stabilization program and also makes possible as an ultimate solution the creation of an essentially sediment-free conveyance channel from Hoover Dam to the lowest diversion.

Yuma Area

The early attempts at bank protection and stabilization works in this basin were intended

to counteract the strong meandering tendencies of the natural river. Early bank protection works by the Bureau in the Yuma area were required to protect the Yuma Levee. The levee was rarely overtopped, but the massive snowmelt floods that occurred each spring often changed the channel alinement until the direct flow of the river was against the levee. When this occurred the levee was undermined and collapsed. These conditions led to the use of heavy quarry-run rock for riprap to protect the levee where it was subject to direct attack by the river. Eventually this riprap was extended to protect the entire levee. The material was not selected as to size but, due to the handling problems involved, stone of about one-half cubic yards was about the practical maximum size.

After the completion of Hoover Dam, no more floods occurred and little if any maintenance work was done on the levee. With the authorization of Morelos Dam, the Mexican diversion structure located in the river below Yuma, the flood hazard in this area was restudied. Although the present flood potential is considerably less than in the natural river, it was still sufficient to require the reconstruction of the Yuma Levee. The levee was reconstructed, in part, to new grade and alinement to meet current conditions. The levee section was increased and heavy riprap placed as indicated. The maximum size of the riprap was limited to one-half cubic yard and was graded down to not less than one-tenth cubic yard. The rock was placed by dumping on the shoulder of the levee and pushing it over the slope with a bulldozer.

The design flood used for the levee rehabilitation was not only smaller in magnitude than pre-Hoover Dam floods but is also not expected to be of as long duration. For this reason, only part of the levee was riprapped. Where riprap was required by the proximity of the river it was placed in blankets either 5 or 10 feet thick measured normal to the embankment slope. The thicker blanket was used only where the alinement of the river threatened to direct the full force of current directly against the levee.

Mohave Valley

About 1944, the change in regime induced by Hoover Dam and Parker Dam resulted in what has been referred to in numerous reports and publications as the "Needles Situation." The backwater from Havasu Lake behind Parker Dam extends into the lower end of Mohave Valley, an area that appears to have been slowly aggrading even under natural conditions. The clear water released from Hoover Dam scoured large quantities of sediment from the streambed and deposited this material in the lower end of Mohave Valley. The buildup of sediment was so rapid that by 1942 the channel was lost and the river flowed through a series of sloughs. The rise in water surface accompanying this aggradation flooded part of the city of Needles, Calif., and threatened the main line and yards of the Santa Fe Railroad.

As a permanent solution to this problem, the Bureau obtained a dredge and in 1949 began work to reopen a channel from Needles to Topock. It was realized that to prevent continued excessive deposition in this channel the river would have to be denied access to the unlimited source of sediment in the upper valley. To achieve this objective it would be necessary to dredge a channel 32 miles in length through the valley and protect the banks of the channel from erosion.

The rectified channel was designed to carry an average flow of 15,000 c.f.s., and levees were provided to contain floodflows varying from 50,000 c.f.s. at the upper reach to 75,000 c.f.s. at the lower reach of the channel.

Bed stability of the rectified channel could not be achieved directly because of the excessive natural grade through the valley. In the design of the channel it was advantageous to adopt a sinuous alinement to reduce the gradient as much as possible as well as for other purposes. By designing to the minimum practicable grade and by confining and deepening the channel, it was possible to obtain conditions under which the bed of the channel can eventually adjust to stability without having to move excessive amounts of bottom material.

Because of the emergency conditions at Needles, the first concern was to open a free-flowing channel through the deposition area. As a result the dredge was put to work to open a pilot channel just wide enough to capture the flow of the river. The river was expected to widen this channel by bank erosion until the design width was reached. Bank stabilization would then be provided as needed to prevent the formation of overwide sections.

Various methods of bank stabilization were tried with something less than complete success. One of these early methods that was tried was a permeable structure designed to cause deposition of the river's sediments in desired areas. These structures were constructed of railroad rails jetted into the bed at 3- to 5-foot intervals supporting 6-inch mesh wire fencing. Initially these structures were reasonably successful, because the river was still carrying a fairly high sediment concentration. However, as the channelization activities began to reduce the sediment load of the river, it was found that these structures no longer performed satisfactorily. It soon became evident that with the velocities of 3 to 6 feet per second encountered

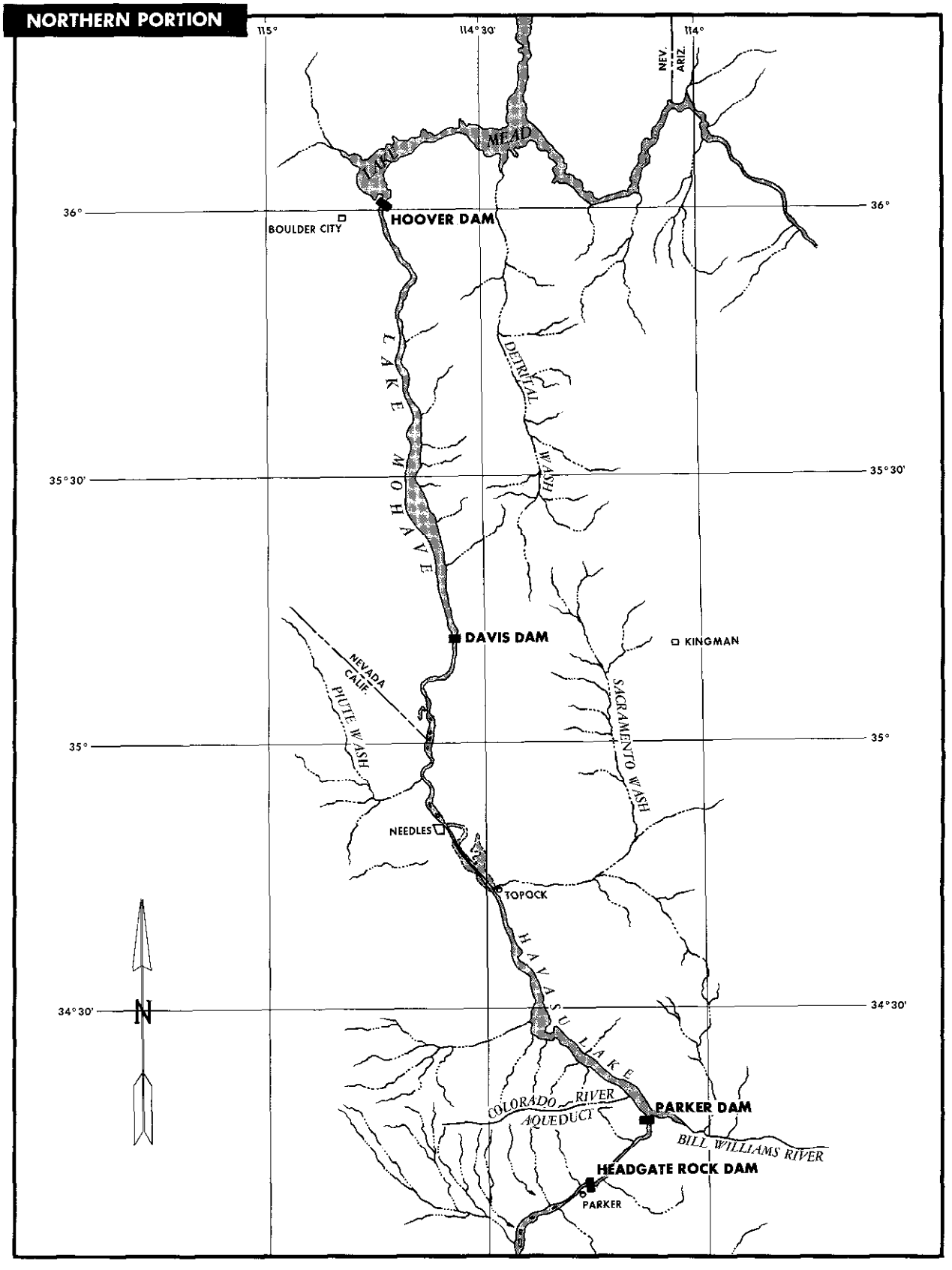
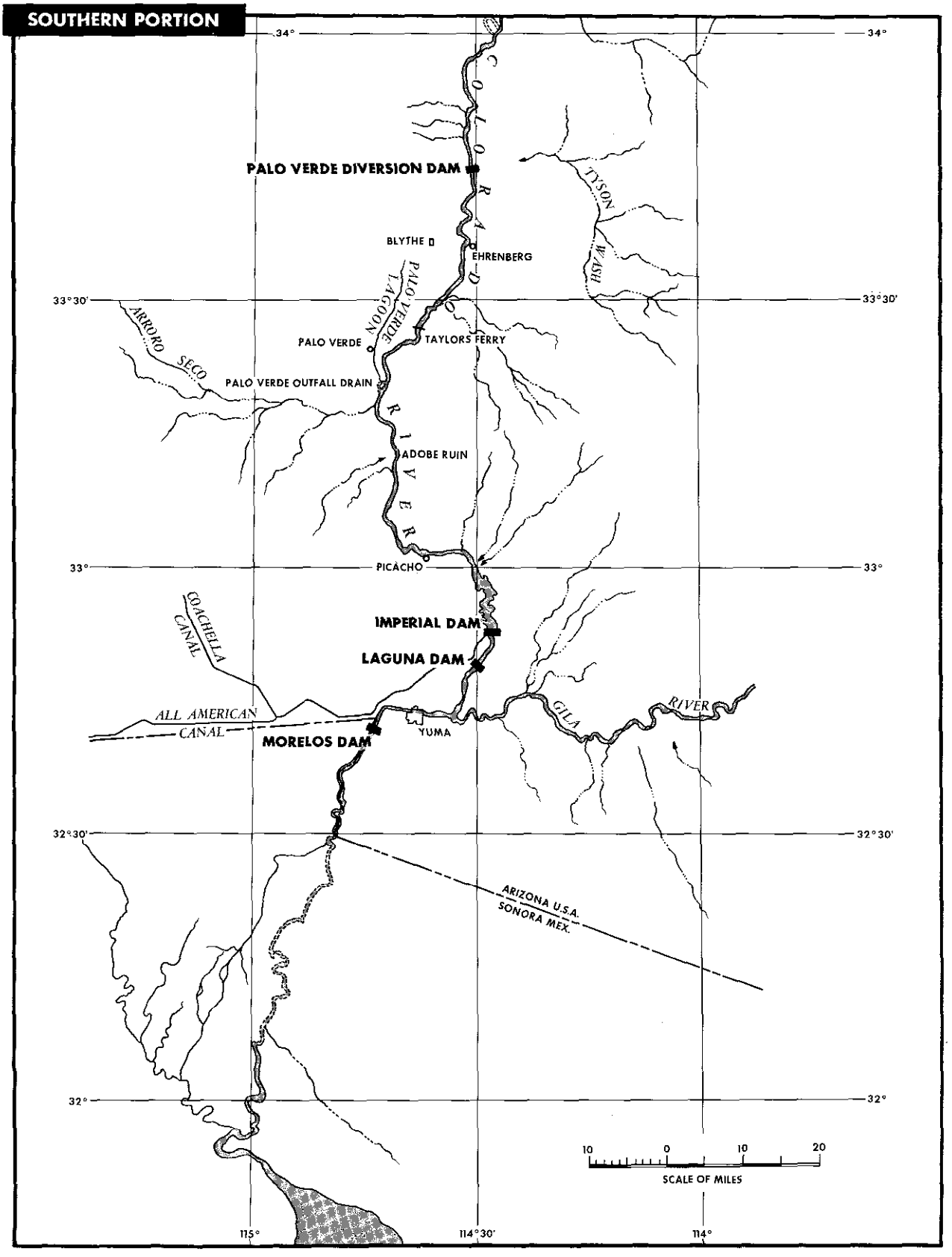


FIGURE 1. — Location map



of Lower Colorado River.

along these banks, the sediment concentration in the flow had to be about 700 parts per million to obtain adequate deposition. When the concentration of sediment dropped to about 300 parts per million or less, the fence itself actually caused some erosion to take place. Jetties formed by Kelner-type jacks were also tried, but with about the same results. In time, all these structures were replaced with rock groins or hard points spaced along the erosion areas. These hard points have been successful in controlling the bank erosion but produce a rough scalloped bank line that may not in all cases be desirable. In addition, such protection does permit some additional bank cutting, which may produce an objectionable amount of sediment in some circumstances.

In later channelization work in the Mohave Valley, a different method of bank stabilization was used that proved highly succ ssful. Because the channel above Needles was to consist of a series of curves, each with a radius of 10,000 feet, and the channel would be dredged to nearly full width, it was decided to place riprap on the concave sides of all bends. This would have resulted in stabilized banks amounting to about one-half of the total length of bank.

Several methods of placing riprap were tried before the final method was adopted. Because the bank material was almost entirely noncohesive sand and silt that was easily erodible, it was possible to place the riprap material on the ground in the form of a windrow and let the river cut the bank away until the windrow was reached (fig. 2). As the river undercut the riprap windrow, the material began to drop down the cutbank and gradually brought the cutting action to a halt.



FIGURE 2. — Riprap material on the ground in the form of a windrow.

Because of the rather large fluctuation of flow in this channel, from about 3,000 to 24,000 c.f.s., and the erodibility of the bank material, it was found that the plan to riprap only the concave bank would not achieve complete stability. As a result, it was necessary in practically all cases to stabilize both banks.

In practice, the windrows were built in advance of the dredge cut on the desired final bank line. The material used for riprap was quarry-run stone with a maximum size of one-fourth cubic yard and with the fines limited to approximately 5 percent. This material was applied at the average rate of about 1½ to 2 cubic yards per linear foot of bank, depending upon the height of the bank. The dredge then opened the channel between the windrows to nearly full width. The material between the dredge cut and the windrow was allowed to erode away and the riprap stabilized the channel bank at the desired location. The final bank line resulting from this procedure presents a neat appearance and has good flow characteristics.

Experience has shown this method of bank protection to be quite successful. Several years have elapsed since this material was placed, and replacement or addition to the original riprap has rarely been necessary. It can be expected that some future maintenance will be required, but judging from past performance this will not require the replacement of a large amount of rock.

Palo Verde Valley

The Bureau of Reclamation has recently begun a program of bank stabilization between the Palo Verde Diversion Dam and Taylors Ferry. This division includes about 27 miles of river and is part of the degradation reach below Parker Dam. The present bed of the channel is entrenched from 12 to 14 feet below the prescour bed. The bed seems to have reached a reasonably stable condition. Either degradation or aggradation may still occur, depending upon flow conditions, but these fluctuations are minor and there is no noticeable trend toward continued degradation.

Despite the general stability of the bed, this reach continues to produce sediment at the rate of about 650,000 tons per year. Since the bed shows little average change, it is apparent that most of this sediment is produced from bank scour. Other observations bear this out, since there are at least 65,000 linear feet of bank in this division which show visible signs of active bank scour.

Although these banks usually cut away rather slowly, only a few feet a year, in one

case measurements show that one bank cut back 300 feet in 1 year.

In addition to bank erosion, there are other channel control problems within this division. Alinement is poor in several places with overly sharp curves that encourage active bank erosion. Although the river in the division as a whole has degraded, there have developed short aggrading reaches in which the channel is overwide and badly braided. Unquestionably the problems of misalinement, overwidth channels, and braiding could have been corrected more advantageously by channelization; however, physical and economic considerations made this solution impractical. As a result, the present program was devised not only to stabilize actively eroding banks but also to improve alinement and other irregularities as much as possible.

The plans for each section of bank to be stabilized is developed individually to fit the particular circumstances at that point. To date one contract has been completed to stabilize about 18,000 linear feet of bank. Included are three structures, designated C–2, A–1, and A–2, that, while not entirely typical, at least illustrate the three main types of stabilization structures planned for this division.

The area selected for the start of this program has the most severe bank erosion in the division. The severity of the erosion is caused by a series of very sharp bends associated with the development of a new meander pattern growing from a cutoff of an older meander. These bends have radii of only 2,500 to 3,500 feet, whereas, experience on this river indicates that for reasonable stability, bends should have radii in excess of 5,000 feet.

In view of the excessive curvature of these banks, a rather high rate of application of riprap was selected. The structure designated C–2 is on the California side of the river on the bend where 300 feet of bank cutting was measured in 1 year. The bend has a radius of approximately 2,500 feet. It was decided to stabilize this bend at its present location. The bank was trimmed to a slope of about 1½ to 1, corresponding to the observed bank angle common to this area. During the trimming process, minor points and irregularities were also smoothed but otherwise the bank alinement was not changed. Since the bank material is "sugar sand," a fine cohesionless sand and silt, a 1-foot blanket of pit-run gravel was placed on the bank as bedding for the riprap. The gravel — natural "wash gravel — was not graded but was limited to 4-inch maximum size. Finally, quarry-run stone was placed on the bank at the rate of 5 cubic yards per linear foot. Specifications limited the maximum size of the riprap to one-fourth cubic yard, and rock spalls or gravel inclusions were not to appreciably exceed the amount required to fill the voids. Both the gravel bedding and riprap were placed by dumping at the edge of the bank and pushing the material over with a bulldozer.

In the case of structure A–1, it was decided that erosion should not be completely stopped. The bend in this area was such that it appeared advisable to allow erosion to continue for a time to obtain a better shaped bank line. A desirable alinement was selected, and, where the present bank coincides with this alinement, the bank was treated similarly to that in structure C–1. Where the final bank line will be back of the natural bank, the riprap material was placed in a windrow along the desired bank line. Since the bank is higher and the curvature greater than the banks in Mohave Valley, the amount of rock in the windrow was increased to 4 cubic yards per linear foot. In a way, this type of structure defeats the avowed purpose of sediment abatement in this division. For this reason, such a procedure would only be used when the long-range benefits of improved alinement seem to be worth the temporary disadvantage of a limited amount of continued sediment production.

Structure A–2 represents a different method of achieving both bank stabilization and improved alinement (fig. 3). This bank combined

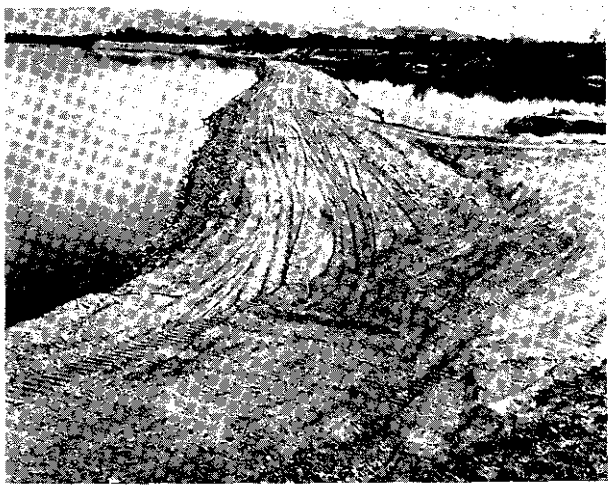


FIGURE 3. — Embankment for bank protection and alinement.

a sharp bend, radius about 3,500 feet, with an even sharper hook at the lower end. Incidentally, this is a form of bank development that is fairly common in this part of the river. The corrective action that was adopted in this case was to build a new bank in the river itself. The

entire structure was designed to ease the curvature of the bend as much as practicable and to close off most of the downstream hook. The part of the design structure that coincided with the existing bank was treated in the same manner as the other bank structures. The new bank or training structure was constructed of unclassified gravel material to a top width of 20 feet. Although levee-shaped, these structures are not intended to act as levees and are built only to an elevation that will give a reasonable freeboard at a flow equal to the normal maximum release through the Parker powerplant. The river side of the structure was riprapped in the same manner as the regular banks.

As yet, these structures have not been in place long enough to judge their performance. Certainly in time there will be some shifting of the riprap material particularly if as expected the channel deepens adjacent to the structure and undercuts the toe of the riprap (fig. 4).



FIGURE 4. — Riprap being placed by bulldozer.

Future plans in this division call for several thousand feet more of bank stabilization. The planned structures will generally follow the similar procedures of using riprap on the bank, through the use of training structures, and more rarely, windrows to achieve some measure of improvement in alinement. Since the initial structures were placed on extreme bends, the riprap on these structures are greater in thickness than will be required on later structures. The height of the banks in this area caused by the general degradation of the channel will require somewhat more riprap than was used in the Mohave Valley, and it is expected that the minimum application in Palo Verde will be 2½ to 3 cubic yards per linear foot.

Future Work

The pressing need to conserve as much as possible the available water in the Lower Colorado River Basin requires the continued improvement of the conveyance characteristics and the reduction of the sediment load in the river. Channelization has been authorized in the Cibola Valley but has been delayed, pending the settlement of the Arizona-California State boundary. Studies are presently being made for requirements of channel corrective work in the Parker valley and below Imperial Dam. Bank stabilization will be an important feature of the channelization work in all of these areas because of the nature of the alluvium that forms the bed and banks of the stream.

In Cibola Valley (between Taylors Ferry and Adobe Ruin), the channelization will consist of dredging a new channel about 16 miles in length. The new channel encompasses a fairly stable reach of the river in which there is a transition from degrading tendencies at the upper end to aggrading tendencies at the lower end. Despite the general stability of the bed, it will be necessary to protect the banks of the new channel to prevent enlargement of the channel width. The stabilization will be accomplished in a manner similar to that used in the Mohave Valley, except that in this case it is undesirable to permit as much bank material to erode away before the riprap is launched. Consequently, the channel will be dredged to as near completed width as possible; perhaps even to the extent of partially launching the rock.

Continued activities in the Palo Verde area will extend bank protection to more of the active sediment-contributing banks in this area. Although this work will in general follow the procedures presently in use, the requirement for improving river alinement where possible will no doubt require some revisions and additions.

Conclusions

During the long period in which the Bureau of Reclamation has been engaged in bank and channel stabilization activities on the Lower Colorado River, many methods of stabilization have been tried with varying degrees of success. From this experience the most effective type of bank protection for the conditions encountered on this river has been determined to be quarry-run rock riprap. This material is especially suited to the stabilization of new banks created by channelization and raw banks caused by bank erosion.

New and economical procedures have been developed for placing this type of riprap, which results in neat appearance and low maintenance of the bank line. However, the use of these methods, particularly the use of windrows, require that the bank material be noncohesive and readily erodible in order to launch the riprap properly.

DESIGN OF STABLE CHANNELS IN ERODIBLE MATERIALS

[Paper No. 43]

By PAUL D. DOUBT, *head, Design Section, Engineering Division, Soil Conservation Service*

The methods of permissible velocities, regime theory, and tractive force theory have been applied with various degrees of intensity for the design and maintenance of stable channels. The first two methods are empirical while the tractive force theory is analytical. Some empiricism has been used in the application of the tractive force theory that has unfortunately led to the questioning of the validity of this theory. This paper is given with the hope that it will clarify the logical application of these hydraulic principles and reduce the empiricism used in applying the tractive force theory, by the discussion of three premises:

First, the tractive force theory is logically a dynamic theory;

Second, the water surface profile must be computed to evaluate the tractive forces;

Third, the tractive force depends on the actual depth of flow and on the *roughness coefficient* of the *earth* materials in the wetted perimeter.

The design of an earth channel consists of the following three facets: (1) capacity, (2) stability, (3) economical proportions of the channel and its associated structures.

Hydrological or delivery studies determine the maximum discharge for a channel. This maximum discharge is the design discharge used for determining the size of the channel. A channel is first designed for size to satisfy the capacity requirement. The second phase of design checks the channel for stability and, if necessary, alters the design. The choice of the economical proportions for the channel and its associated structures is coupled with the designs for capacity and stability. Thus, the design of a stabilized channel often requires trial solutions.

In this presentation only the facet of stability design is considered. A channel is considered stable if, for all discharges equal to or less than the design discharge along with their associated sediment loads, the channel banks will not erode or silt and its beds will not degrade or aggrade beyond the designed limits.

Channels are classified into two major groups according to the functions for which they are designed. The procedure for determining the stability in both groups is the same.

One of these major groups consists of channels designed to convey only clear water. The assumption that there is no aggradation of channel bed or sides is used in the design of this group of channels. Thus, no encroachment on channel capacity occurs. The channel is designed to prevent degradation of channel bed and sides.

The other group of channels is designed to convey clear water and to convey the sediment-water complex. These channels are designed to meet their capacity requirements for the maximum aggraded condition. They are designed to meet their stability requirements for the conditions of maximum degradation and clear water flow. These channels usually require stabilizing structures.

Understanding the ways that a grain of sediment is transported facilitates the establishment of the various laws of sediment movement. Sediment is transported by water in three ways. One of these ways is the combination of the other two. The behavior of a particle of sediment can be observed in a model flume having a noncohesive sand and gravel bed. As the velocity of flow increases from zero, the pebble or grain begins to vibrate or oscillate in its rest position. Transportation of a particle takes place in the following sequence:

1. Increasing the velocity of the water causes the pebble to be dragged or rolled from its rest position along the bed in *traction.*
2. A greater velocity causes the grain to jump or skip along the bed. It is called movement in *saltation.*
3. A still greater velocity causes the grain to be carried in *suspension.*

The particles carried in saltation are transported alternately in traction and suspension.

The sediment load is often divided into two groups according to the way sediment is transported past a section.

1. The *bedload* is the rate of sediment transportation by traction.
2. The *suspended load* is the rate of sediment transportation by suspension.

Since it is difficult to measure the rate of the sediment transportation by saltation, such sediment is included in either suspended load or bedload.

The ratio of bedload to suspended load varies widely at different locations and for different stages. The bedload is usually a small percentage of the sediment load, but it is as important as the suspended load. It is the bedload that is chiefly deposited in shoals and on the beds of deltas and alluvial fans at the mouths of rivers.

Tractive force is the force created by the movement of the water over the wetted perim-

eter of the channel and causes sediment movement in traction. It acts in the direction of flow. This force acts over the area of the earth-water interface rather than on the individual particles.

An upward force component created by the motion of a complex current causes transport in suspension.

The design of a stable channel requires the determination of the location of the *evaluative* sections for each kind of material in which the evaluation is made of (1) the maximum actual tractive force for the bed and the sides; and (2) allowable tractive force for each kind of material in the bed and the sides.

The location of evaluative sections for each kind of materials according to their physical properties is determined from the water surface profile. The determination of the location and definition of evaluative sections will be discussed later.

The value of the allowable tractive force (safety factor = 1) is the maximum tractive force that the materials can safely withstand before erosion occurs. In designs the safety factor should be greater than 1.

Determination of Tractive Forces

The hypotheses that sediment movement of sands and gravels in traction is caused by the frictional forces at the boundary is well substantiated by experimentation and observations.

Manning's roughness-coefficient n_t is used to evaluate the frictional force of the earth material in the wetted perimeter of the channel. The relation suggested by Lane [1] is

$$n_t = \frac{D_{75}^{1/6}}{39} \qquad (1)$$

where D_{75} = the size (inches) of the earth materials in the wetted perimeter of which 75 percent (by weight) is smaller.

Manning's roughness-coefficient n_t must not be confused with Manning's coefficient n. The coefficient n is used to evaluate energy losses caused by friction, bends, vegetation, and eddies in a channel conveying water. It is used to determine the water surface profile and is never less than Manning's roughness-coefficient n_t.

The evaluation of the total tractive force acting on the peripheral materials of the channel is a dynamic problem and not a static problem. Thus, tractive force is evaluated by determining the frictional force that is caused by roughness of the earth materials in the wetted perimeter and that produces the energy loss at the earth-water interface. The energy loss at the interface is easily evaluated, since the rate of energy loss caused by the earth materials in the wetted perimeter is known to be

$$s_t = \frac{n_t^2 Q^2}{(1.486)^2 a^2 r^{4/3}} \qquad (2)$$

where

s_t = rate of energy loss at the interface of the earth material and the flowing body of water, foot-pounds per pound of water per foot length of channel;

Q = actual discharge, cubic feet per second;

a = actual flow area, square feet;

r = actual hydraulic radius, feet;

n_t = Manning's roughness-coefficient for the earth material in the wetted perimeter.

The weight of the body of water that flows past a section in the time interval t is $\gamma_w Q t$ pounds where $\gamma_w = 62.4$ = unit weight of water, pounds per cubic foot.

t = time interval, seconds.

The total work done W is the energy loss at the interface. It is the product of s_t and the weight of the body of water, or

$$W = s_t \gamma_w Q t \qquad (3)$$

where W = work done, foot-pounds per foot length of channel.

The total frictional force F that produces this energy loss is obtained from the relation

$$W = FD = F\frac{Q}{a}t \qquad (4)$$

where F = the total frictional force, pounds per foot length of channel.

$D = \frac{Q}{a}t$ = the distance the body of water is moved in the time t, i.e., the distance the force F acts, feet.

Equate equations 3 and 4 and obtain

$$F\frac{Q}{a}t = s_t \gamma_w Q t \quad \text{or}$$

$$F = \gamma_w s_t a \qquad (5)$$

The average unit frictional force on the wetted perimeter is obtained by dividing the total frictional force by the wetted perimeter p or

$$f = \gamma_w s_t r \qquad (6)$$

The unit frictional forces vary in magnitude along the wetted perimeter (fig. 1). They are equal to the unit tractive forces but are oppositely directed.

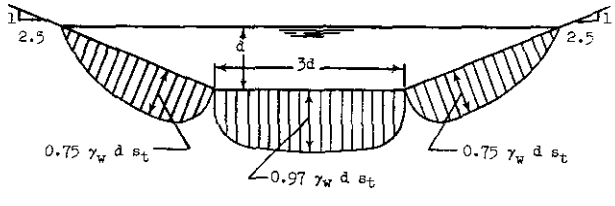


FIGURE 1. — Distribution of actual tractive forces in a trapezoidal channel.

[1] LANE, EMORY W. DESIGN OF STABLE CHANNELS. Amer. Soc. Civil Engin. Trans. 120. 1955.

The maximum tractive force in a trapezoidal section is generally obtained by relating it to the tractive force of an infinitely wide channel. Certain assumptions along with the relation given in equation 6 are used to obtain the tractive force of an infinitely wide channel.

To evaluate tractive forces in an infinitely wide channel of homogeneous earth material, assumptions are made that all energy losses (1) occur on the bed, and (2) are distributed uniformly over its bed.

The first assumption is reasonable for subcritical flow. The second assumption is applicable because the bed of an infinitely wide channel has a uniform distribution of tractive forces. The unit discharge q in an infinitely wide channel has a cross sectional flow area equal to the depth of flow d. Equation 6 becomes

$$f=T_\infty=\gamma_w\, d\, s_t \qquad (7)$$

where

T_∞=tractive force on the bed of an infinitely wide channel, pounds per square foot;
d=actual depth of flow, feet;
s_t=rate of friction loss resulting from flow over the bed materials.

The relations of the maximum actual tractive force on the bed and sides of a trapezoidal section to the tractive force on the bed of an infinitely wide section are based on experimental data and analysis.[2]

The maximum actual tractive force is the maximum tractive force for the evaluative section. The locations of evaluative sections are determined for each earth material in the channel bed and sides. For prismatic channels, the evaluative section is the section having the smallest depth of flow. This occurs at the farthest downstream section in a reach of accelerated flows and the farthest upstream section in a reach of retarded flow. In general, the evaluative section is that section having the largest value of $\frac{d}{K^2_d}=\frac{d\, n_t^2}{(1.486)^2\, a^2\, r^{4/3}}$.

The location of the evaluative section depends on the hydraulic characteristics of the channel and water surface profile computations are required.

The parameter d is evaluated by water surface profile determinations. This value is obtained by use of Manning's coefficient n, which is frequently greater than Manning's roughness-coefficient n_t corresponding to the earth materials in the wetted perimeter of the channel.

The evaluation of s_t at a section in a reach is

$$s_t=\frac{n_t^2\, Q^2}{(1.486)^2\, a^2\, r^{4/3}}$$

The relation of the maximum actual tractive force in natural channels to infinitely wide channel can often be estimated by approximating the actual section with a trapezoidal section.

The actual depth of flow d and the value of s_t associated with the natural channel are used to determine the tractive force T_∞ in the corresponding infinitely wide channel

$$T_\infty=\gamma_w\, d\, s_t \qquad (7)$$

Values of maximum tractive forces for channels with curves are based on judgment coupled with very limited experimental data.[3] [4] Channel curves cause the maximum tractive force to increase. The maximum tractive forces, in a channel with a single curve, occur on the inside bank at the upstream end of the curve and near the outer bank downstream from the curve. Compound curves further increase the maximum tractive forces.

Allowable Tractive Force T_{ab}

For noncohesive materials [2] of gradation D_{75},

$$T_{ab}=0.4\, D_{75} \ldots .0.25 \leqq D_{75} \leqq 5.0 \qquad (8)$$

The allowable tractive force for channel sides T_{as} is less than that of the same materials in the bed of the channel, because the gravity force aids the tractive force in moving the materials. The allowable tractive force for channel sides composed of noncohesive granular materials,[2] on considering the effect of the side slope z and the angle of repose with the horizontal Φ_R, is

$$T_{as}=0.4\, K\, D_{75} \ldots . 0.25 \leqq D_{75} \leqq 5.0$$

where

$$K=\sqrt{\frac{z^2-\cot^2\Phi_R}{1+z^2}} \qquad (9)$$

Thus, the ratio of T_{as}/T_{ab} is K.

When the unit weight γ_s of the constituents of the material having a grain size larger than D_{75} is significantly different from 160 lb./ft.^{3}, the allowable tractive forces T_{ab} and T_{as} are to be multiplied by the factor Γ

$$\Gamma=\frac{\gamma_s-\gamma_w}{97.6} \qquad (10)$$

where γ_s=unit weight of the constituents of material with a grain size greater than D_{75}, lb./ft.^{3}

γ_w=62.4=unit weight of water, pounds per cubic foot.

The evaluation of allowable tractive forces

[2] BUREAU OF RECLAMATION. PROGRESS REPORT ON RESULTS OF STUDIES ON DESIGN OF STABLE CHANNELS. Hydraul. Lab. Rpt. HYD-352, 36 pp., illus. 1952. Denver, Colo.

[3] See footnote 1.

[4] NECE, R. E., GIVLER, C. A., and DRINKER, P. A. MEASUREMENT OF BOUNDARY SHEAR STRESS IN AN OPEN CHANNEL CURVE WITH A SURFACE PITOT TUBE. Mass. Inst. Technol. Tech. Note 6. 1959.

for cohesive[5 6] materials has not been as successful as the evaluation of the allowable tractive forces for sands and gravels. There is some doubt regarding the parameters required to define the allowable tractive force of cohesive soils. If the tractive force principle is applicable, it appears that the plasticity index, void ratio, percent clay, along with other possible parameters, may enter into the relation. The time required for degradation of highly plastic soils is indeterminable, because of the scarcity of reliable information.

Existing natural channels show that, as the plasticity index for materials in the channel sides is increased, the allowable steepness of channel sides is increased, provided the channel bed is not degrading.

The allowable tractive force for soils within the range of plasticity index $I_p < 10.0$ and $D_{75} < 0.25$ are difficult to determine. Materials of this region are extremely vulnerable to erosive forces. Channel banks or beds composed of this type of materials will usually require protection. Their design without revetment, lining, or structures is generally impossible.

[5] DUNN, I. S. TRACTIVE RESISTANCE OF COHESIVE CHANNELS. Jour. Soil Mech. and Foundations Paper 2062. 1959.

[6] SMERDON, E. T., and BEASLEY, R. P. CRITICAL TRACTIVE FORCES IN COHESIVE SOILS. Agr. Engin. 42, 1961.

CRITERIA AND METHODS FOR USE OF CHECK DAMS IN STABILIZING CHANNEL BANKS AND BEDS

[Paper No. 44]

By W. R. FERRELL and W. R. BARR, *engineers, Los Angeles County Flood Control District*

Introduction

Erosion of our mountain masses is a very slow process and goes on practically unnoticed in many parts of the United States. However, in regions where there is a high demand for new lands suitable for development and other use by man, the process becomes of prime concern. The importance of stream erosion is amplified by the fact that the areas most directly affected, such as the alluvial fans at canyon mouths, provide not only the room for expansion but also actually are considered as premium building sites.

Although the effects of mountain erosion are now becoming of greater concern in this country, particularly in southern California, such erosion has been recognized and dealt with in the Alpine regions of Europe for centuries. In Europe, many applications of channel stabilization principles were developed and placed into practice with strong reliance on hand labor and the use of on-site materials. In 1954 the Los Angeles County Flood Control District and the U.S. Forest Service in Los Angeles County undertook what probably was the most concerted effort ever made in this country to determine the most effective and efficient means of reducing mountain channel erosion, using modern materials and techniques. A comprehensive report of the findings of this study, which included a review of preceding practices in Europe and the United States, was published by the Flood Control District in 1959 under the title "Report on Debris Reduction Studies for Mountain Watersheds." This study was greatly facilitated by research work conducted in this area by the U.S. Forest Service in the 1940's under the Los Angeles River Watershed "Memorandum of Understanding." The plan set forth in the report provides for a $20,000,000 stabilization program that is currently under way. As the work progresses, the design and construction techniques established in the report are continually updated as experience dictates.

Evidence of Need for Stabilization

In various parts of the United States the need for bank and bed stabilization may vary, depending upon physiography, land use, and economic justification. In the semiarid Southwest, and especially the coastal plains and valleys of southern California, alluvial fans below the canyon mouths have aggraded for centuries, fed by sheet erosion and channel downcutting with its accompanying bank sloughing within the mountains. The complete occupancy of these fans below the smaller frontal canyons has necessitated a comprehensive system of debris dams, debris basins, and lined channels to desilt mountain streams and convey the desilted flows to spreading basins or to the ocean.

Trapping silt and debris in the basin is only part of the job. These erosion products must then be removed and disposed of, and along with the excavation and hauling costs comes the problem and expense of locating and purchasing disposal sites. The latter must also be located near the canyon mouths to reduce haul distances.

In the larger canyons, reservoirs built and maintained for the purpose of flood protection and water conservation become silted to the point where storage necessary for flood regulation or worthwhile conservation activities becomes inadequate. Desilting is an expensive and

frustrating operation that involves sluicing and subsequent removal from downstream works or mechanical excavation and disposal similar to that of a debris basin. It can readily be seen that competition for available land in the foothill zone is entered into by both public agencies and private citizens.

Primarily as a result of a medium-sized storm in the 1961–62 storm season, the Los Angeles County Flood Control District excavated 1,235,600 cubic yards of debris from its debris basins at a cost of $1,044,600, or about $0.85 per yard, which does not include various extras such as disposal site costs. In actuality the total costs are currently running close to $1.50 per yard. The great increase in cost is manifest when compared to the 1938 cost of $0.33 per yard. Figure 1 shows Puddingstone diversion dam before and after 328,000 cubic yards of debris had been removed. During the same storm the storage loss in 11 of the District's major flood-regulating and water conservation reservoirs was computed to be 1,720 acre-feet (2,768,500 cubic yards). These figures would be increased manyfold for a capital storm. The need for stabilization to reduce erosion in southern California, therefore, is based primarily on economic considerations.

Design of Stabilization System

Through experience and experimentation a procedure has been developed that reduces to a minimum the time required to prepare plans for a stabilization system. The planning consists of three basic phases: field engineering, office engineering, and economic analysis. The field engineering phase is very important in the development of a stabilization system because every effort must be made to utilize desirable features provided by nature. The engineer is guided by two basic rules in the utilization of these natural features; these are (1) the entire channel reach being considered must be stabilized, either by existing natural conditions such as exposed bedrock, natural dams, etc., or by the stabilization system proposed; and (2) the structures should be so located that the spillway height above natural streambed does not exceed 17 feet nor is less than 10 feet. This rule is based on economic considerations of structure design, as will be discussed later. The importance of keeping the proper balance between site conditions, continuous stabilization, and desirable structure height cannot be overemphasized. The distance upstream that stabilization can economically be justified must be estimated for the initial layout. In southern California it has been found that this economic limit is reached when the channel gradient approaches 20 percent; however, in special cases a 25 percent limit may be permissible. Other considerations may modify the termination point, such as access road costs.

The definition of a stabilized reach as used above is that state of channel regime at which all downcutting is halted and bank erosion is reduced to an absolute minimum. To accomplish this requires that the successive stabilization structures be so placed that the debris impounded by one will create an artificial gradient that terminates at the toe of the next structure upstream (fig. 2). This gradient has been found to be approximately 0.7 of the natural gradient and is the figure used for design and layout purposes. Figure 2 is a cutaway sketch showing this stabilized condition.

Experience has shown that accurate topographic maps with a 2-foot contour interval, including all terrain lying within 30 feet vertically of the channel bottom, are highly desirable for use in connection with the field layout, and a precise profile along the streamline itself

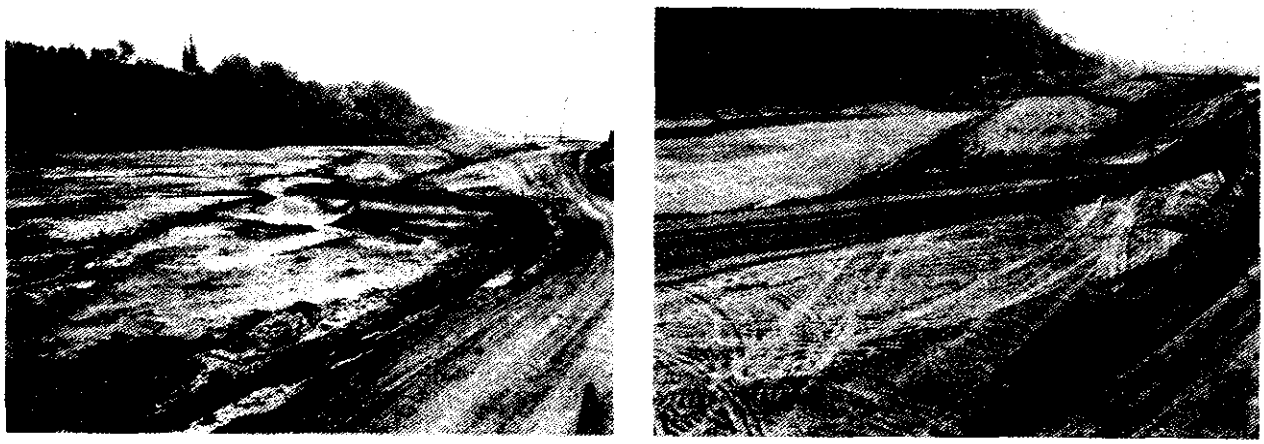


FIGURE 1. — Puddingstone diversion dam after 1961-62 storm season (*A*) and after debris was removed (*B*).

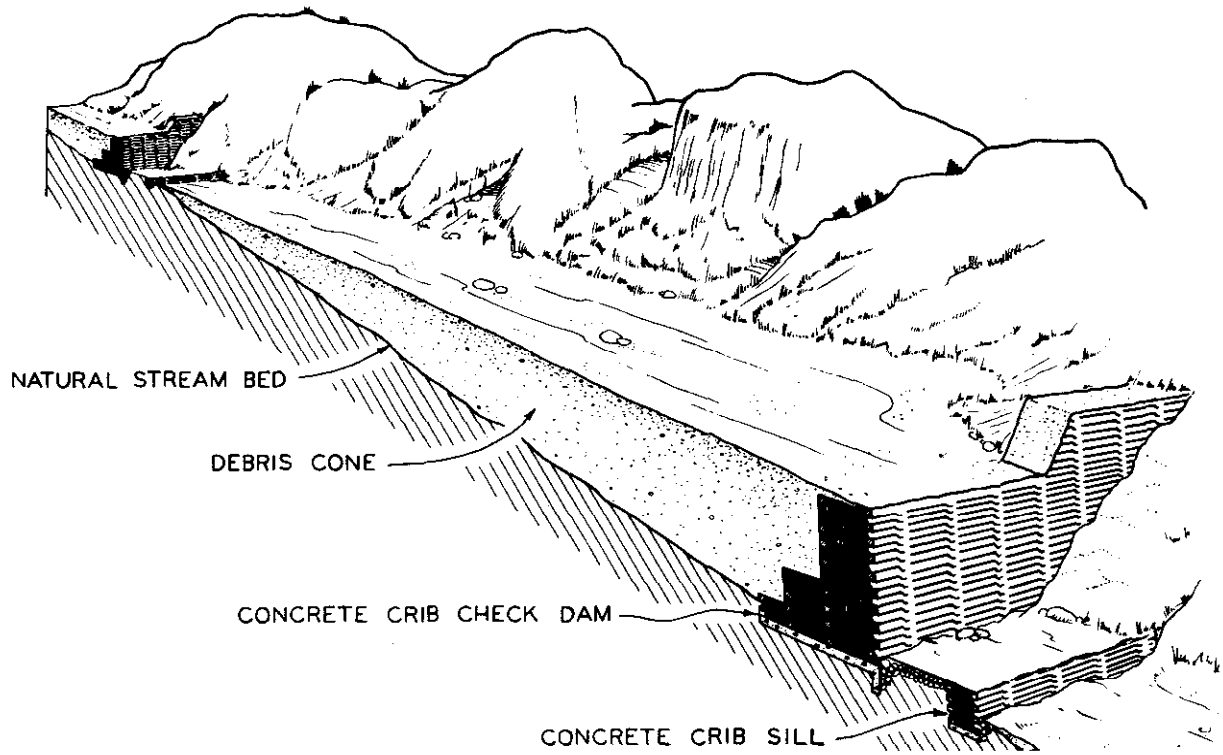


FIGURE 2. — Cutaway view of channel stabilization system.

is necessary. The use of topographic maps based on aerial photographs as aids in planning has shown poor results, especially where vegetation covers the canyon bottom at the time of the photograph. Small falls of 5 to 15 feet in height, critical in the design of a channel stabilization system, are usually unaccounted for in this process. After tentative field location of all structures has been completed on the basis of the foregoing, the office engineering phase commences. Accurate topographic maps enable the office engineer to locate the best site for the structures, to establish required rights of way, and to determine project benefits.

A detailed study of various types of construction materials that have been used extensively for stabilization structures, and some new ones yet untried, was made as a part of the previously mentioned comprehensive study published in 1959. On the basis of cost, required height, ease of construction, availability of backfill material, access possibilities, and maintenance, certain types of construction materials were found to be particularly well suited for specific situations. The materials, in order of general preference, are concrete crib, metal bin, rubble masonry, and such miscellaneous materials as corrugated metal arch, timber crib, and soil cement. Study of the site, access, and size of watershed will usually indicate the need for a structure of either concrete crib or metal bin (fig. 3). Concrete crib is used at all locations where access is not a prime consideration, as it offers several additional and desirable features in the form of structural stability, longevity, and construction flexibility. The metal bin type is often used in regions where access is difficult and expensive, and airborne operations are indicated.

Important features in the design of concrete crib structures include the basic concept of a stable, yet most economical unit possible for a given height of structure and design depth of flow over the spillway. To insure safety against the most common cause of check dam failure that occurred in the past, namely, undercutting at the toe and abutments, all foundations not on bedrock are designed as continuous reinforced concrete spread footings. A cutoff wall is provided at the downstream edge to protect against undercutting and to eliminate foundation piping by insuring a hydraulic gradient of less than unity at all points under the structure.

Additional safety against erosion at the toe from the pounding of desilted flows passing over the spillway is provided in the form of a transverse sill placed downstream at a distance equal to the spillway height plus 5 feet, with the in-

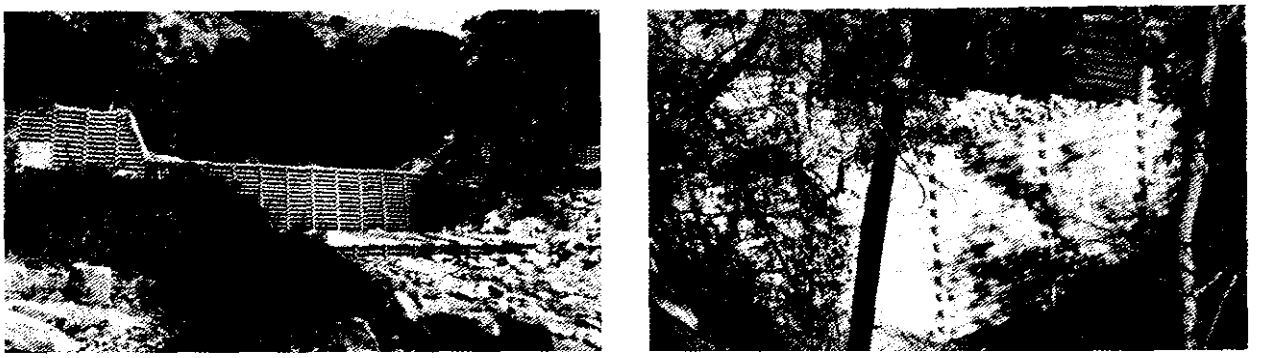


FIGURE 3. — Typical stabilization structures: Left, concrete crib; and right, metal bin.

terspace filled with boulders. These sills become largely inoperative once the debris cone from the structure downstream reaches design elevation. Spillways are gunited to provide a tough wearing-surface against the abrasive action of sand and gravel, and a gunited low-flow chute is provided down the face of the structure. The abutments are stepped as the canyon sidewalls permit, based on the module of the cribbing (6 feet). Economic analysis of various structure sizes has indicated that the structure height should not be less than 10 feet from streambed to spillway, or foundation costs will comprise too great a part of the total. The maximum height is generally limited at 17 feet by the allowable bearing value of concrete at the "column points" formed by the intersection of crib members where structure loads are transmitted to the foundation. For higher structures these "column point" areas must be increased by the use of double crib members or by special oversized members. This has been found too costly with respect to additional benefits, except in special cases. Figure 4 shows a typical concrete crib structure.

The use of metal bins for stabilization structures has involved the same careful approach, but with extra precautions taken to insure sliding stability (fig. 5). This is necessary, because metal bin structures are placed at existing streambed grade with no possibility of keying action such as is provided by the cutoff wall in the case of the concrete crib. The metal bin type construction is not subject to a rigorous analysis. Therefore, to solve unanswered design questions, a special test structure packed with pressure gages, piezometers, and strain gages was constructed at a typical canyon site in cooperation with the Metal Products Division of Armco Steel Corp. The structure was loaded to the theoretical point of failure and observations of settlement, horizontal movement, and structure stress and strain were made during the loading program. This research has provided assurance that the design criteria developed for metal bin dams will provide the utmost in economy but with no sacrifice in stability.

If the particular area to be stabilized is within a very small headwater area or a small tributary branch to a main channel, such materials as soil cement, timber crib, or corrugated metal arch may be found desirable. Each reach of channel requires analysis in all respects, with the appropriate choice of structure type based on the above criteria. Design manuals, which include nomographs and charts, have been developed to facilitate a full design of each structure. (See fig. 6 for an illustration of rapid design procedure.) After the design is completed, the customary computation of volumes and quantities of materials is made, followed by the cost estimate. Added at this time must be the cost of delivering materials to the site, namely, the provision of access, the cost of which must be distributed appropriately over the structures served. In many watersheds the presence of high falls, narrow canyon passages, or long reaches of stable channel will necessitate a change from concrete crib construction, where access roads are used for delivery of equipment and materials, to metal bin structures which utilize helicopters for this purpose. Taken into account must be the cost of building access roads as opposed to that of airlifting materials to the site. Also, hillside road construction can sometimes create more erosion products than are eliminated by the planned stabilization system.

Right-of-way acquisition is left for consideration at this time because of the several factors that affect the boundaries of lands required. It has been this flood control district's experience that property owners often are willing to cooperate after they realize that the stabilization of channels through their lands will work to their advantage and access roads will in many cases be open to them. Easements for construction

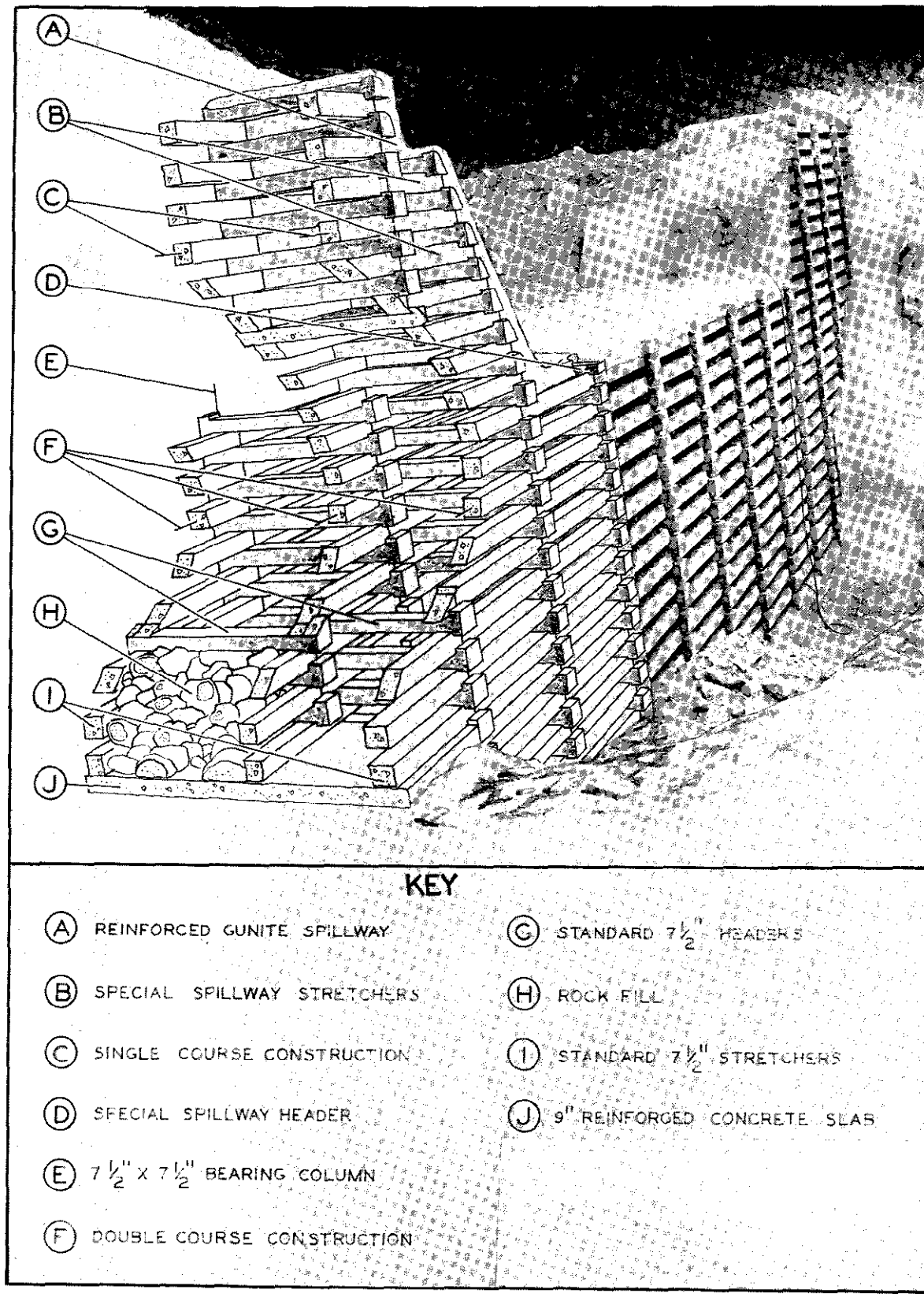
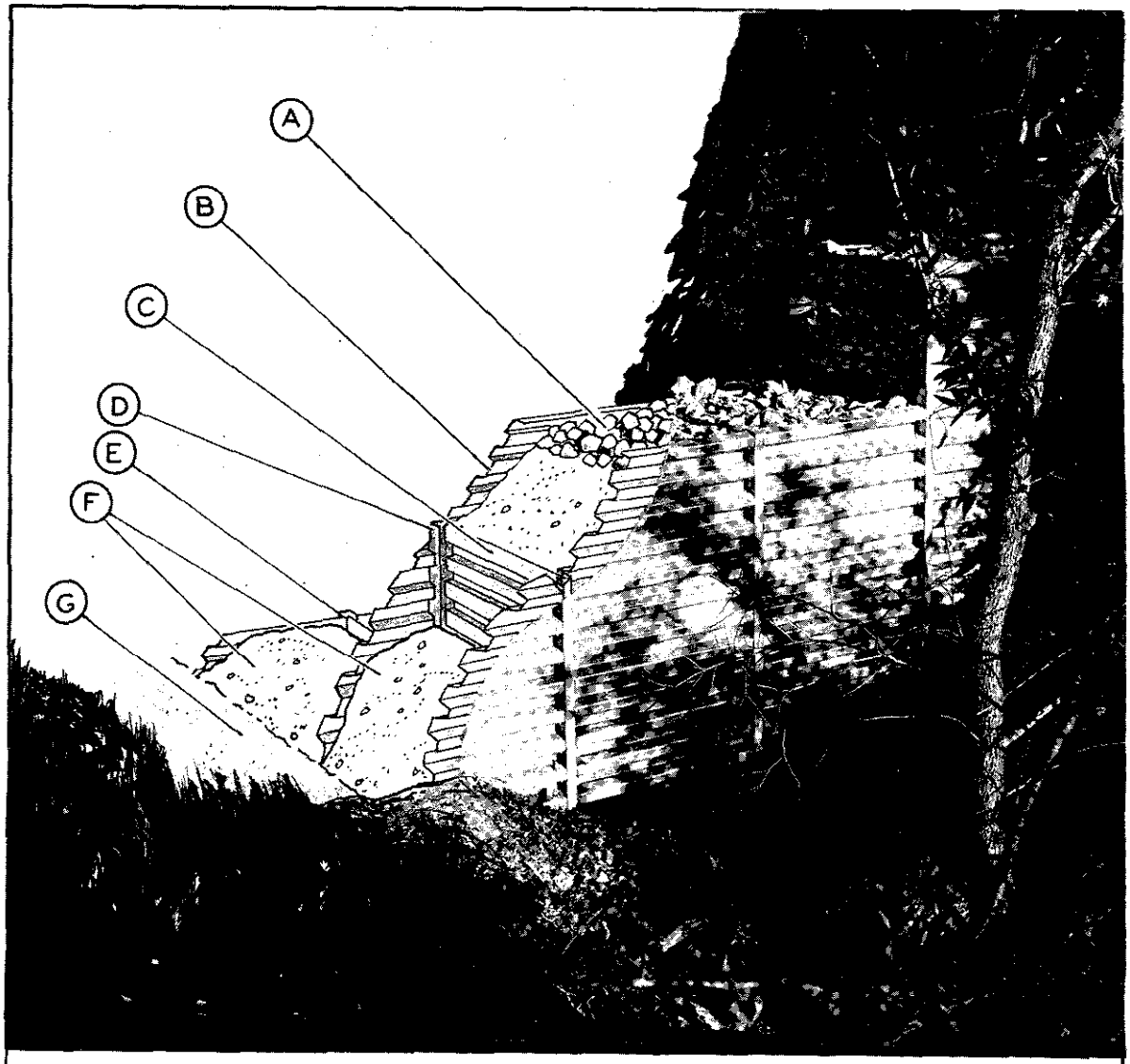


FIGURE 4. — Combination photograph and cutaway drawing of a concrete crib structure designed for the Cooks Canyon project.



KEY

(A) ROCK SPILLWAY SURFACE

(B) STRINGER

(C) SPACER

(D) CONNECTOR

(E) REAR BIN (FOR ADDED STABILITY)

(F) COMPACTED BIN FILL

(G) ORIGINAL STREAM BED

FIGURE 5. — Combination photograph and cutaway drawing of a typical metal bin structure.

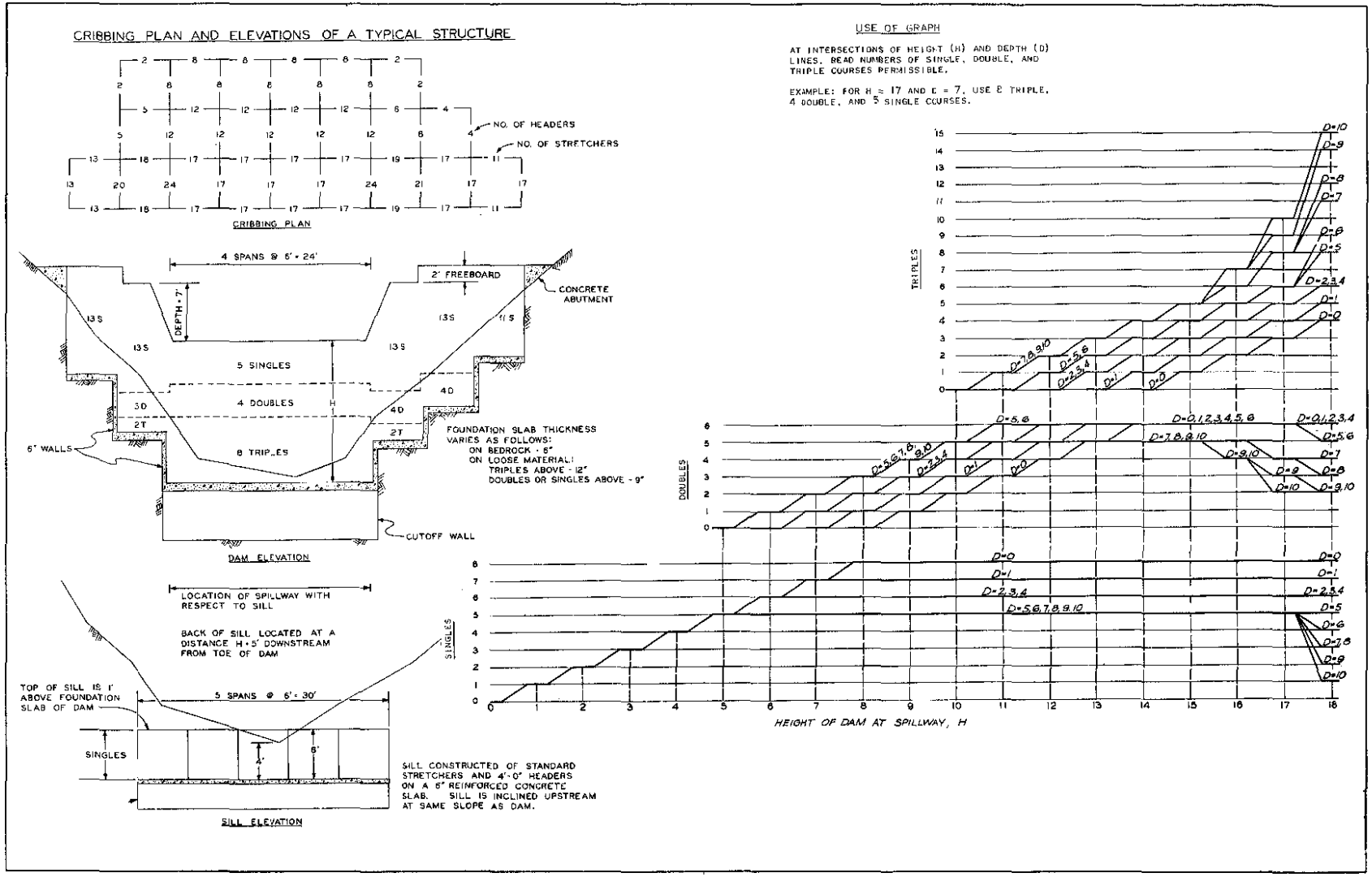


FIGURE 6. — Cribbing plan, elevations, and overflow gravity section design for typical stabilization structures.

and operation and maintenance of structures within their lands are often obtained at little or no cost. However, since some expenditures are likely to be incurred in connection with right-of-way, this item must be considered in the total cost estimate. The topographic map of the channel bottom area discussed previously is used to define the limits of right-of-way needed. The general criterion is to acquire easements or fee title if necessary for the channel bottom and the side slopes to a distance of 5 feet above the computed high-water line. In some cases right-of-way must also be obtained for access roads.

The final step in the design phase is the economic analysis. An evaluation of the benefits realized from the installation of a stabilization system are needed for a determination of the benefit-cost ratio. All benefits are not readily apparent, and experience has shown that further research is necessary (and is being conducted) to provide the parameters necessary for their conversion to dollar values. Those benefits that can be converted are:

(1) The value of the new debris storage capacity available above existing streambed behind each of the structures of the system. The debris that will be impounded results mainly from sheet erosion and small side canyon flushing, since channel downcutting and its associated bank sloughing will have been halted.

(2) The net reduction in erosion from the watershed. This is the long-time reduction that is achieved by virtue of the halting of channel downcutting and associated bank erosion. This benefit has been very difficult to evaluate, because a progressive reaction takes place opposite to the process of accelerated erosion; that is, as the channel bottom stabilizes and aggrades, the side-slope scars are able to stabilize at the toe, thus allowing native vegetation to re-establish and eventually heal the scar area completely.

(3) The value of water conservation. Water is conserved as a result of the recurrent filling during storms of the voids within the porous debris deposits behind each structure and their subsequent draining during dry periods. This amounts to a considerable volume of temporary storage for water that otherwise might tax downstream conservation facilities during storm periods and thus be lost for conservation use. Studies have shown the voids available for this temporary storing of water will average around 35 percent of the total volume.

The monetary value of the benefits accruing is ascertained on the basis of (1) unit costs to remove debris from downstream facilities in order that they remain operable; and (2) the cost to the district of purchasing water from outside sources for ground water replenishment. The present figures being used for debris removal and purchased water are $1.50 per cubic yard and $14.00 per acre-foot, respectively.

Among the intangible benefits for which no attempt has been made to arrive at a monetary value are (1) the promotion of vegetation on the stabilized bank slopes that can take place once the halting of side-slope sloughing has been accomplished; and (2) the change in shape and character of the channel bottom, which, as the V-shaped canyons aggrade and become U-shaped, give vegetation an opportunity to establish itself in the channel bottom area. These benefits provide areas for recreational use high in esthetic value.

Once the monetary value of the benefits has been established, the ratio of benefits to cost is determined on the present worth basis, capitalized over a 50-year period. Fifty years is considered to be a conservative period in view of the ruggedness of the materials used. However, experience has proved that an annual maintenance cost will be incurred and therefore must be included in the benefit-cost analysis. A benefit to cost ratio of one or greater is normally considered justification for proceeding with the project. This position is reinforced by the above-discussed intangible benefits that help meet the expanding need for new recreation areas and the protection of existing facilities.

Construction Procedure

In order to make maximum use of both labor and equipment, the construction of stabilization structures must be organized on an assembly line basis. While the structure at one end of a system is in the cleanup stage, site clearing is under way further along the line of progress, and structures in between will be in intermediate stages such as foundation-excavation, foundation-pouring, crib member placement, etc.

Routing over access roads and selection of staging areas should be carefully planned to limit traffic interference with construction. This is especially important since the roads are generally one lane wide, and may often pass over the abutments of partially completed structures.

Experience has shown that the costs of obtaining complete foundation analyses, although required if final structure design drawings are prepared prior to construction, are high and unnecessary. Every effort should be made to work with nature, and this is particularly true at this point where much excavation can be saved by timely decisions in the field during construction as to foundation depth and stepping, with subsequent savings all along the line. To provide this flexibility, it is desirable that a

well-qualified project engineer be on the job, empowered to make such revisions as field conditions indicate.

Once a site has been located on the ground, a system of wires, set across the canyon by a survey crew to mark the positions of the toe and heel of the dam, will greatly facilitate construction. Plumb bobs, marking abutment breaks, the spillway, or other special features, are suspended from these lines, permitting rapid checks during excavation and initial construction. Vertical control is established by temporary bench marks with elevation checks as required.

Equipment should be carefully chosen with respect to the type of structure to be built. When the concrete crib construction is utilized, D–8 or D–9 dozers, often augmented by explosives for site clearing and foundation excavation, are frequently needed. A backhoe is normally used for cutoff walls and minor abutment excavations, whereas a crane of ½- or ¾-yard capacity is necessary for handling heavy crib members and placing concrete.

To conserve labor, forms only for the foundation are specified for most concrete pours, and screeding is done with a specially constructed aluminum rail fitted with handles. Reinforcing steel is bent in prescribed fashion at a staging area before transport to the site.

Fill material for concrete crib structures is specified as clean rock in the downstream bays only, and pit-run material from the channel bottom is used for upstream bays. This procedure is used to reduce settlement and leaching from the downstream bays and also to reduce the total quantity of clean rock required. However, in some canyons the volume of rock needed will still make quarrying necessary, which greatly increases costs and should be kept to a minimum. Figure 7 shows typical construction details, and figure 8 shows three views of construction progress on a typical structure.

Metal bin structures, due to their use in less accessible locations, require relatively small, compact, and often self-contained equipment. Often a good deal of undesirable and expensive hand labor is required at present, but plans to develop highly versatile equipment, transportable by air, are being considered to reduce costs.

Operation and Maintenance

Depending upon the types of structures included in the project, certain repair work will be required periodically, particularly in the vicinity of the spillway. When repairs are made, such special equipment and materials as self-contained hand portable tools and epoxies should be used.

Access routes to the sites, whether trail or road, should be kept passable to assure rapid attention to necessary repairs. Experience has shown that annual inspections are needed after each storm season to assure correction of maintenance problems.

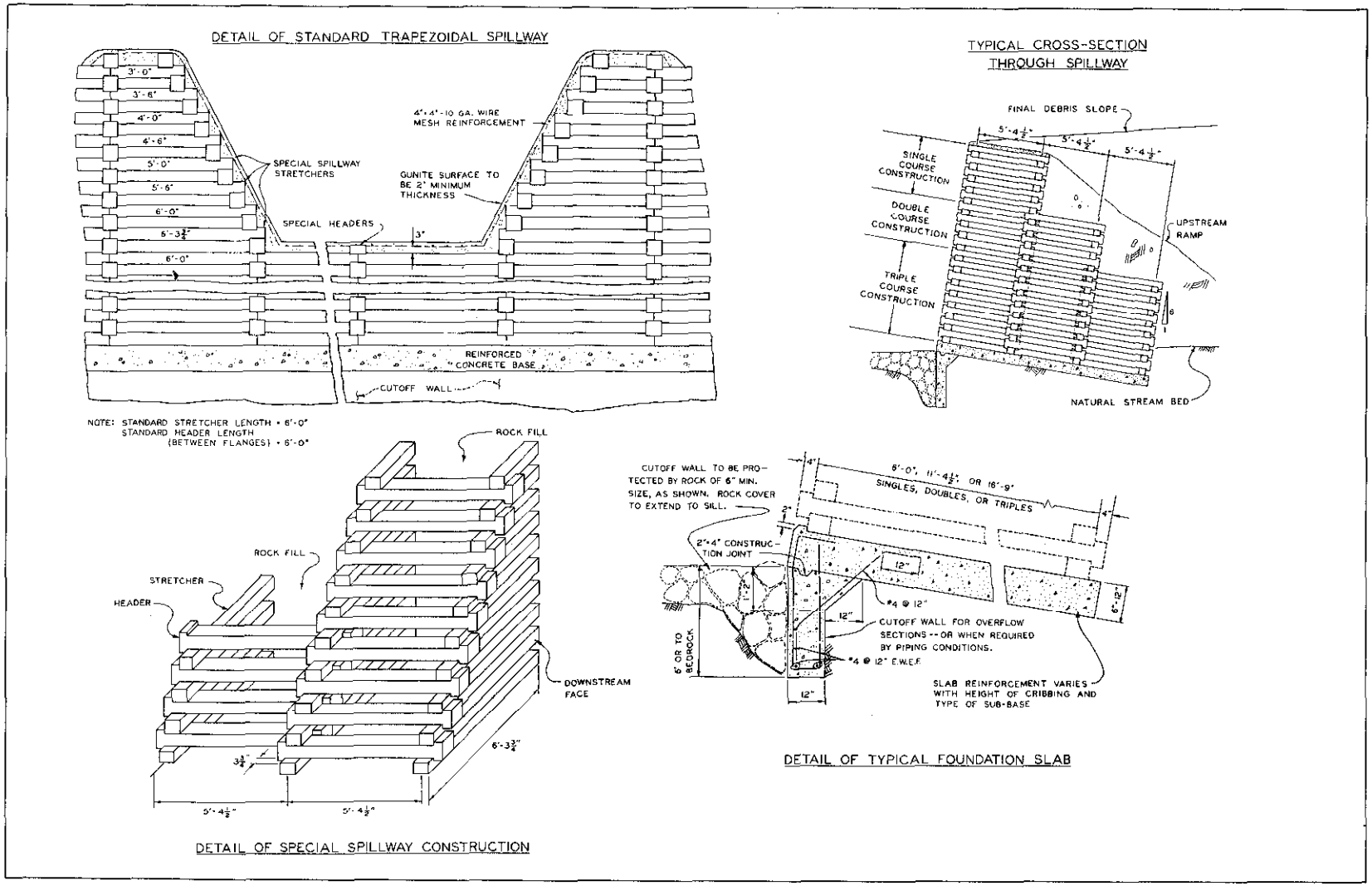


FIGURE 7. — Design and details for concrete crib check dam construction.

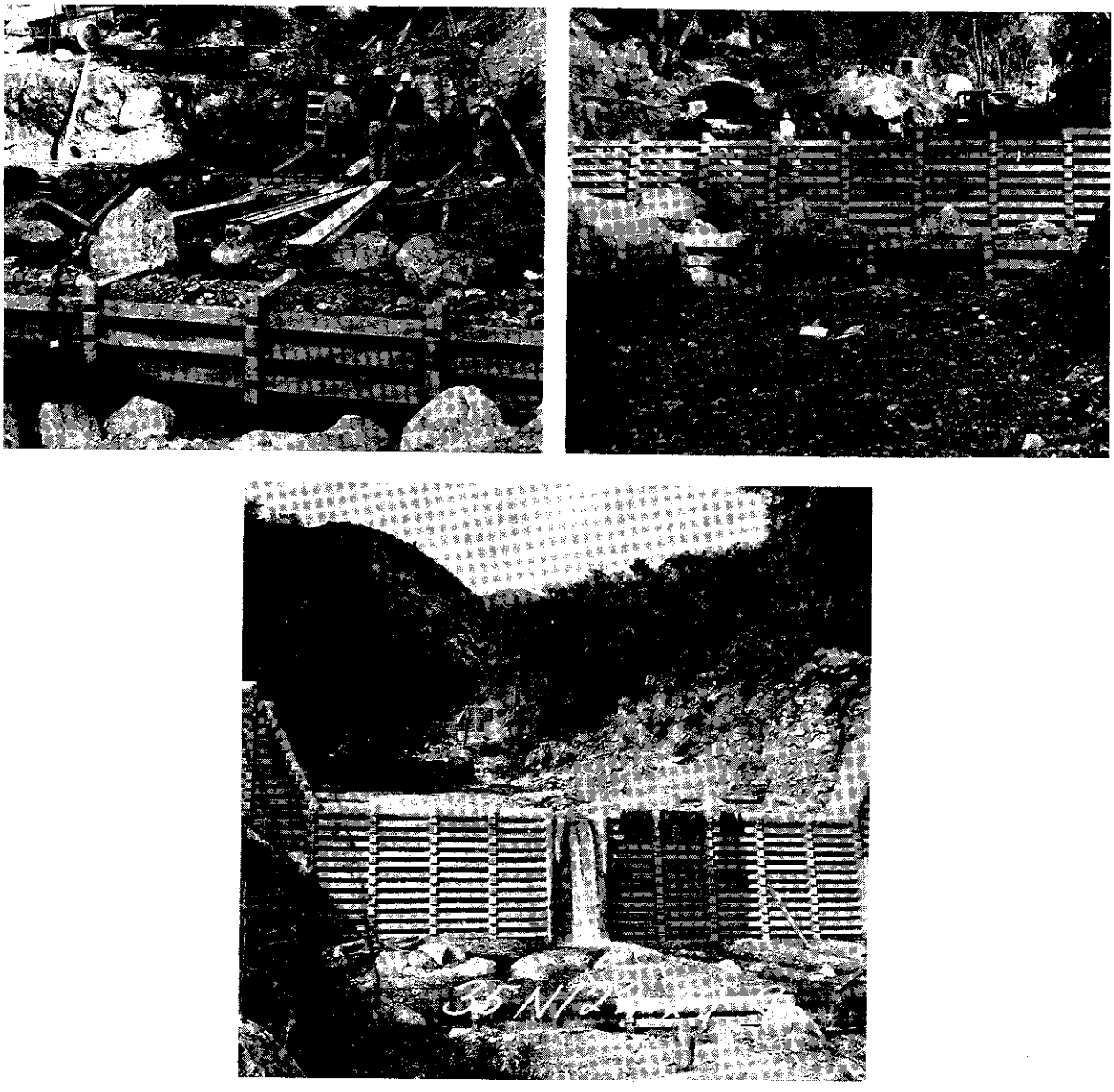


FIGURE 8. — Construction progress on a typical structure.

BANK STABILIZATION OF MISSISSIPPI RIVER BETWEEN THE OHIO AND MISSOURI RIVERS

[Paper No. 45]

By A. J. TIEFENBRUN, *civil engineer, U.S. Army Engineer District, St. Louis*[1]

Synopsis

Regulation, control, and stabilization of channel achieved on an alluvial river are dependent upon the specific demands of navigation and flood control. The earliest plan of improvement in the middle Mississippi consisted of closing dams or solid dikes constructed of brush and stone to confine the flow to a single channel. Experiments with various types led to the adoption of the permeable structure consisting of pile dikes and crib dikes. Although dikes cause deposition and accompanying accretions, in many instances the dike system may induce a heavy attack on the opposite bank downstream. Unless banks are protected, large scale caving will occur. Standard bank protection consists of an underwater part (lumber mattress and stone ballast) and upper bank part (riprap paving). In addition to controlling the trace of the channel at low water, bank protection must be adequate to prevent meandering at high stages. Experimental stone foundation has been used in the repair of revetments where the river attack is mild and the toe of the bank is above or near the low water plane. This type of foundation has proved more stable and less costly than lumber mattress foundation. Sediment from upstream overbank areas and from bank erosion still are factors in regulating river works, but their contribution is diminishing. Indications are that we are now entering a stage in the middle Mississippi where impermeable dikes may be needed. Present methods of river regulation, control, and stabilization have proved effective and at the lowest possible cost. However, new improved methods are constantly being searched for. Improvement over present methods will probably come under the impact of increasing navigation demands and the trend for greater utilization of flood plain areas.

Bank Stabilization

The methods and degree of regulation, control, and stabilization of channel achieved on an alluvial river are dependent upon the specific demands for navigation and flood control during a particular era. The outstanding example of progressive degrees and methods of stabilization for an alluvial river is the Mississippi. Here, the bank stabilization program dates back well over a century. In order to understand the nature of the land through which this river flows, the erosive effect of the river current action on its banks, and the natural changes in the river's regimen, a brief description is given of the more important physical characteristics of its basin.

The Mississippi River drains an area of 1,240,000 square miles and extends from the Appalachian Range on the east to the Rocky Mountains on the west and from the lower St. Lawrence watersheds on the north to the Gulf of Mexico on the south. For general consideration, the Mississippi may be divided into three major reaches or sections — the upper, middle, and lower Mississippi. The river above the mouth of the Missouri is commonly called the upper Mississippi, and from the mouth of the Ohio to the Passes the lower Mississippi, the reach between the two being referred to as the middle Mississippi. The several reaches present an almost endless diversity in topography and geology.

The evolutionary life of the Mississippi River is closely associated in geologic history with the Glacial Age in North America. The Continental Ice Cap in its extreme advances reached as far south as the Ohio River, once considered to be the mouth of the Mississippi. For a time, before the Gorge of Niagara opened to pass the waters of the Great Lakes, drainage of these lakes reached the Mississippi through the Illinois River. In a similar way, drainage in southern Canada poured into the Mississippi valley until the ice had receded to the Hudson Bay and drainage to the north was then possible. As a result of these geological actions and this period of glacial drainage, the bed of the Mississippi River was worn down by the erosive action of a much greater volume of water than that of present day ordinary flow. At present, the river follows a winding course between low banks in a very wide flood plain bordered by high bluffs of sedimentary rock.

My detailed observations and discussion will be confined to that reach of the Mississippi between the Ohio and Missouri Rivers. In the middle Mississippi, the distance between bluffs averages about 5 miles, except in the vicinity of

[1] The author obtained some of his information from papers entitled—
"Navigation Work of the Corps of Engineers on the Middle Mississippi River and Its Relation to Flood Flows," by J. C. DeBolt, U.S. Army Engineer District, St. Louis. 1947.
"Bank Protection on Mississippi and Missouri Rivers," by J. H. Jackson, Corps of Engineers. 1935.
"Protection of Beds and Banks of Inland Waterways, Deep Water Channels and Drainage Canals," by H. D. Vogel, Corps of Engineers. 1940.

Sawyer Bend, opposite the northern boundary of St. Louis, where it attains a maximum width of about 12 miles. As a result of filling of the old rock-bound valley with glacial outwash, the present river, in general, flows in a bed of glacial material with bedrock at a considerable depth below the streambed. In its natural state, the river between high banks was relatively wide and shallow, with the channel being bifurcated by numerous bars and islands with their accompanying chutes and sloughs. At low water, channel depths varied from 3 to 10 feet over widths ranging from 125 feet to 2,500 feet. At flood stage, depths varied from 25 to 60 feet and widths from 2,500 feet to as much as 25,000 feet.

The middle Mississippi, in its length of approximately 195 miles, has an average gradient of 0.6 foot per mile. Discharges at St. Louis, where records exist for over 100 years, show extremes of about 20,000 cubic feet per second in 1933 and 1,300,000 cubic feet per second in 1844, the ratio of these extremes being 1 to 65. Average discharge of the river is approximately 175,000 cubic feet per second. As would be expected, variations in stage are considerable, ranging from minus 6.2 feet in 1940 to a maximum of 41.3 feet in 1844 as measured at St. Louis. Average annual duration of bankfull stage ranges from about 5 days at St. Louis to about 36 days at the mouth of the Ohio River. Practically all of the silt borne in the middle Mississippi originates in the Missouri River; only about 7 percent is contributed by the upper Mississippi. The material of the riverbed is mainly sand and silt, and that of the banks is about 70 percent sand, 20 percent silt, and 10 percent clay.

The river shows a marked tendency to meander in the alluvial valley and is seldom found in the geographical center of the flood plain. Several major local changes in its alinement have occurred since it was first mapped by man. One of the most notable occurred in 1881, a rather cataclysmic one, when the Mississippi broke into the channel of the Kaskaskia River and changed its course so radically that St. Marys, Mo., originally on the main bank, was left 5 miles distant from the river, and Kaskaskia, the first capital of Illinois, was changed from the left bank to the right bank of the river (fig. 1). The entire characteristic of the

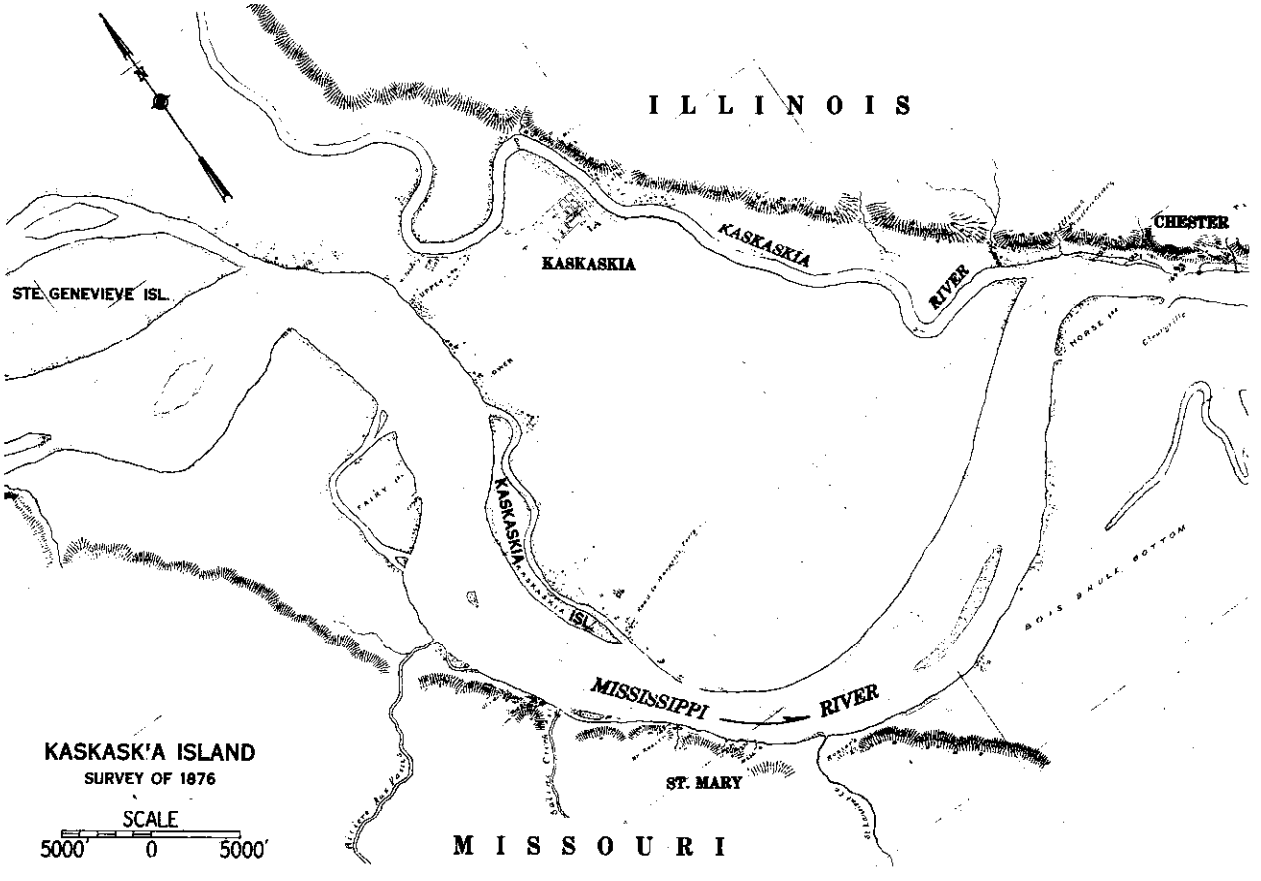


FIGURE 1. — Kaskaskia Island before channel change.

river in the vicinity was altered and formed an 11-mile section of relatively young stream in comparison to the river above and below. Great difficulty was encountered in establishing a new and usable trace (fig. 2).

A notable characteristic of the river is the instability of its banks. Experience has shown that there is severe attack on the banks where the radius is less than 6,000 feet and moderate where the radius is not less than 10,000 feet. An increase in velocity and angle of impingement will increase the erosive action. Direction of current in a bend changes as the stage increases. At low water, the point of maximum attack is usually at or near the head of the bend, and as the stage increases, it moves downstream, and at extremely high water it is usually at or near the foot of the bend. As the bank of a river is eroded, a portion or projection is frequently left intact, which, for some reason, is more resistant to attack than the bank adjacent thereto. Here the current is deflected away from the bank, causing a whirl or suction eddy to develop along the bank below the projection. As a result, the attack on the bank is greatly increased in the area immediately below the projection. Lower ends of bank protection also often act as a false point after some caving has taken place below it. Should an eddy current then develop, there is danger of the bank cutting behind the protection work and thus destroying it from below. This type of caving normally accompanies high river stages. The low water caving is normally uniform and fairly free from eddy action.

The rate of progress and the extent of bank caving depend upon the material of which the bank is composed and its drainage, the curvature of the bank and its general direction with reference to the line of attack, the velocity of the current, the position and depth of the channel with relation to the bank, and the stage of the river. The period of greatest caving occurs normally during receding stages. However, the greatest aggregate of caving results from the continuous attack of the river during low and moderate stages. It very rarely happens that both banks cave in the same locality at the same time, and, as a general rule, erosion on

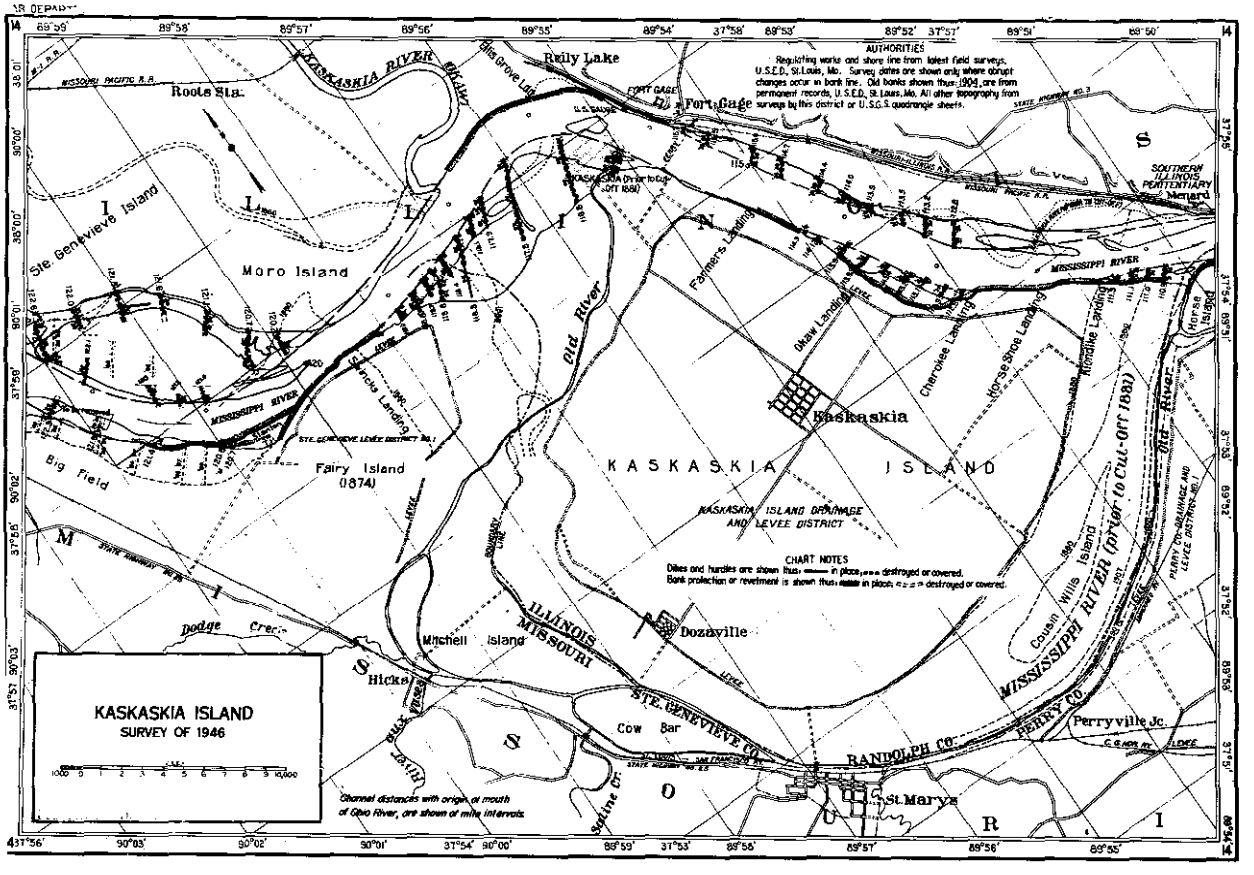


FIGURE 2. — Kaskaskia Island after channel change.

one bank is followed by an accretion on the other.

The first Federal work for improvement of the Mississippi River was authorized by Congress in May 1824 and provided for the removal of snags between New Orleans and the Missouri River. This project, interrupted during the Mexican and Civil Wars, has been carried on as required ever since. The first work in the nature of permanent improvements, "a pier to give direction to the current of the Mississippi River near the City of St. Louis," was authorized by Congress in the Acts of 4 July 1836 and 3 March 1837.

At that time, the Mississippi River opposite St. Louis was divided into two channels by Bloody Island. For some time, Duncan's Island, immediately downstream and adjacent to the right bank, had been growing rapidly, as well as the shoal connecting it with Bloody Island. During extreme low water, steamboats coming from the South had to use the eastern channel and, turning the head of Bloody Island, drop down to the city landing. The course of the river was tending more and more to the east and threatened to take the channel entirely away from St. Louis. To prevent this, Robert E. Lee, then a Lieutenant in the Corps of Engineers, recommended closing of the eastern channel by a dam from the head of Bloody Island to the Illinois shore and the construction of a longitudinal deflecting dike at the foot of Bloody Island to wash away the shoal connecting it with Duncan's Island. These works consisted of training walls of wood piling, brush, and stone.

The dike from the foot of Bloody Island was constructed in 1838 and proved effective in deepening the channel over the shoal. Robert E. Lee, however, abandoned his first proposal for damming the eastern channel and, instead, recommended construction of a longitudinal dike from a point opposite Venice, Ill. Although useful in preventing the existing unfavorable condition from becoming worse, the dike was not in itself a cure for the condition. Subsequently, a dam was built across the channel east of Bloody Island, made a roadway, and finally closed the eastern channel. The project included revetting the western shore of Bloody Island. (See fig. 3.)

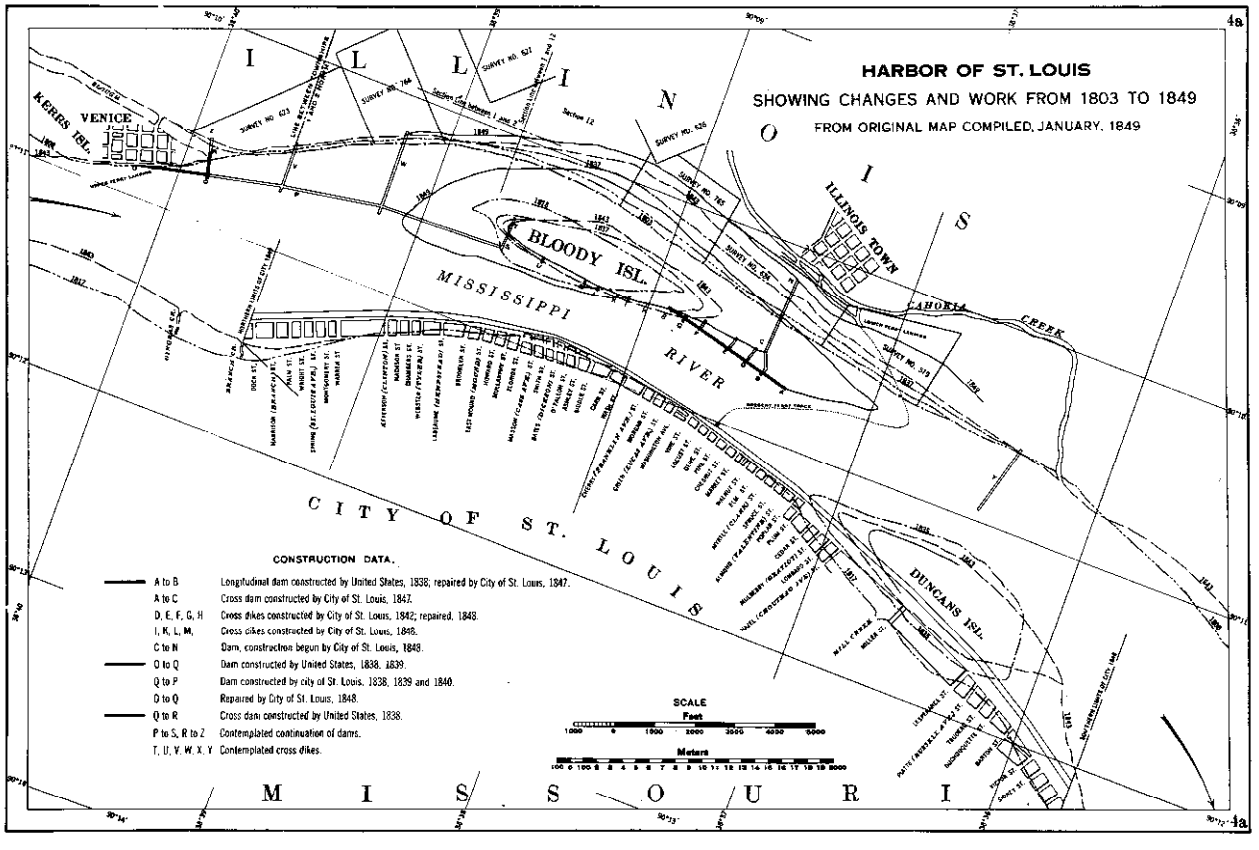


FIGURE 3. — Channel changes, St. Louis Harbor.

In a report on Harbor of St. Louis, dated 3 February 1844, Captain T. J. Cramm states:

It is well attested, by observation on the different reaches of a river, whether its banks be of alluvium or of rock, that where the bank is concave, or hollowing toward the river, on that side will be found the deepest water, the strongest current, and the least tendency to deposit; and where the bank is convex, or rounding toward the river, on that side will be found the shoalest water, the weakest current, and the greatest tendency to deposit. . . . .It is greatly to be regretted that the original founders of the town of St. Louis disregarded these circumstances when selecting the site of the city upon a reach where the bank was convex instead of concave toward the river.

There has been and there will probably continue to be difference of opinion regarding the effect of bank revetments. In the past, some opposed them primarily because they did not believe they would be effective and because of the enormous expense they involve, as well as the difficulty of maintaining them. The advocates of revetment, however, argue that by preventing the caving of banks, the supply of materials for building up shoals below is largely reduced and navigation is thereby improved. Congress, in the River and Harbor Act of 1886, imposed on the Mississippi River Commission the following condition: "That no works of bank protection or revetment shall be executed in said reaches or elsewhere until after it shall be found that the completion of the permeable contracting works and uniform width of the high-water channel will not secure the desired stability of the river banks." This limitation was based on the belief that the river, if once regulated, would not scour its natural banks. The Senate Committee in 1898 expressed the opinion that: "In view of the great cost, comparatively great taking into consideration the results, it would be more judicious to limit revetment to levee and harbor protection."

When engineers consider the hydrography and the direction of currents, the Mississippi when low is not the same river as when high, and the problem of permanent improvement involves reconciliation of these differences. The type of structures used in contracting and training an alluvial river ranges from simple expedients using locally available material to carefully planned and constructed permanent works. The comprehensive plan for regulation of the middle Mississippi River was approved 31 March 1881 and provided for continuous improvement downstream from St. Louis, utilizing revetment and permeable dikes, to reduce the width of the river to 2,500 feet. This authority has served as a basis for all subsequent river regulating works.[2] As previously indicated, the earliest plan of improvement in the middle Mississippi consisted of closing dams to cut off sloughs and minor watercourses in order to confine the flow to a single channel. Originally, these closure structures were low, solid dikes consisting of alternate layers of brush and stone to a height of about 8 feet above low water.

Although some success was obtained with solid dikes, experiments with various other types of structures had shown that timber hurdles or screens would produce a more rapid deposition of sedimentary matter. These early experiments indicated that the true role of a dike was to train the river by encouraging it to deposit sediment where desired, thus utilizing the force of the river itself to improve channel conditions. Consequently, when the existing project was adopted, the permeable dike was substituted for the solid structure (fig. 4).

The permeable piling dike consisted of two rows of single piling, well braced and connected by stringers. This type was improved gradually until 1893, when it was replaced by dikes of two or more rows of 3-pile clumps bound together with longitudinal stringers for better distribution of strain, the number of rows of clumps depending upon the depth and swiftness of water. In practically all cases, a foundation mattress of willows or lumber was sunk with stone on the line of the dike to prevent bed scour, the piling being driven through the mattress to a penetration of 20 to 25 feet. (See figs. 5, 6, and 7). This type of dike construction was used in constricting the major portion of the middle Mississippi River into a single navigation channel. Currently, there are 487,500 linear feet of pile dikes operative in the middle Mississippi River. During the early stage of the project, dikes were spaced at distances approximately 2½ times their length. As time went on, the spacing was decreased to two times their length, and at present, spacing is about 1½ times their length. Where pile dikes could not be used, due to underlying rock ledges, crib dikes have been successfully employed (figs. 8 and 9).

Curtain or screen mattresses, hung on the upstream faces of the pile dikes, have been used at times to increase the rate of fill caused by the dikes. These screens have been used extensively since 1940 and have proved very successful, when used in series, in deepening the channel by inducing deposit between dikes (fig. 10). Permeable dikes were built throughout their lengths with tops at about the 20-foot stage, St. Louis gage. Most dikes in the middle Mississippi River acquire a considerable amount of

[2] The existing project for the middle Mississippi River provides for obtaining and maintaining a minimum channel depth of not less than 9 feet, a minimum width of not less than 300 feet at low water, with additional width in bends from the mouth of the Ohio River to the northern boundary of St. Louis; thence 200 feet wide with additional width in bends to the mouth of the Missouri River; to be obtained by regulating works (piling dike and bank protection) and by dredging.

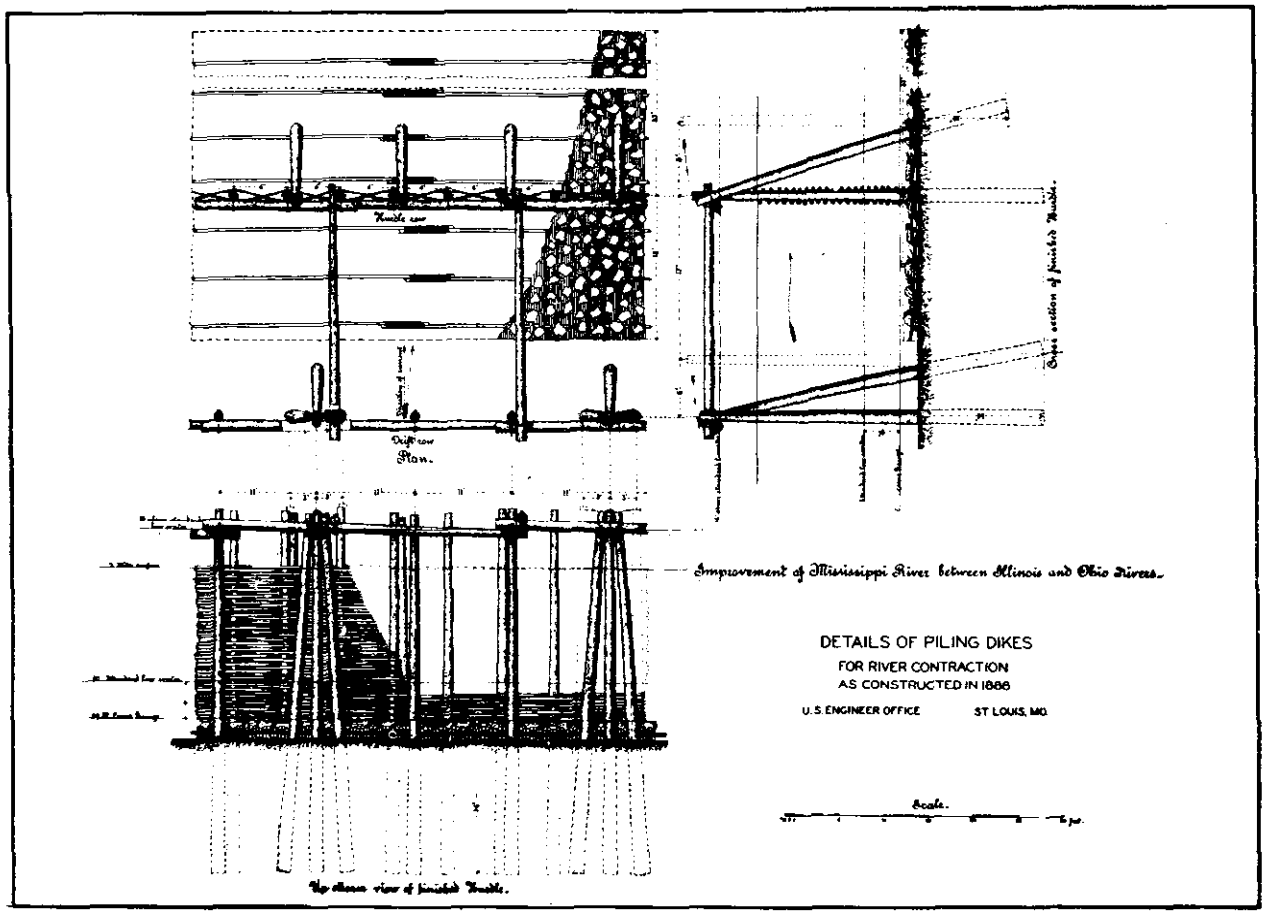


FIGURE 4. — Early type of permeable dike.

fill behind them before the end of 5 years.

Contracting the river in a short time often creates changes that are too radical for the river to adjust itself to in the time allotted. This often results in the failure of the dikes or entails excessive maintenance work before fills accumulate in the dike system. The success of these improvements requires, in general, construction of a continuous system of regulating works.

Although dikes cause deposition and accompanying accretions, in many instances the dike system may induce a heavy attack on the opposite bank downstream. If unchecked, this erosion of the banks could eventually result in an unsatisfactory alinement and a channel condition as bad as that which the dike system was installed to correct. Furthermore, in an alluvial stream like the middle Mississippi, meandering is inevitable if the velocity is sufficient to erode the banks. This potential condition is everywhere present, so that unless the banks are protected, large-scale caving will occur.

The standard bank protection consists of an underwater part and upper bank part. The underwater part is made of woven mattress of lumber or willows laid parallel to the river bank and sunk to the bed of the river with stone ballast (fig. 11). The upper bank part consists of paving extending from the edge of the bank at about mean low water to approximately the 20-foot stage on the St. Louis gage. It consists of a 10-inch layer of riprap stone from 3 to 150 pounds in weight laid on a 6-inch layer of crushed stone. The bank upon which the stone is to be laid is graded to a uniform slope of about 1 vertical on 3 to 4 horizontal in such a manner that it presents a smooth surface (fig. 12). From 1873 to 1963, approximately 157 miles of bank revetment have been constructed on the middle Mississippi River, of which about 115 miles are currently active. An additional 15 miles of revetment are required to complete the bank stabilization program.

The order of executing work in the improvement of any stretch of river is not fixed and

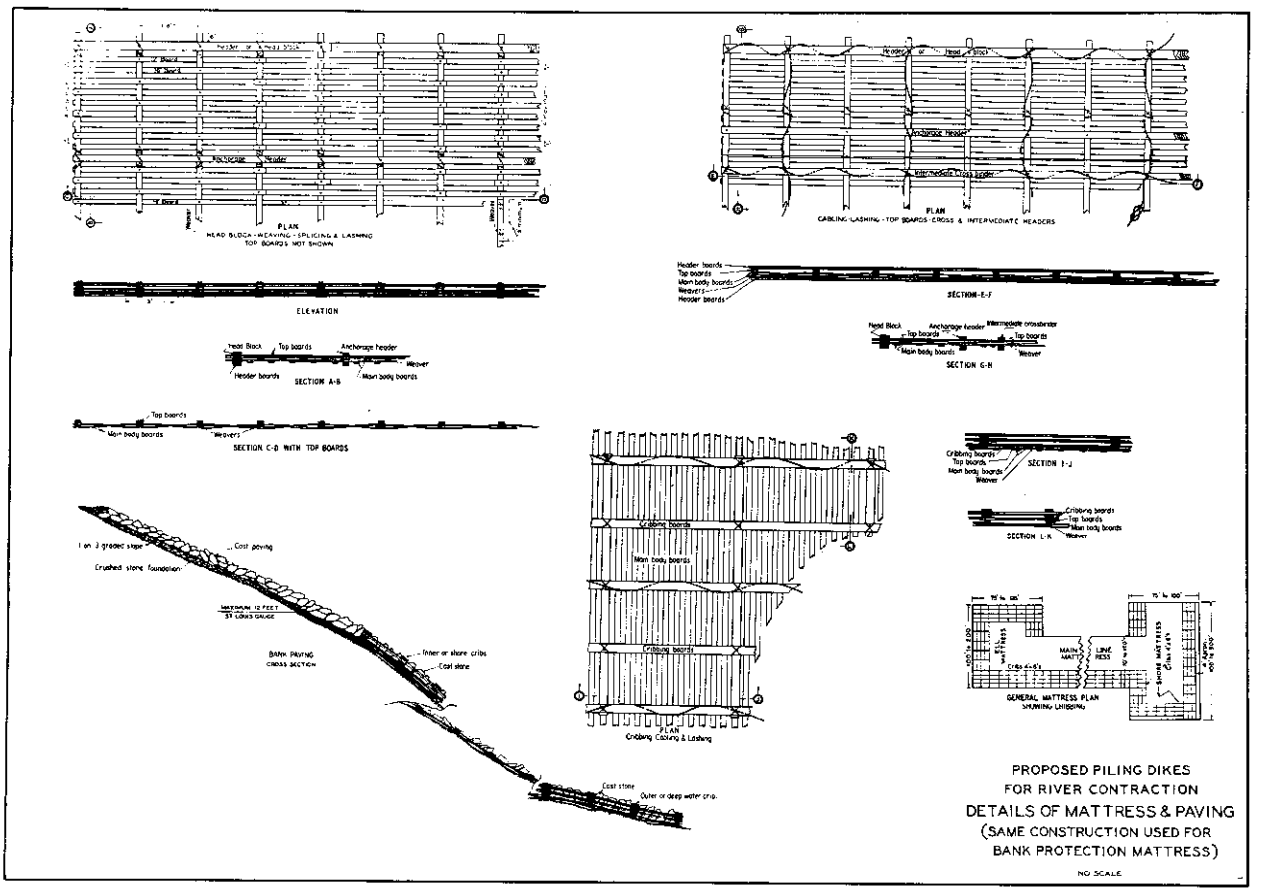


FIGURE 5. — Details of mattress and paving for piling dikes.

depends essentially upon conditions at the site. Work ordinarily should begin with the alinement and fixing of crossings in which the first step would be to fix the outer bank at the lower end of the upstream bend in order to aline the flow of the crossing. This is followed by bank protection to hold the desired curvature at the upstream end of the lower bend. The order of contraction and bank protection in the adjoining bends would depend upon the rate at which the outer bank attains the desired alinement. Normally, work should begin at the upstream end of a reach, construction thus taking advantage of the current. Certainly, bank protection work must proceed downstream because of the method of construction. In the construction of dike systems, however, local conditions may permit construction work to begin at any point or several points simultaneously.

Riverbanks fail as a result of attack from without and within, i.e., erosion or subsidence. Erosion results from direct current action of the flowing water against the banks, from wave wash, and eddy action. The banks of the river are subject to seepage caused by drainage after heavy rains or by falling river stage. Excess moisture seeps through the banks carrying particles with it and causing the material to disintegrate. The training and protective works must be so designed that they will be effective not only at low water but at medium stage. Experience has shown that in the middle Mississippi the ideal shape of a bend, both for bank protection and for dependable navigable depths, is one having a radius of about 14,000 feet. Shaping the bank into a smooth general curve is essential to remove any obstructing points, otherwise eddy action is almost certain to develop and endanger the protection works. In the St. Louis district the tendency of the channel to shift in reaches can best be controlled by construction of contraction works and revetments to introduce a slight sinuosity into reaches, which will tend to fix the deep water channel along the outer banks of the minor bends thus created.

In addition to controlling the trace of the channel at low water, bank protection must be

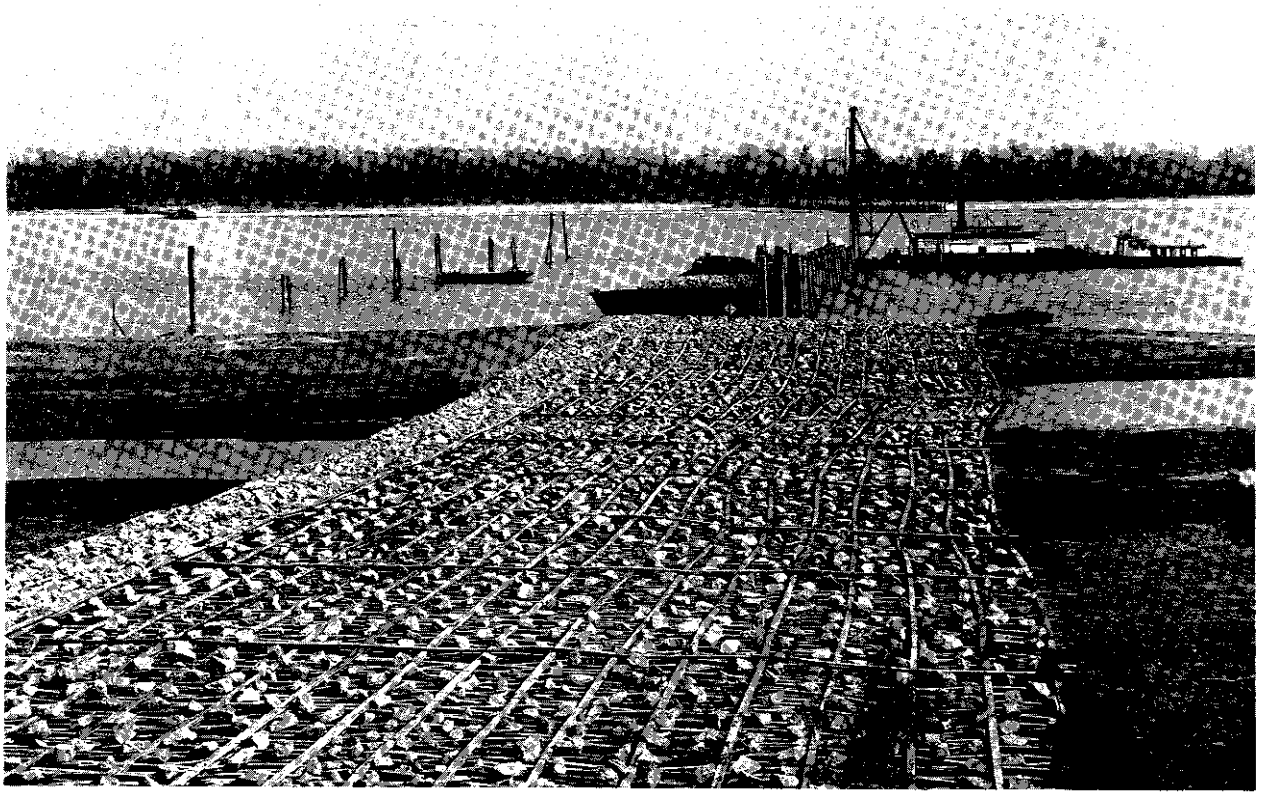


FIGURE 6. — Mattress and paving for piling dikes under construction.

adequate to prevent a tendency to meander and form new channels at high stages. As a rule, a comparatively smooth flow begins in the crossover immediately downstream from a rock bank and comes into the bend parallel to the bank line to be protected. In this case, the starting point of the revetment is located at the downstream end of the rock bank. Where a crossing approaches a bend at such an angle as to allow direct current attack against the revetment, the revetted bank is difficult to maintain. In such cases, the best results are obtained by contraction works upstream from the bend, to train the channel flow so as to enter the bend on a tangent to the curve. Where the head of the bend does not begin at a rock bluff, the bank line is shaped into the desired curvature by the direct attack of the current which caves away the bank until flow is eased into a parallel position with the bank. In such cases, the starting point for revetment should be selected well above the point where the main current begins to flow along the concave bank. In some instances, a few short spur dikes should be built upstream to take the brunt of the attack in the event the crossing may shift and take position upstream from the head of the revetment.

Major causes of failure of bank protection aside from ice damage are (1) erosion of the bed and saturation of the banks during extremely high river stages — the high velocities tend to pull material through the small openings in lumber or willow mattress thus undermining the revetment to some extent which, together with the over-saturated condition of the bank as the river recedes, causes a sloughing or settlement of the revetment; (2) a sharp, severe attack at various stages that causes bed erosion and undermines the foundation mattress with resultant sliding or sloughing; and (3) extremely low river stages tend to undermine the bank paving because of the increase in velocity at some localities. The composition of the soil has much to do with the stability of bank protection.

Revetment must be sufficiently flexible to permit its conformity to the bank and bed and to allow its adjustment to changes that may develop. The type of revetment and its effectiveness are largely dependent upon the manner in which banks have been prepared to receive it. The principal aim is to obtain a well rounded curvature, leading tangentially to and from straight or curved reaches. Drainage should be provided and the bank graded to a uniform

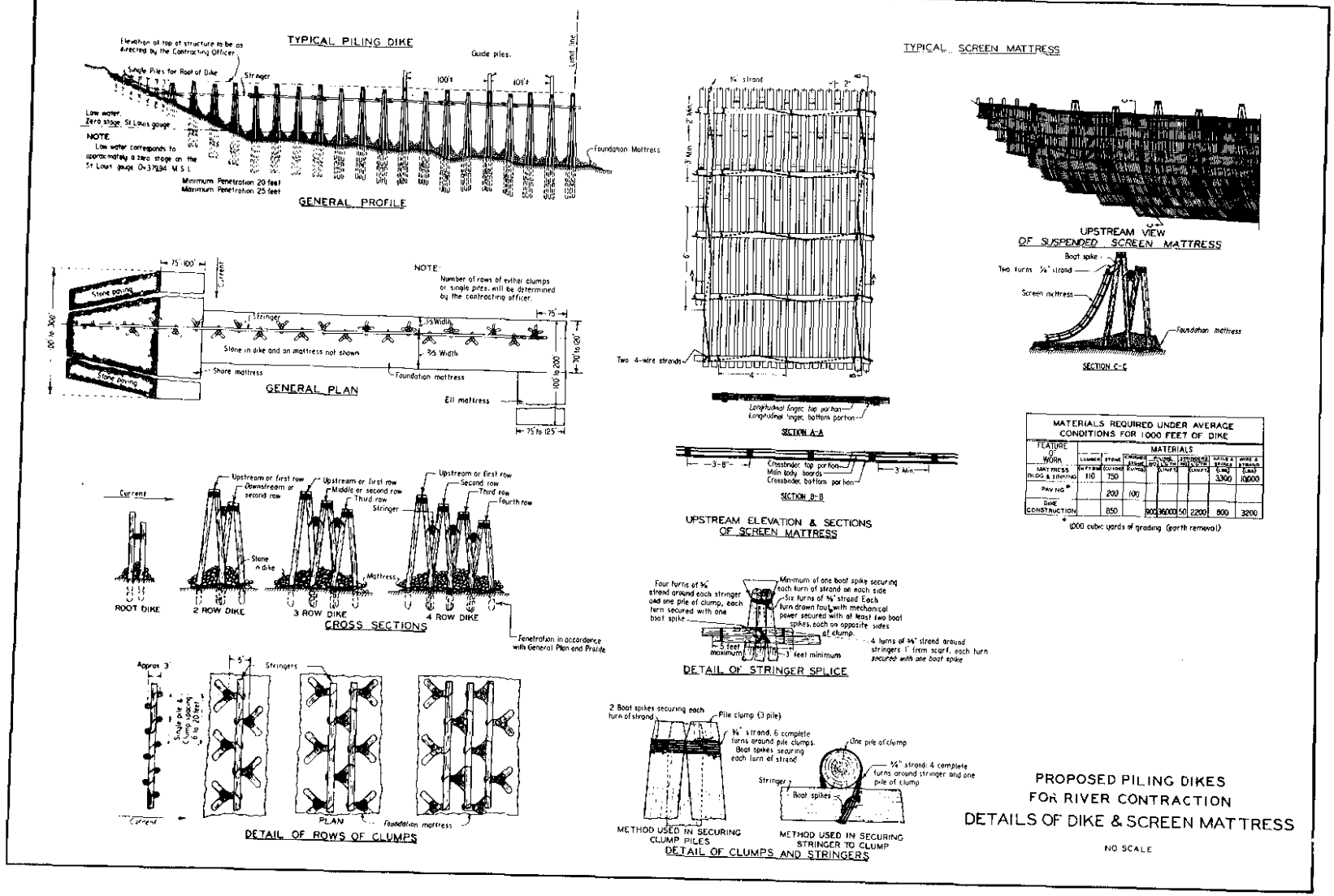


FIGURE 7. — Details of piling dikes.

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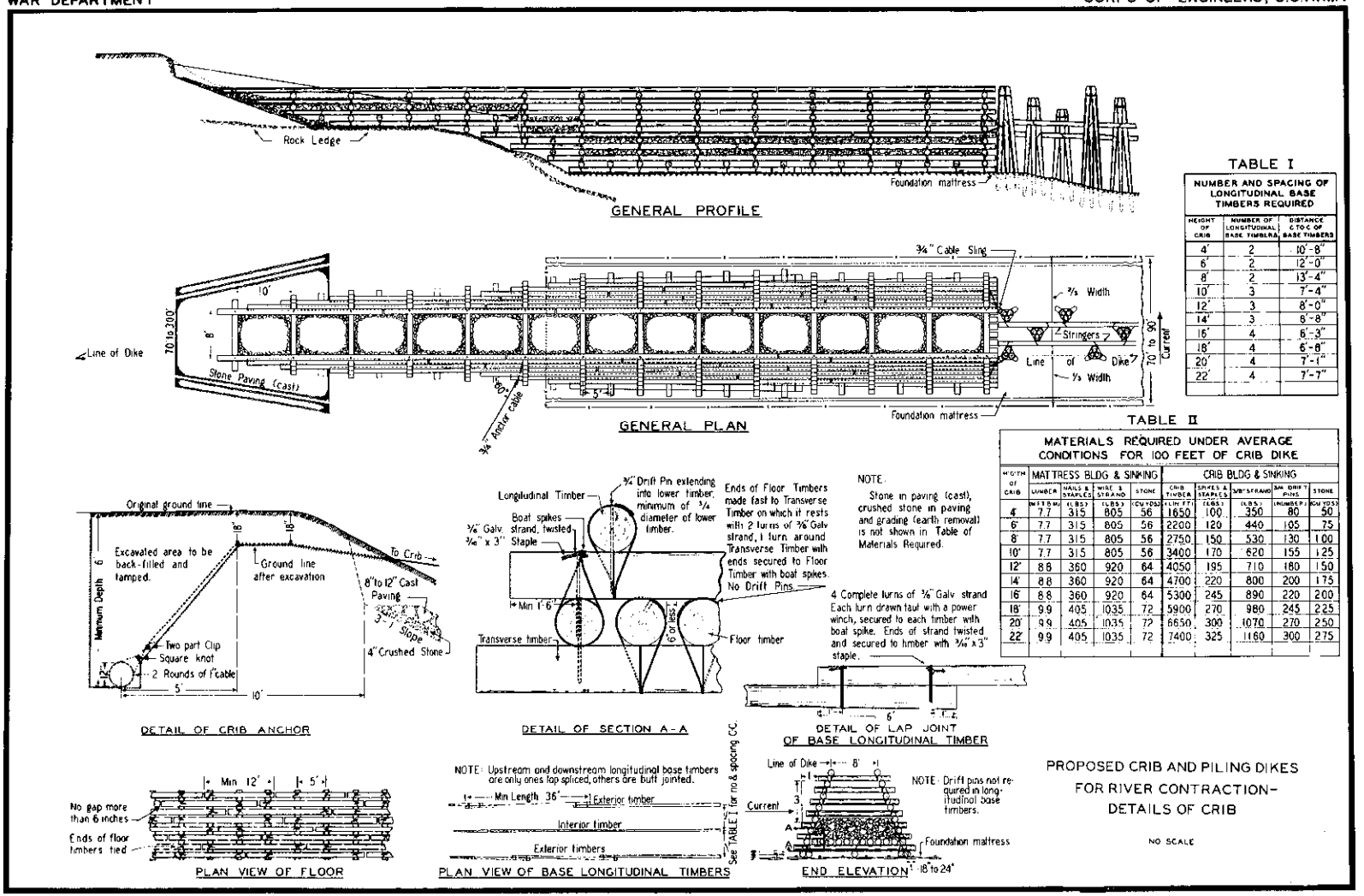


TABLE I

NUMBER AND SPACING OF LONGITUDINAL BASE TIMBERS REQUIRED

HEIGHT OF CRIB	NUMBER OF LONGITUDINAL BASE TIMBERS	DISTANCE C TO C OF BASE TIMBERS
4'	2	10'-8"
6'	2	12'-0"
8'	2	13'-4"
10'	3	7'-4"
12'	3	8'-0"
14'	3	8'-8"
16'	4	6'-3"
18'	4	6'-8"
20'	4	7'-1"
22'	4	7'-7"

TABLE II

MATERIALS REQUIRED UNDER AVERAGE CONDITIONS FOR 100 FEET OF CRIB DIKE

WIDTH OF CRIB	MATTRESS BLDG & SINKING				CRIB BLDG & SINKING				
	LUMBER	NAILS & STAPLES	WIRE & STRAND	STONE	CRIB TIMBER	SPIKES & STAPLES	⅜" STRAND	¾" DRIFT PINS	STONE
	M F B M	(LBS)	(LBS)	(CU YDS)	(LIN FT)	(LBS)	(LBS)	(NUMBER)	(CU YDS)
4'	7.7	315	805	56	1650	100	350	80	50
6'	7.7	315	805	56	2200	120	440	105	75
8'	7.7	315	805	56	2750	150	530	130	100
10'	7.7	315	805	56	3400	170	620	155	125
12'	8.8	360	920	64	4050	195	710	180	150
14'	8.8	360	920	64	4700	220	800	200	175
16'	8.8	360	920	64	5300	245	890	220	200
18'	9.9	405	1035	72	5900	270	980	245	225
20'	9.9	405	1035	72	6650	300	1070	270	250
22'	9.9	405	1035	72	7400	325	1160	300	275

FIGURE 8. — Details of crib dike.



FIGURE 9. — Crib dike under construction.



FIGURE 10. — Screen mattresses for piling dikes.

lope of about 1 on 3. Where the bank material s unstable, the slope may be decreased to about on 5. Because of its flexible nature, riprap is enerally more suitable and of longer life than ther types of paving.

During the past few years, the number of revetment slides and the high cost of repair showed the need for developing a more stable and economical foundation. Experimental stone foundations were constructed at the base of

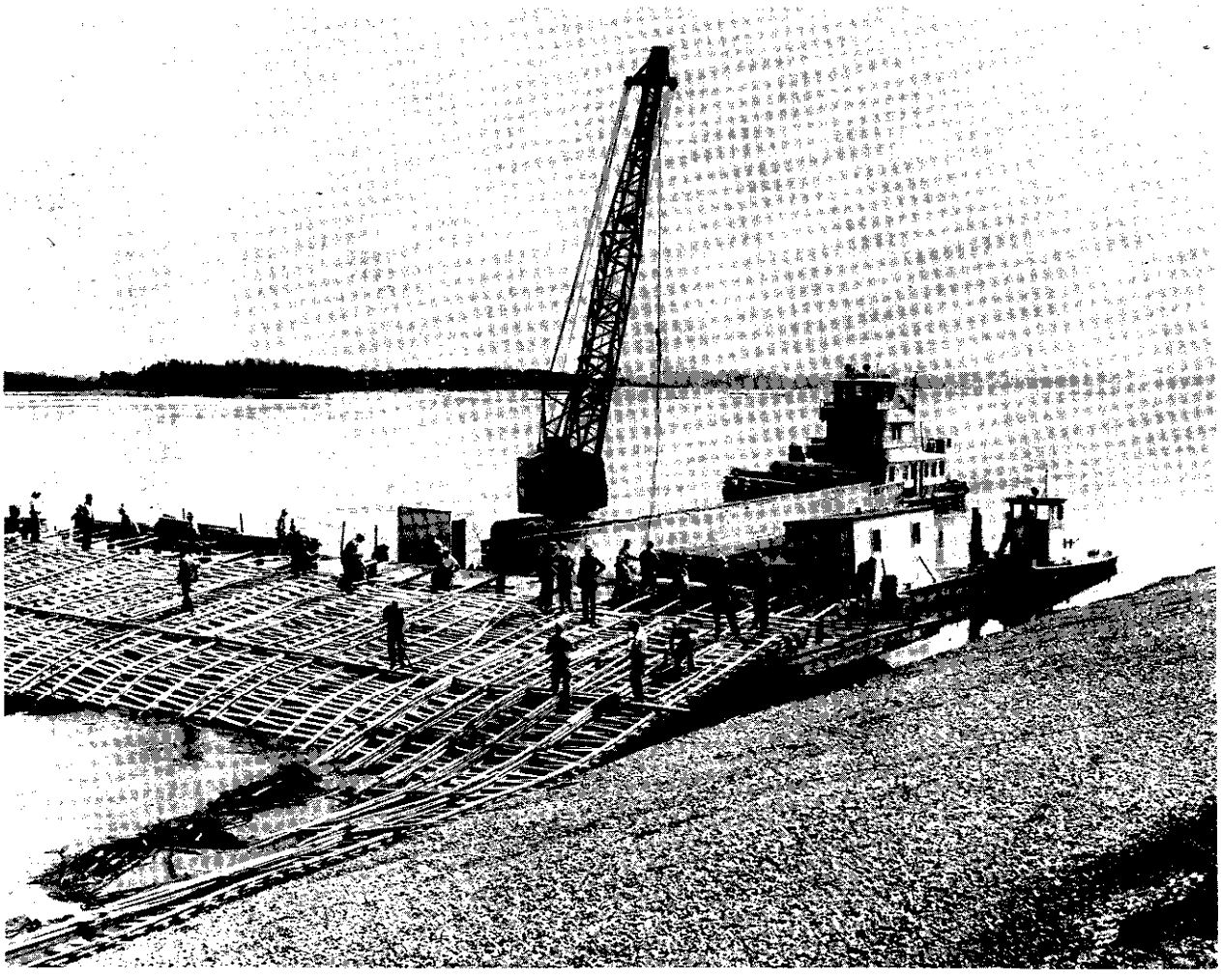


FIGURE 11. — Woven mattress for bank protection.

deteriorated revetments. The foundation consisted of approximately 1 ton per foot of ballast or quarry-run stone at the toe of the revetment slope. In every case where stone foundation had been substituted for the lumber mattress, there has been no evidence of deterioration or failure. This type of foundation has proved more stable and less costly than lumber mattress. The method has been adopted and is now being used throughout the middle Mississippi.

Sediment from upstream overbank areas and from bank erosion still are factors in river-regulating works, but their contribution is diminishing. The upper Mississippi has never been considered a heavy silt-laden river, whereas, the Missouri in the past has transported enormous quantities of silt into the middle Mississippi. Observations made since 1910 show that the average silt load at St. Louis has diminished about one-third. This is attributable in a large part to the extensive water resource developments constructed and land treatment measures undertaken in the Missouri valley since the turn of the century. Indications are that we are now entering a stage where impermeable dikes are needed. Acting as baffles, they scour out the riverbed at low stages when very little sediment is in the river. At higher stages, when the silt load is greater, they serve as sediment traps more effectively than the timber dikes. Rock fill dikes are generally cheaper when the height is less than about 23 feet. At greater heights, due to the volume of stone required, pile dikes become more economical. Other factors, however, such as repair and maintenance costs and general effectiveness, must be considered. In some areas, stone dikes have a distinct advantage over timber pile dikes, due to their low cost, greater efficiency, and virtual indestructibility.

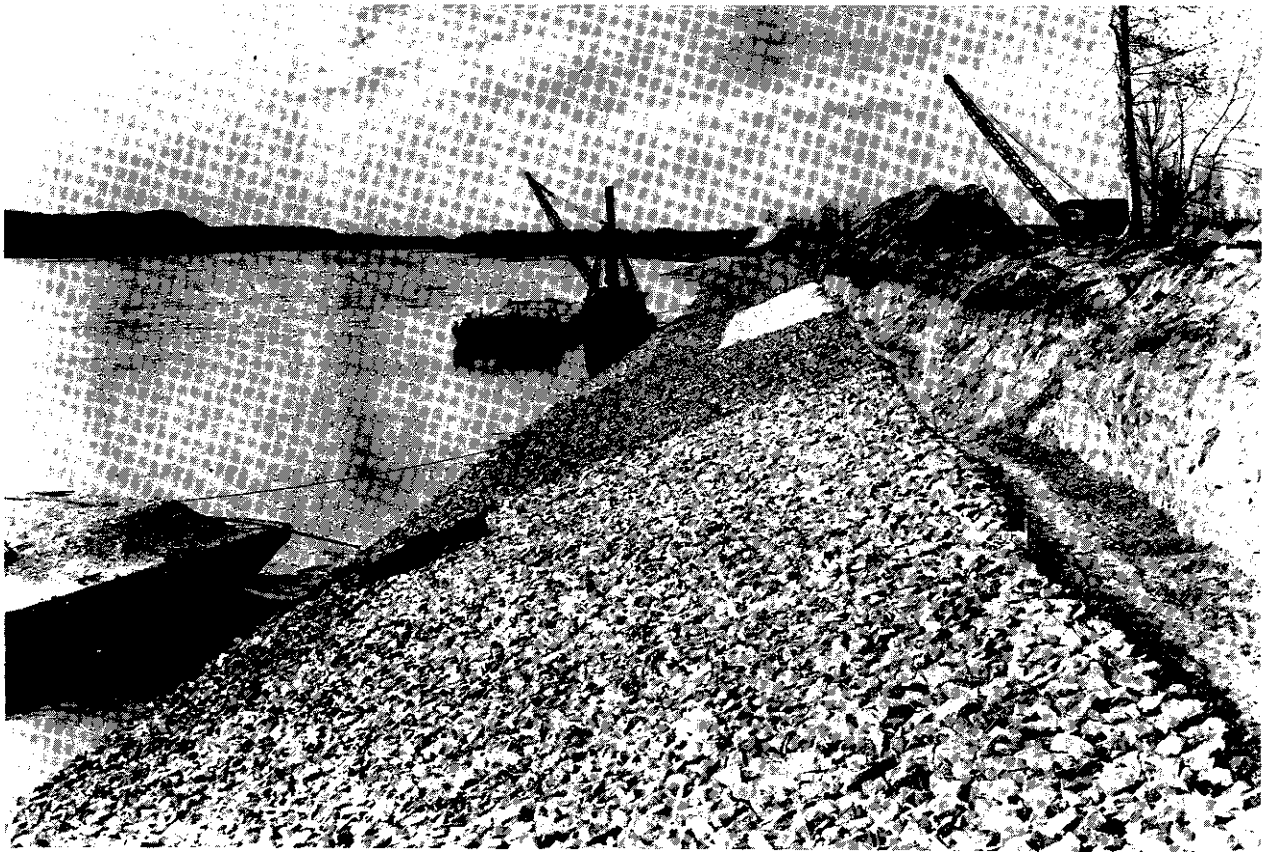


FIGURE 12. — Paving for bank protection.

It is evident that the degree, methods, and materials used for river regulation, control, and stabilization were compatible with the specific navigation and flood control needs of each era. The present methods have proved effective and at the lowest possible cost. Materials are readily available and maintenance costs are relatively low. Nevertheless, new and improved methods are constantly being searched for in the light of more intensive consideration of the 12-foot waterway. Improvement over present methods of protection will probably come under the impact of increasing navigation demands and the trend for greater utilization of flood plain areas. Neither the potential of the river nor the needs of man have yet been satisfied.

DOWNSTREAM PATTERN OF RIVERBED SCOUR AND FILL

[Paper No. 46]

By WILLIAM W. EMMETT, *hydrologist*, and LUNA B. LEOPOLD, *chief hydrologist, U.S. Geological Survey*

Introduction

Progress has been made in describing riverbed scour and fill at a given stream section. One needs only the data routinely collected at a stream-gaging station to observe scour and fill at that station. However, similar progress has not been made to determine whether or not the scour and fill observed at a given section extends over a relatively long reach of channel. Gaging stations are generally located too far apart to draw any conclusions as to scour processes between stations. It remains necessary then to establish a sufficient number of cross sections along a channel to describe the downstream pattern of riverbed scour.

This paper presents recent observations of channel scour and fill over relatively long reaches of three streams in the western United States. Separate sections have been devoted to those observations on an ephemeral channel and those on perennial streams.

The authors acknowledge with thanks the

work of Leon A. Wiard, Robert M. Myrick, and other colleagues of the U.S. Geological Survey for data important to the present discussion.

Ephemeral Streams

Geographic Setting and Basic Measurements

The Arroyo de los Frijoles is a sandy ephemeral channel located about 4 miles northwest of Santa Fe, N. Mex. Physiographic and hydraulic characteristics of channels in this locality have been previously studied.[1] Only a summary of those characteristics pertinent to the present paper are presented here.

Normally, flows are the result of runoff from local thunderstorms during the summer months. The flash flood is typical. Peak flow occurs within several minutes after the initial flood wave. Throughout the study reach, the channel increases in size from a rill near the watershed divide to a width of about 100 feet downstream. The bed is composed of medium sand and a moderate amount of gravel. This material extends downward many feet, far deeper than the scour that occurs. The medium sand grain diameter is about 0.5 mm.

Scour and fill data from the Arroyo de los Frijoles were collected by means of scour chains during the 5 years, 1958–62. Determination of the amount of scour and fill using the chain method is a simple procedure. The chains are buried vertically in the streambed with the top link at or slightly above the bed surface. After a flow, the elevation of the streambed is resurveyed and the bed is dug until the chain is exposed. If a scour has occurred, a part of the chain will be lying horizontally. The difference between the previous streambed elevation and the elevation of the horizontal chain is the depth of scour. The difference between the existing bed elevation and the horizontal chain is the depth of fill. If no scour has occurred, the depth of fill is the increase in bed elevation.

Scour chains, each 4 feet in length, were installed along a reach of nearly 6 miles. The location of the chains usually followed the low-water channel. Thus, a flow in any reach of the channel could be expected to pass over the chain location. Over most of the study reach, chains were placed on 1,000-foot intervals. In one reach of 2,000 feet, chains were placed at 100-foot intervals. This spacing was believed sufficient to determine any downstream trend in the scour pattern in this arroyo. At seven of the chain sections, additional chains were installed across the width of the channel and provided an indication of any lateral variation in scour.

[1] LEOPOLD, L. B., and MILLER, J. P. EPHEMERAL STREAMS—HYDRAULIC FACTORS AND THEIR RELATION TO THE DRAINAGE NET. U.S. Geol. Survey Prof. Paper 282–A. 1956.

Observations of Scour and Fill

On the average, there are about three flows down the arroyo each year. Scour and fill data are available for most flows in the 5-year period. However, since some chains were installed before others, an equal length of record does not exist for each chain location. In addition, some chains, usually those in the uppermost or lowermost reaches, were not surveyed after each flow. These missing segments of data disallow a true accumulative value of scour and fill. However, the net change in bed elevation since the time of the initial survey may still be obtained.

By 1959, the majority of the chains had been installed along the arroyo. Scour and fill data for a sample flow, for the year 1962, and for the period 1958-62 are shown on figure 1. For each flow on this figure, the lower dashed line represents the depth of scour and is plotted against distance along the channel. The upper dashed line represents the depth of fill. The heavy solid line represents the net change in bed elevation after scour and fill. The upper part of figure 1 shows the drainage area studied and the general location of the chains by chain number.

The nature of the flash flow does not always flood the entire length of the arroyo with each storm. It may be that the flow-producing rain was so located that only lower reaches received runoff. Or, for a smaller storm near the headwaters, a part, or possibly the entire flow, may be absorbed into the ground by percolation before it reaches a downstream section. A third possibility is that a particular chain section may be left dry or the section has very little scour because it was not in the low-water path of flow. For a single storm then, it is likely that there may be a considerable variation in the recorded depth of scour from section to section. This is further exemplified in the reach centered around station 25,000 feet where the chains are placed on 100-foot intervals. Despite individual variations, a general consistency prevails among the data; that is, at most all sections along the channel, there is a scour and subsequent fill. All flows produce this same pattern with the magnitude of scour primarily dependent upon hydraulic factors of individual flows and these are related to the intensity and total amount of rainfall.

In the data for some flows, short reaches do not indicate a scour. An example is in the reach between stations 2,000 to 3,000 feet for the flow of July 5, 1962 (top profile, fig. 1). In these reaches, the combined action of previous aggradation and of the chain sagging in the vertical had left the top link of the chain some distance below the bed elevation. Thus, scour may and probably did occur, but not to the depth necessary to register on the chain. After

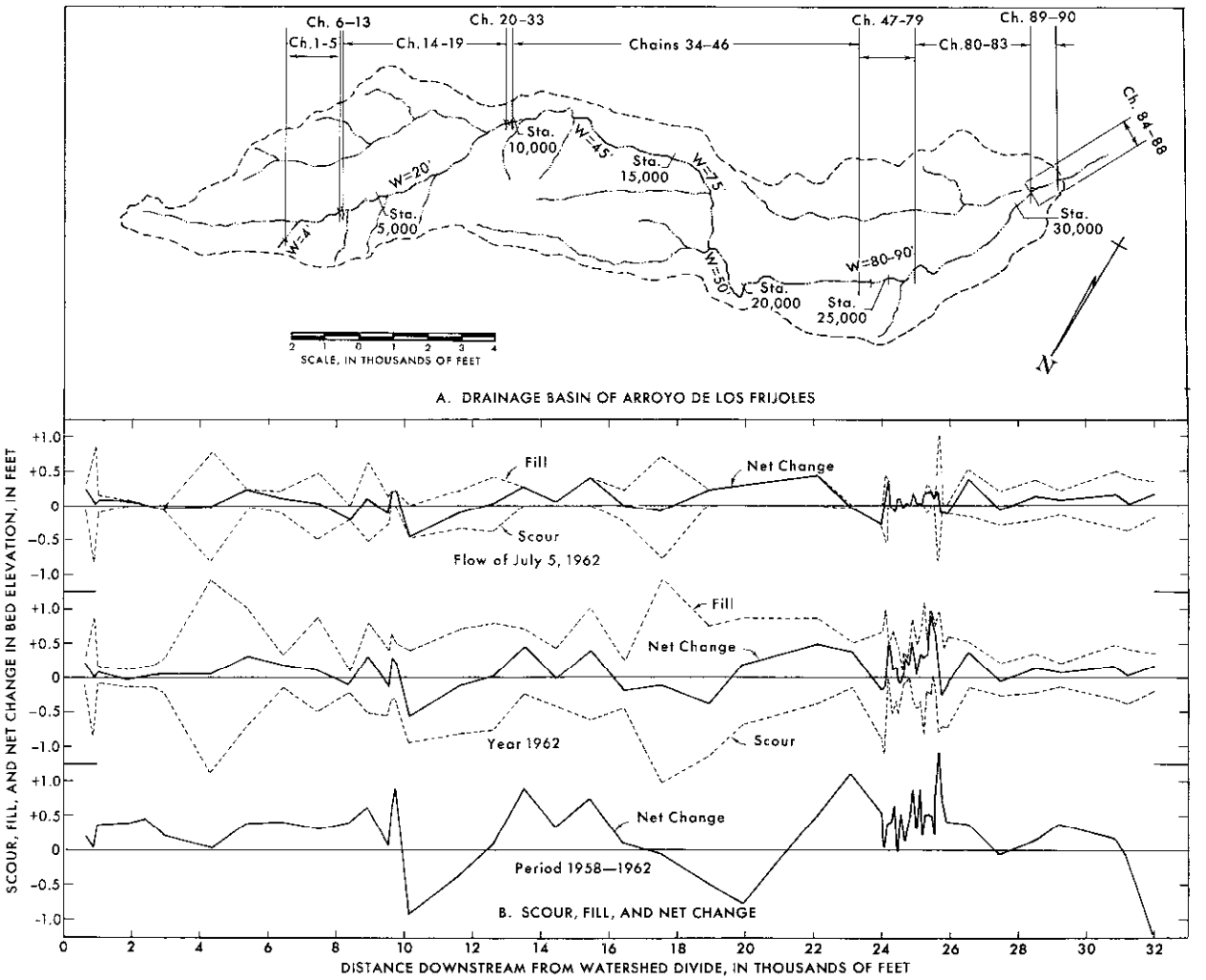


FIGURE 1. — Arroyo de los Frijoles near Santa Fe, N. Mex.

the flow of July 5, 1962, extra length was added to the top of the chains to compensate for the channel aggradation. In addition, closer watch was maintained to check against the chains losing length by sagging.

Three reaches of the channel are the objects of special study and at which flow rates are measured by indirect means, that is, from slope of the high-water marks, estimated roughness, and measured cross-sectional areas of flow. These reaches are centered around stations 2,850 feet, 9,700 feet, and 25,000 feet. It is also within these reaches that the chain sections are located to determine the cross-channel pattern of scour. The mean depth of scour at a section may be determined by averaging the values from the several chains at each of the sections. In a comprehensive report describing various hydraulic and geomorphic processes in the Arroyo Frijoles locale the authors have related these mean values of scour to discharge per unit width of channel. The results of this study are summarized on figure 2. Despite considerable scatter among the data, the mean scour depth appears to be proportional to the square root of discharge per unit width of channel.

An increasing depth of scour is not observed in any single profile as in those shown in figure 1, because during individual flows discharge rarely increases downstream as drainage area increases. This follows from the local nature of most thunderstorm rains and because of the importance of water loss by infiltration into the bed during flows of short duration.

Probably for similar reasons, the depth of scour is apparently independent of the channel

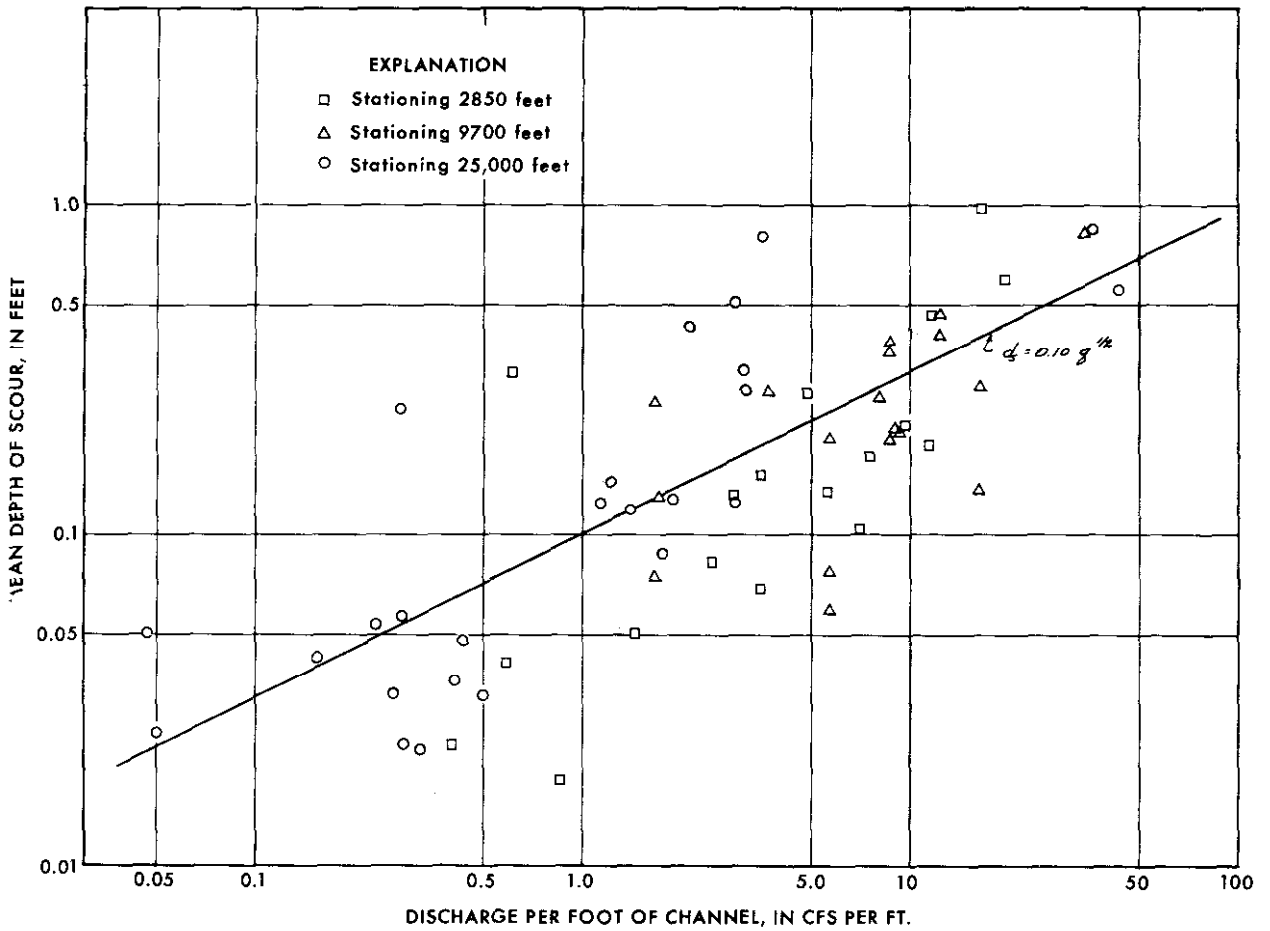


FIGURE 2. — Depth of scour as a function of unit discharge.

width. Channel widths average about 4 feet between stationing 0 to 1,000 feet, 20 feet between stationing 1,000 to 9,000 feet, 45 feet between stationing 9,000 to 13,000 feet, 75 feet between stationing 13,000 to 16,000 feet, 50 feet between stationing 16,000 to 21,000 feet, and 80 to 90 feet from stationing 21,000 feet throughout the rest of the study reach. In addition to the increase of width downstream, local variations occur on the order of threefold to fivefold. No systematic relationship could be established between local depth of scour and the corresponding channel width. Further, since slope decreases downstream as width increases, the magnitude of scour also appears to be independent of slope.

Seven sections provided data to study lateral variation in scour across the width of the channel. Two of these sections, chosen as representative of a particular reach, are illustrated in figures 3 and 4.

Figure 3, illustrating a section at station 25,000 feet, indicates a slight aggradation for the 5-year period. The whole width of channel scours during nearly every flow, but the amount of scour and fill varies across the width.

Figure 4 illustrates a section near station 9,700 feet. Again, considerable variations exist among the yearly records. The 5-year average indicates little net change across the width of the channel. Of particular interest is the lateral distribution in depth of scour. A fourfold decrease in depth of scour is usually noted across the 40-foot width of channel. This difference may be attributed to the higher streambed elevations near the right bank, but even this part of the width scours at times and then fills. The data indicate that some flows may have missed these chain locations entirely and certainly, for all flows, the depths of flow were much shallower on the right side of the channel. The chains at higher elevations are thus expected to indicate less scour.

The scour chains provide an economical and

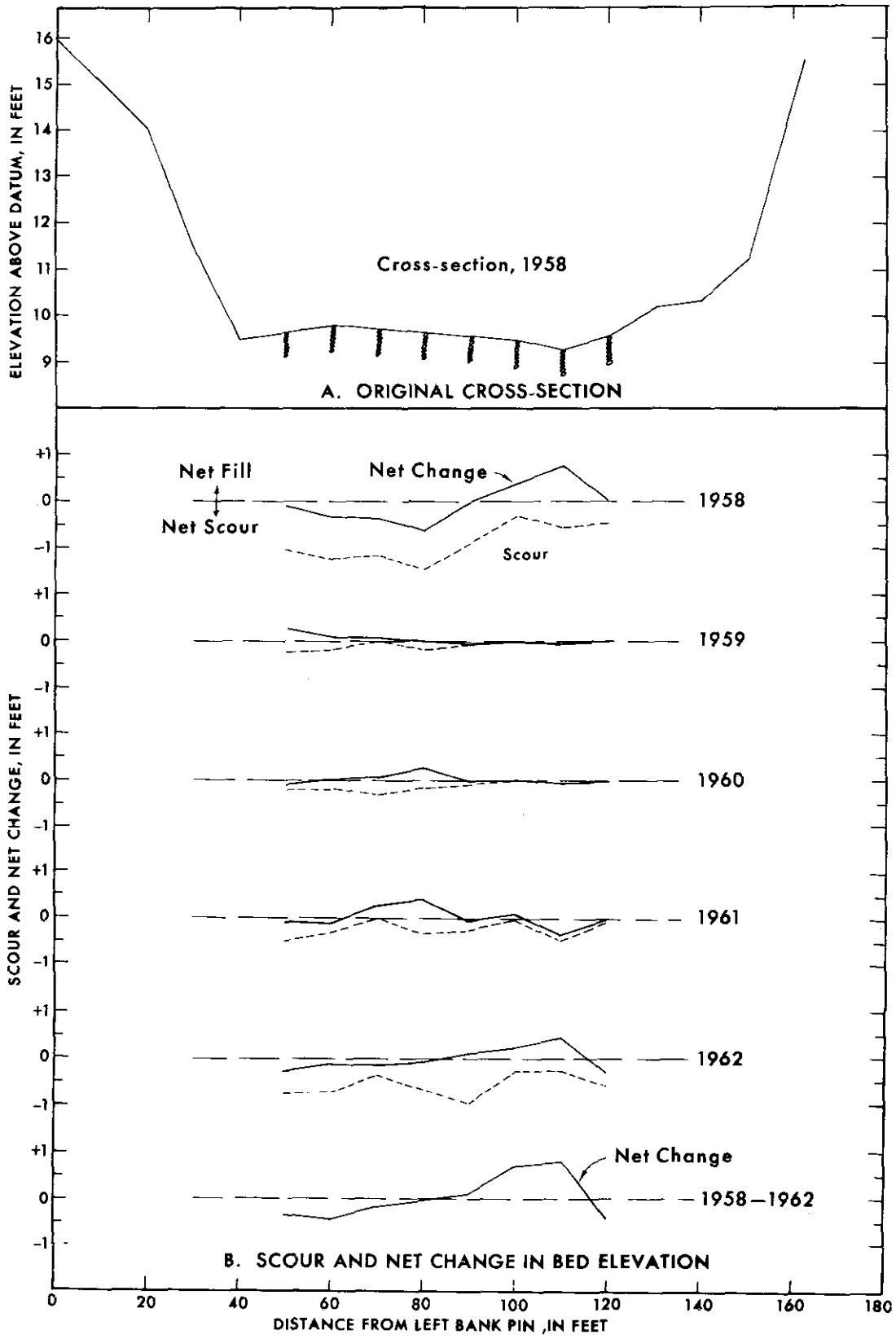


FIGURE 3. — Scour and net change in bed elevation at stationing 25,000 feet for a 5-year period.

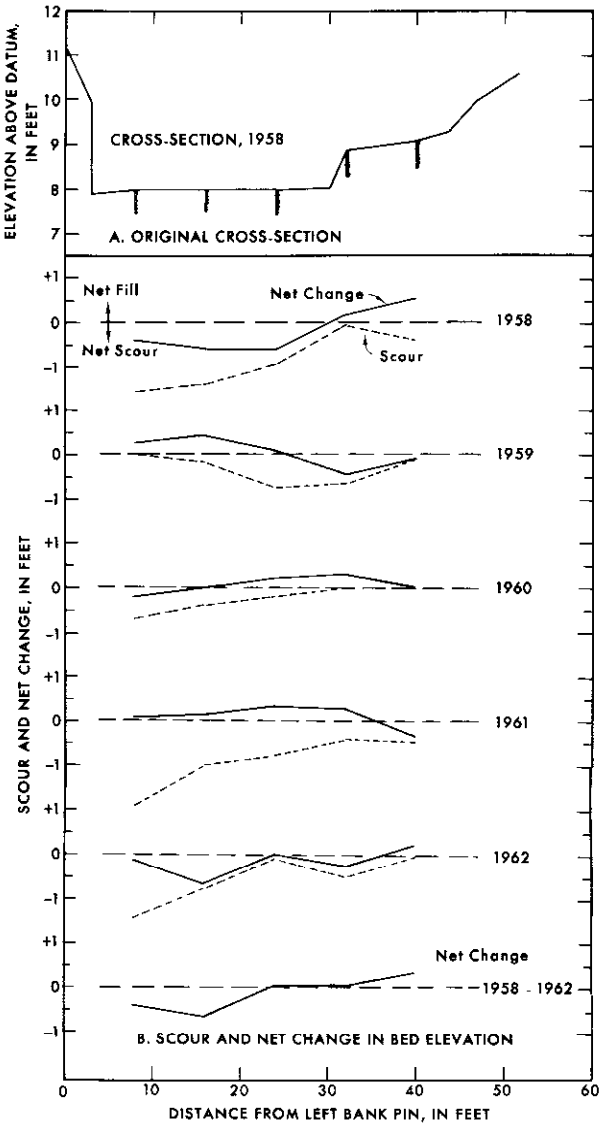


FIGURE 4. — Scour and net change in bed elevation at stationing 9,700 feet for a 5-year period.

simple way of recording net scour and fill for individual flows. It is important that elevations be determined from a reliable bench mark in the immediate vicinity of the chain cross section. We use a 4-foot length of iron reinforcing bar driven nearly flush at each end of the cross section, and one of these is also used as local bench mark for elevation surveys. This iron pin is labeled by a brass tag on which elevation and section number are stamped.

The chain should have a wide or open link so that sand will pack in the chain link. The bottom of the chain is anchored to a rock or a metal head to keep it from pulling out.

The disadvantage of the method is that one does not know when during the flow the scour or fill occurred nor the relative simultaneity of scour or fill at different chains.

Perennial Streams

Geographic Setting and Basic Measurements

The Rio Grande del Ranchos is a small perennial stream on the west slopes of the Sangre de Cristo Range about 7 miles south of Taos, N. Mex. Peak discharges occur in the spring and are normally produced by snowmelt. The study reach consists of a straight reach of 250 feet followed by a curved reach of 700 feet, as can be seen in figure 5. The streambed is predominantly gravel and quite uniform in size from section to section. Median particle size ranges from 21 to 33 mm. with the minimum median size found in the curved reach. The riffle or bar at station 1 + 00 is a predominant feature of the straight reach. A total of seven sequences of pool-riffle occur within the study reach. Channel width at bankfull stage varies from 17 to 36 feet.

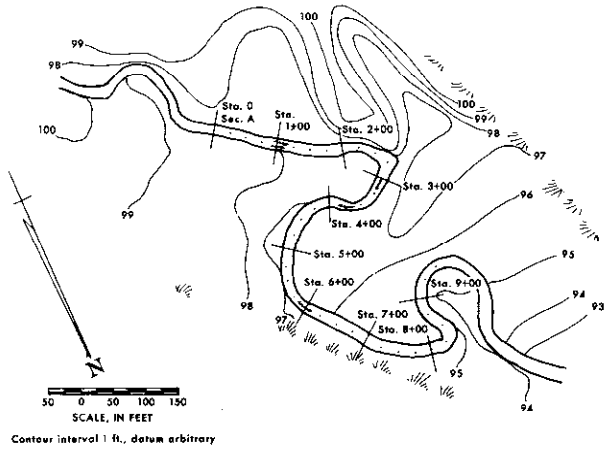


FIGURE 5.—Rio Grande del Ranchos near Taos, N. Mex.

Data were collected by Leon A. Wiard and his associates on the Rio Grande del Ranchos during and immediately after the 1961 snowmelt. A total of 32 measurement stations were established along the study reach. At each measurement station, channel cross sections were obtained by level and rod during the peak flow and at a low flow by depth measurements in connection with velocity observations. Depth measurements were referenced to the earlier survey by known water-surface elevations. High-flow discharge was 130 c.f.s. and low-flow discharge was 25 c.f.s. The data then consist of 32 cross-sections surveyed during spring high

flow (near bankfull stage) and again at low flow in the succeeding summer.

The Popo Agie River is a larger perennial stream in western Wyoming. The study reach is located about 1 mile northeast of Hudson, Wyo., and consists of a curved reach of 2,000 feet followed by a straight reach of 2,100 feet (fig. 6). Peak flow is normally associated with the spring runoff from snowmelt. The streambed is predominantly gravel, ranging in size from fine gravel to large cobbles. Seven sequences of pools-riffles were observed within the study reach. Channel widths at near bankfull stage vary from 86 to 160 feet.

Data were collected at 15 stations along the 4,100-foot reach. Channel cross sections at these stations were obtained during the peak flow in the spring of 1961 and at a low flow later that summer. A second set of data was collected during the rising stage in the spring of 1962 and during a lower, but falling stage that summer. The high-flow observations were obtained by depth measurements referenced to known water-surface elevations. Low-flow cross sections were surveyed with rod and level. High-flow discharge in 1961 was 1,400 c.f.s. and low-flow discharge in 1962 was 310 c.f.s.

Figures 7 and 8 are typical of the cross-sectional measurements obtained. For each stream a section is shown from the curved reach and another from the straight reach.

For both streams and for each measured cross section, the plotted cross sections at high and low flow were superimposed and provided the basis for a scour-fill study. Areas were planimetered from these plots to show the difference in bed between high and low flows.

Low-flow measurements were taken after the high-flow measurements. It is believed, however, that the order in which the measurements were obtained is not of prime importance. Our studies indicate that the cross-sectional configuration of a stream before a high flow is reasonably similar to that some time after the high flow.

The 1961 Popo Agie data and the Rio Grande del Ranchos data represent high-stage measurements obtained during peak flow and the low-stage measurements during the stable low flow. Under these conditions, the changes occurring in streambed elevation were similar in that the bed elevation was scoured during the high flow.

The 1962 Popo Agie data were collected during a different condition of flow. Here, high-stage data were collected during the rising stages of the peak flow and the low-flow data were collected before the stable low flow. In addition, the collection of these data was in the

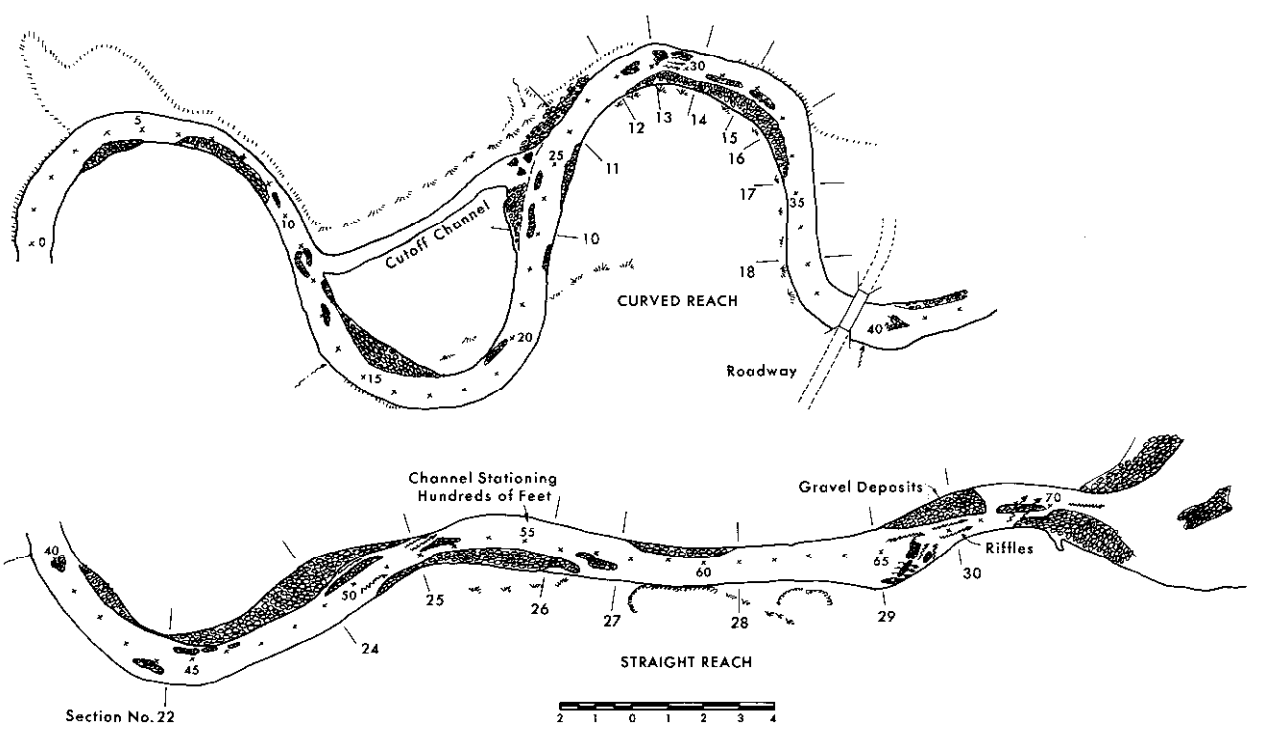


FIGURE 6. — Popo Agie River near Hudson, Wyo.

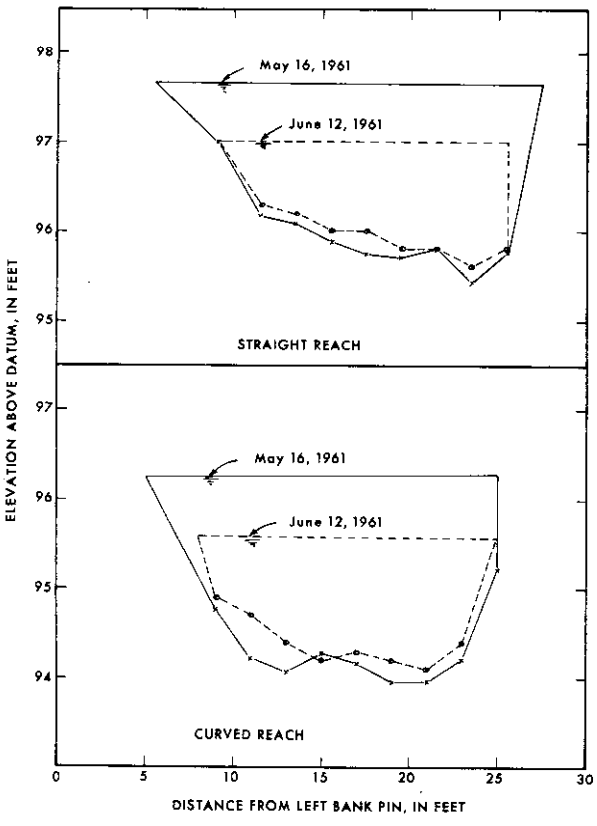


FIGURE 7. — Typical cross sections, Rio Grande del Ranchos, 1961.

aftermath of an unusually large flood from an early spring thaw. Because of streambed conditions left by the flood and because of the river stages at the times of this survey, these data are inconclusive in describing a scour pattern and are omitted from the present discussion.

Observations of Scour and Fill

On figures 9 and 10, profiles of the streambed and water surfaces are plotted against distance along the channel. Below these profiles is a plot of the net changes in cross-sectional areas of the bed and, separately, the cross-sectional area of flow. Values are considered scour if the bed elevations at high flow are lower than those at low flow. Thus, a negative area within this curve represents the total volume of material per foot of length soured from the streambed. A positive area represents the total volume of material brought in from upstream and temporarily deposited as a fill.

It is apparent that at the time of the high-stage measurements a scour generally extended over the entire reach. No marked difference in the magnitude of scour occurs between pool and riffle. Likewise, it appears to make no difference as to whether the measurements were observed in the straight or curved reach. The few sections where fill did occur could not be related to their location within the reach.

It might be expected that scour should be greatest at sections of smallest flow area which, for constant discharge, have the highest mean velocity. To test this hypothesis, flow areas at both high and low stages are plotted at the bottom of figures 9 and 10. No systematic correlation between the flow area and magnitude of scour is apparent. Since discharge is constant at a given stage, this plot may also be considered the mirror image of a comparison of velocity to magnitude of scour.

The absolute magnitude of flow area may not provide an ideal comparison for the influence of area. Some cross-sectional areas at low flow are larger than others at high flow. The relative increase in flow area from a low stage to a high stage would more accurately describe the changes that a particular section experienced. Superimposed as a dashed line on the flow area plot is the relative increase in cross-sectional flow area. Efforts to relate this relative increase to the magnitude of scour were also unsuccessful.

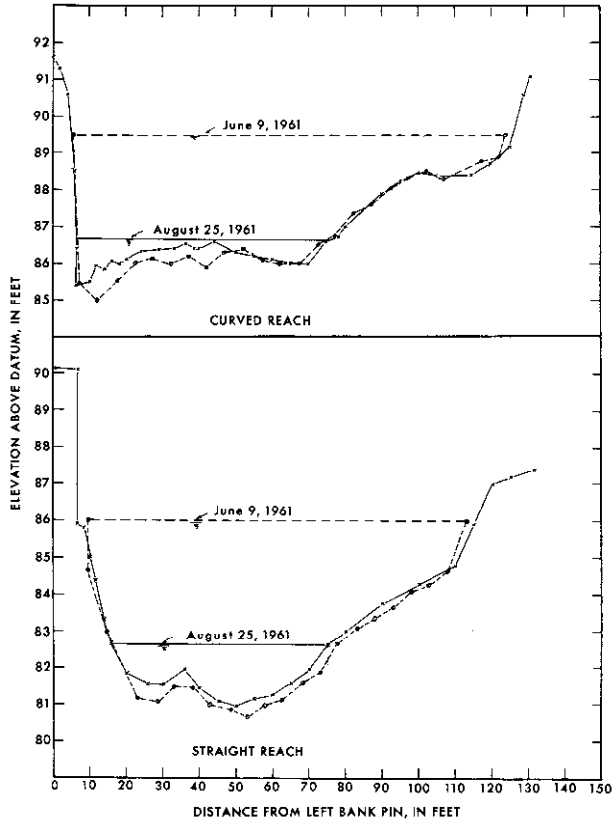


FIGURE 8. — Typical cross-sectional data, Popo Agie River, 1961.

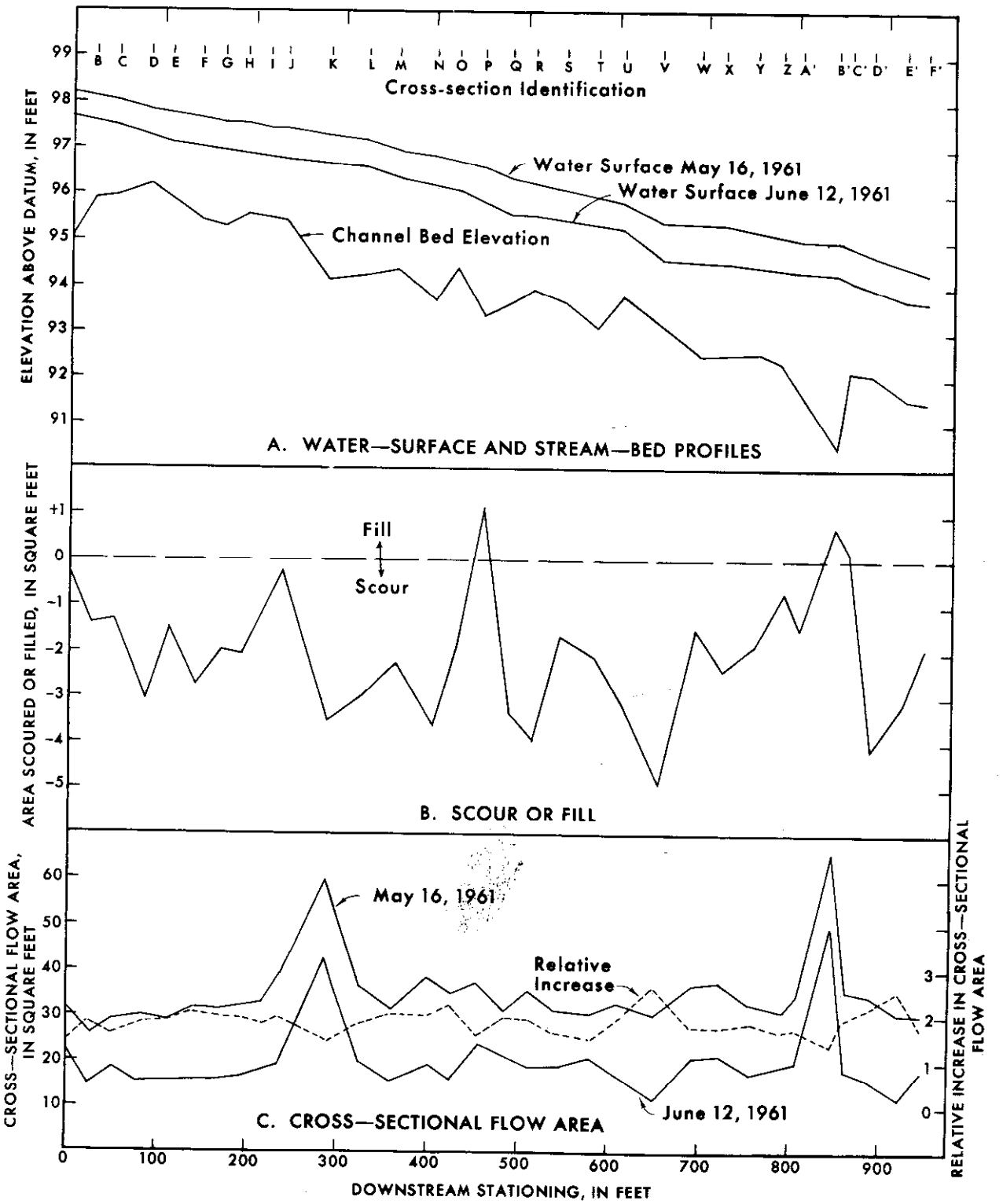


FIGURE 9. — Rio Grande del Ranchos near Taos, N. Mex.

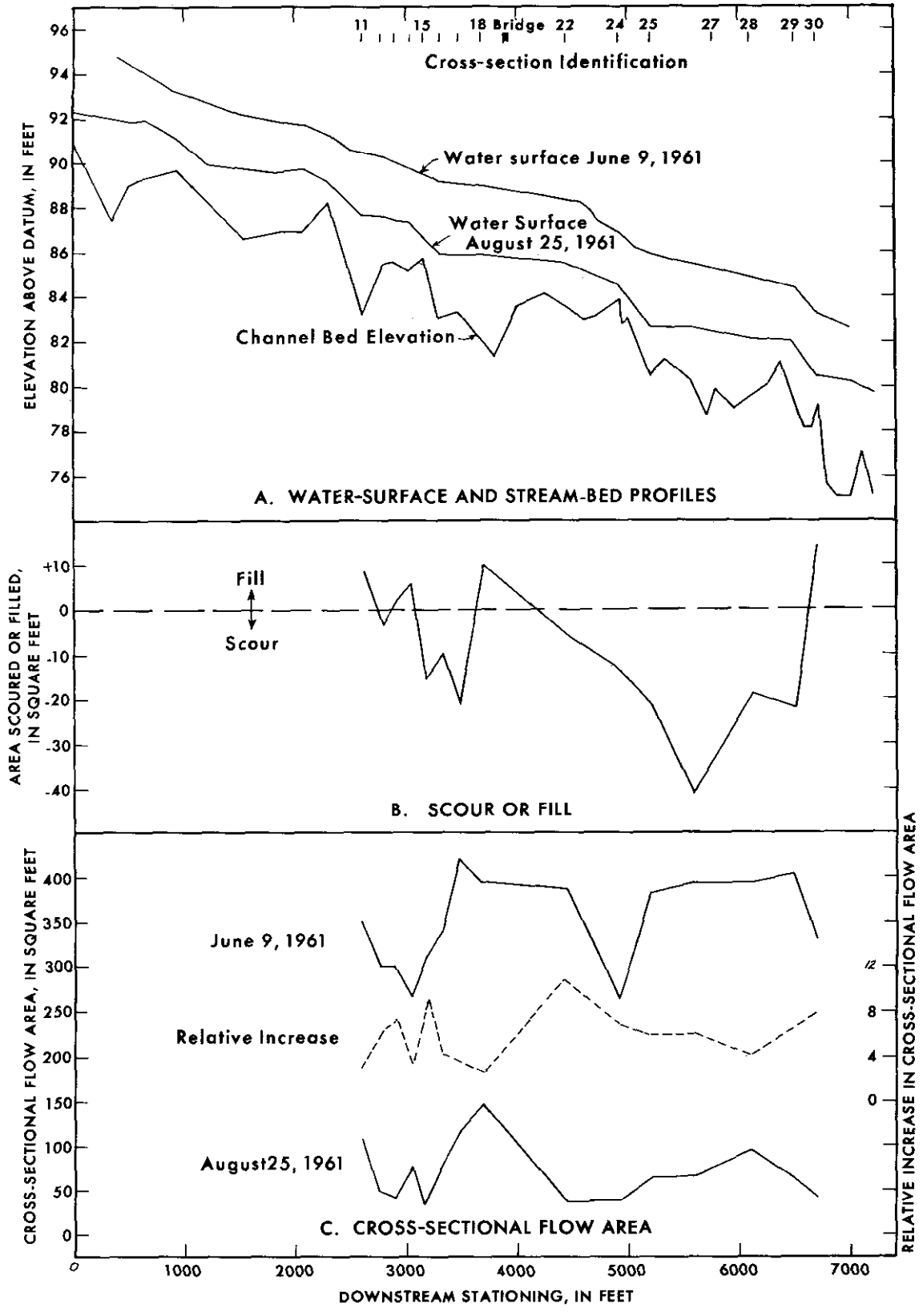


FIGURE 10.—Popo Agie River near Hudson, Wyo.

Similar comparisons were made for other physical dimensions of the stream. Data for channel widths and depths were given the same analysis as for flow areas. No meaningful correlation could be established between these channel properties and the magnitude of scour.

Discussion

At some time during a period of high flow, the channel bottom of fine gravel or sand streams may scour. At present it is not possible to list the characteristics of channels in which this occurs. One tends to think that the scouring action is limited to streams in semiarid areas, but the experience of the U.S. Geological Survey at stream-gaging stations shows that scour occurs during high flow in some rivers in all physiographic and climatic regions.

The extent of the scouring action appears to encompass the entire reach, without regard to any of the physical dimensions of the stream, and applies to pool as well as riffle and to curved as well as straight reach.

Values of scour indicate a large volume of sediment being lifted from the streambed. The data collected might lead one to imagine that this volume of sediment would appear as a block of material being picked up as a unit off the streambed and being carried downstream at a velocity approximating that of the water near the bed. But the interaction or interference between grains tends to reduce the mean speed of the particles which then move at a mean velocity very small relative to the velocity of the flow. This conclusion is not only suggested by the theory of queues but has been verified experimentally by Langbein.[2]

[2] LANGBEIN, W. B. Unpublished data.

[3] LANE, E. W., and BORLAND, W. M. RIVER BED SCOUR DURING FLOODS. Amer. Soc. Civil Engin. Trans. 119: 1954.

There is an apparent disparity between the observed magnitude of scour and rates of accumulation of sediment in reservoirs. Lane and Borland [3] showed that the volume of sediment computed by multiplying observed mean scour depth at a few cross sections by average channel width by channel length is much larger than the observed accumulation of sediment in a downstream reservoir. They argued that the observed scour was nonrepresentative of the whole reach and at unmeasured points fill was simultaneous. Particularly, they said, scour occurs only at the narrow sections of rivers where observations are usually made, whereas at wide sections, if observations were available they would show simultaneous filling.

The present data dispute that argument. We measured, albeit in reaches of modest length, scour in narrow as well as wide sections, in riffles as well as pools, and in curves as well as straight reaches. The observations are deficient, as pointed out earlier, in not showing what was happening in the reach at any one instant of time. Nevertheless, if scour followed by fill in a narrow section were contemporaneous with fill followed by scour in wide sections, the latter should show no scour or much less than that occurring elsewhere. Rather, the magnitude of scour was of the same order in all types of sections.

We interpret the process, then, quite differently from Lane and Borland. Scour is associated with dilation of the grain bed through the scour depth, but individual particles may move intermittently, and at a speed much less than that of the water. The volume of material scoured and moved may be large, but because of its low mean speed downstream, the whole volume does not move entirely out of a long reach but, in effect, is shifted downstream only a limited distance.

REPUBLICAN RIVER CHANNEL DETERIORATION

[Paper No. 47]

By WILLIAM L. NORTHROP, *chief, Water Control Section, U.S. Army Engineer District, Kansas City*

Synopsis

A loss of two-thirds of the operational channel capacity of the Republican River below the Trenton and Harlan County Dams since design of the projects has caused increased damage from local flooding and, if not rectified, will reduce the effectiveness of flood control storage provided in the reservoir system. A study was inaugurated to determine the cause of the loss and, if practicable, to recommend corrective measures. Many factors were involved, such as the sediment load, bed material, uncertainties in streamflow records due to shifting channels, a long drought period, reservoir regulation effects, agricultural encroachments, and an unexpected clogging of the channel by woody growth. A summary of the sediment and hydrologic factors is presented. It is concluded that, when assuming the initial operational capacity, the implications of a temporary increase in channel capacity due to a cycle of wet years were overlooked. Channel losses were caused principally by agricultural encroachment and clogging by phreatophytes. It appears practicable to restore that part of the channel capacity lost due to clogging by mechanical

removal of the woody growth and prevention of regrowth. Retention of woody growth on selected banks, construction of some cutoffs, and installation of light bank protection, will be required to control caving banks.

Introduction

Loss of operational channel capacity on the Republican River has caused increased damage from floods originating below the Harlan County and Trenton Dams and, if not rectified, will reduce the effectiveness of flood control storage provided in the reservoirs. Operational capacities of 10,000 c.f.s. below Trenton Dam and of 15,000 c.f.s. below Harlan County Dam were assumed in reservoir design studies on the basis of discharge data and flood damage information available in the 1940's. Subsequently, in making experimental releases from both projects in 1957, it was found that flows exceeding 3,500 and 4,000 c.f.s., respectively, would cause initial damage. Investigations have now shown that: (1) the original determinations of capacity were optimistic, in that channel enlargements due to a wet cycle, 1935-47, should have been discounted; (2) woody growth began to establish itself in the sandy bed of the stream in the early 1950's, simultaneously with the beginning of a 4-year drought period and, contrary to past history, had grown to sufficient height by 1957 to partially block the channel; and (3) farm crops planted to lower levels since 1950 have reduced the stage of initial flood damage by more than 1 foot. The total length of caving banks between Harlan County Dam and the mouth of the Republican River has been reduced from 121 miles in 1951 to 20 miles in 1961 due primarily to woody growth established on the banks during the drought period; however, the clogging of the channel by woody growth and increasingly unfavorable channel alinements seem to have aggravated some of the remaining caving bank areas.

Basin Description

Geography

The Republican River rises in northeastern Colorado and flows northeast, east, and southeast through Colorado, Nebraska, and Kansas to join the Smoky Hill River at Junction City, Kans., to form the Kansas River. Principal features are shown in figure 1. The basin is

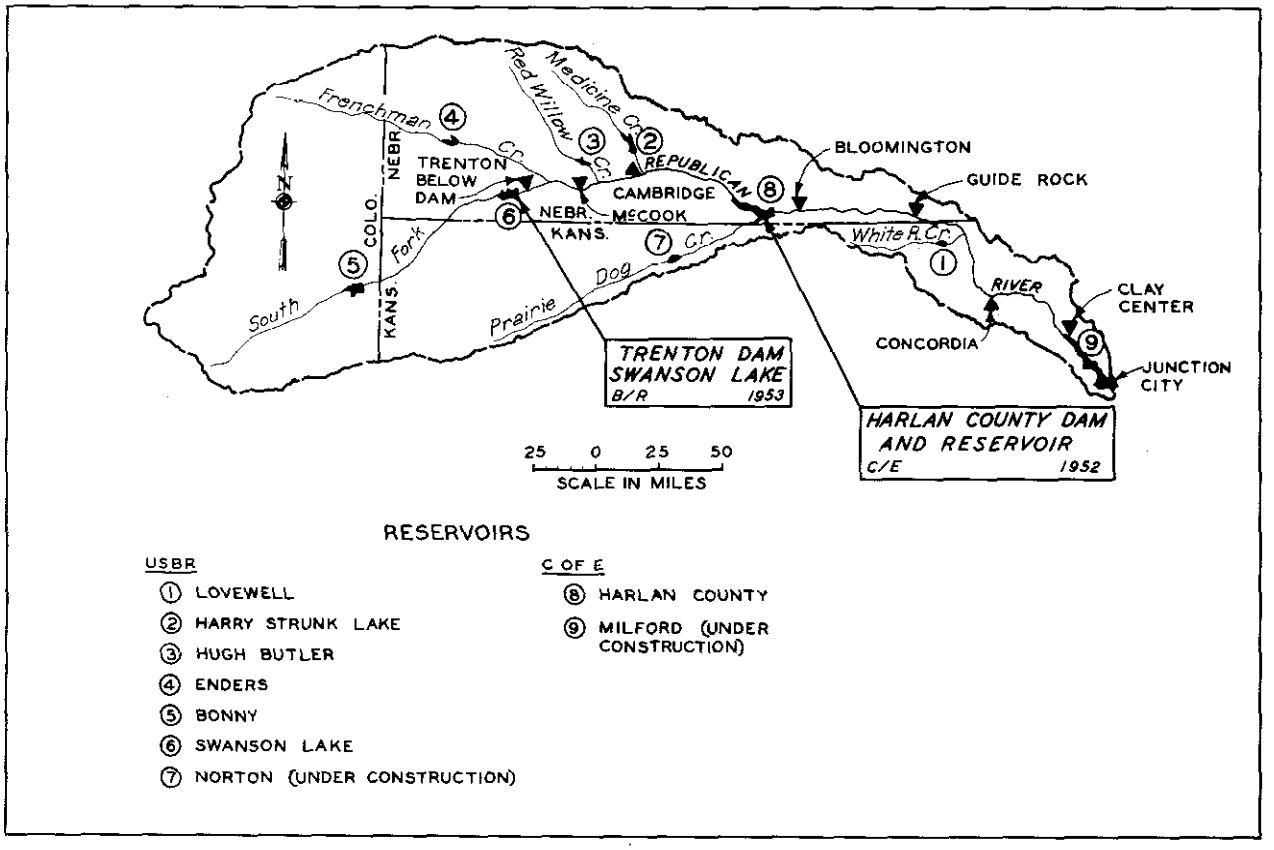


FIGURE 1. — Features of Republican River basin.

nearly 400 miles long from west to east, and it has a total area of 24,955 square miles. The western part is fan-shaped up to 120 miles wide, and it makes up about four-fifths of the basin area. The eastern part is a relatively narrow corridor, 160 miles long and 20 to 30 miles wide. The gross drainage area includes sandhills of the upper basin, estimated to cover 7,170 square miles, which do not contribute surface runoff. The main stem valley below Trenton Dam is relatively wide and a large percentage is used for agricultural purposes. An elaborate irrigation system associated with the reservoir projects is in the final stage of development. The Republican River is a heavy sediment bearing stream with a shifting sand bed and banks of easily erodible soil. Woody growths, principally of willows and cottonwoods beginning about half bank level and extending outward and onto back channel areas, have been recognized as characteristic growth. Caving banks on the outside of bends have plagued valley residents since the land was brought under cultivation.

Climate

Almost all of the Republican River basin lies west of the 97th meridian, which is generally accepted as the boundary between the semiarid plains to the west and the more humid region to the east. The climate is characterized by extremes. The mean temperature is 51° F., but extremes of minus 43° and plus 118° F. have been recorded within the basin. The normal growing season ranges from 150 days in the western section to 190 days in the eastern. The annual precipitation is 16 inches in Colorado and increases easterly to 30 inches in the lower basin. The rainfall rarely is well distributed, as there are localized cloudbursts and prolonged droughts in the weather picture. In May 1935, a maximum rainfall of 24 inches in less than 24 hours was reported at one location in the upper Republican River basin. At other times, droughts and duststorms have prevailed. The average annual snowfall of 27 inches normally does not contribute to flood runoff.

Streamflow

The variable climate produces wide fluctuations in streamflows. Observed discharges of the Republican River at Clay Center, Kans., during the 1934 drought were 1 c.f.s. on 9 to 11 August; and, within a year, the discharge reached 195,000 c.f.s. on 3 June during the 1935 flood. A recurrence of such extremes is not expected now that there is an appreciable degree of upstream reservoir control, but these values illustrate the climatic conditions that prevail in the basin. Presently, there are 48 stream-gaging stations operated in the basin by the U. S. Geological Survey. The average streamflow at the Clay Center gage (98 percent of the gross drainage area) is 1,079 c.f.s., or the equivalent of 0.57 inches of annual runoff.

Basin Development

A comprehensive plan for flood control and irrigation is nearing completion in the Republican River basin. The U.S. Bureau of Reclamation has completed five multiple-purpose reservoirs in the upper basin areas, one on White Rock Creek in the lower basin, and has a seventh project under construction. The U.S. Army Corps of Engineers has completed the Harlan County Reservoir and has the Milford Reservoir under construction. Extensive irrigation projects in the valley of the main stem and tributaries are associated with all reservoir projects. The two principal reservoirs directly related to the channel capacity problem are described below.

Harlan County Dam

Harlan County Dam and Reservoir on the Republican River in south-central Nebraska has a flood control allocation of 500,000 acre-feet and an irrigation, conservation, and sediment reserve pool of 350,000 acre-feet. The dam controls the runoff from an uncontrolled area of 9,277 square miles below the Trenton, Enders, Red Willow, and Medicine Creek Dams of the Bureau of Reclamation.

Trenton Dam

Trenton Dam on the Republican River was completed in 1953. The conservation pool and sediment reserve of 120,200 acre-feet form Swanson Lake, which has an area of 4,974 acres. In addition, the reservoir has a flood control allocation of 133,800 acre-feet. The gross drainage area at the damsite is 7,766 square miles, of which 5,891 square miles lie below Bonny Reservoir.

Flood Damages

Since the turn of the century, the Republican River basin has experienced many damaging floods. By far the most devastating flood of record occurred in 1935, causing an estimated $9,000,000 in damage at that time and level of development. This flood originated in the headwater areas and converted a relatively narrow channel, meandering through a pleasant green valley, to a sandy waste 1,000 to 3,000 feet in width. The channel damage decreased as the flood crest attenuated downstream. In 1947, a flood, originating principally between the Trenton damsite and the Harlan County damsite, also caused extensive damage. Since construction of the reservoir projects, the principal flood damage potential has been eliminated. However, floods originating below the dams have caused average annual damages of about $300,000 along the main stem since 1953.

Reduction in Channel Capacity

At the time Harlan County and Trenton Dams were being designed and constructed, the Republican River had a relatively large channel capacity and gave some indications of further increase. A review of the discharge measurements at pertinent stations indicated a capacity below Harlan County Dam of 15,000 to 20,000 c.f.s. The nominal flood stage at Bloomington, which was then 1.2 feet higher than at present, was apparently used to establish the upper limit of regulated reservoir releases. Likewise, a value of 10,000 c.f.s. was used below Trenton Dam. After the reservoirs were constructed in the early 1950's, a severe drought period followed and it was not until the spring of 1957 that the occasion arose for making substantial reservoir releases. At this time, less than a third of the assumed channel capacity was found to be available below Harlan County Dam and limitations were equally serious below Trenton Dam.

Discharge Relations

Stage-discharge relations developed from U.S. Geological Survey streamflow measurements at Bloomington, Nebr., shown on figure 2,

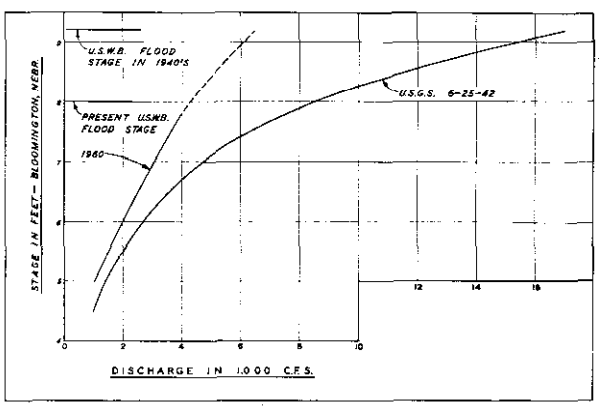


FIGURE 2. — Stage-discharge relations in 1942 and 1960, Republican River near Bloomington.

illustrate generally the changes that have been experienced in channel capacity between Trenton Dam and the Kansas-Nebraska State line. Deterioration below the State line gradually diminishes. It is recognized that single-line rating curves at all Republican River stations are an oversimplification, due to shifting channels and occasional forced movement of gaging equipment. Nonetheless, by analysis of flow records and field observations, it has been established that the usable channel capacity for nondamaging reservoir releases below Harlan County Dam, which had been assumed to be 15,000 c.f.s. in 1943 to 1945, was reduced to 4,000 c.f.s. in 1957. It is evident that about half the 11,000-c.f.s. loss in operational capacity is due to lowering the flood stage a little more than 1 foot since that time, and the remaining loss is an actual reduction in the carrying capacity of the channel. The reduced flood stage reflects the extension of agricultural crops to lower land during the dry years of the 1950's. This was accelerated, no doubt, by the construction of Trenton and Harlan County Dams, which many people in the valley assumed would eliminate all flood problems and permit cultivation of previously unused low areas.

Flow Velocities

Reductions in channel capacity, as indicated by an analysis of the rating curves over a period of time, can be observed also by a close inspection of measured velocities at the discharge stations. At two stations, Bloomington and Guide Rock, measurements at the same sections were made before and after the deterioration in channel capacity. The situation at Bloomington is shown on figure 3. Maximum velocities have been reduced from 8.7 to 6.7 feet per second and the area of high velocities has been reduced substantially. In areas now blocked by woody growth, velocities have been reduced from 5.0 to 1.0 feet per second. Discharges at approximately the same gage height were 8,130 c.f.s. in 1951 and 3,760 c.f.s. in 1961.

Channel Areas

Surveys of degradation ranges below the reservoirs indicate considerable change in channel configuration. Some sections have increased in total area and others have decreased. In most sections, a discernible trend is toward a well-defined narrow central channel and an increase in low island and shore bar heights. Summary data on the ranges below Harlan County Dam indicate very little average change in cross-sectional area for the entire reach between 1951 and 1957, although individual sections have changes up to 15 percent of the area (table 1). Changes were fairly well distributed between the upper and the lower half of the sections. Similar data for the ranges below Trenton Dam, shown on table 2, indicate a net loss between 1949 and 1958 of 7 percent in the average total channel area and a loss of 19 percent in the average lower half. Of the entire average loss 87 percent occurred in the bottom half of the channel and loss at one location was 40 percent of the entire section.

Woody Growth

In 1952, a profuse growth of willow seedlings, mixed with cottonwood seedlings in some places, was observed on many islands and shore bars in

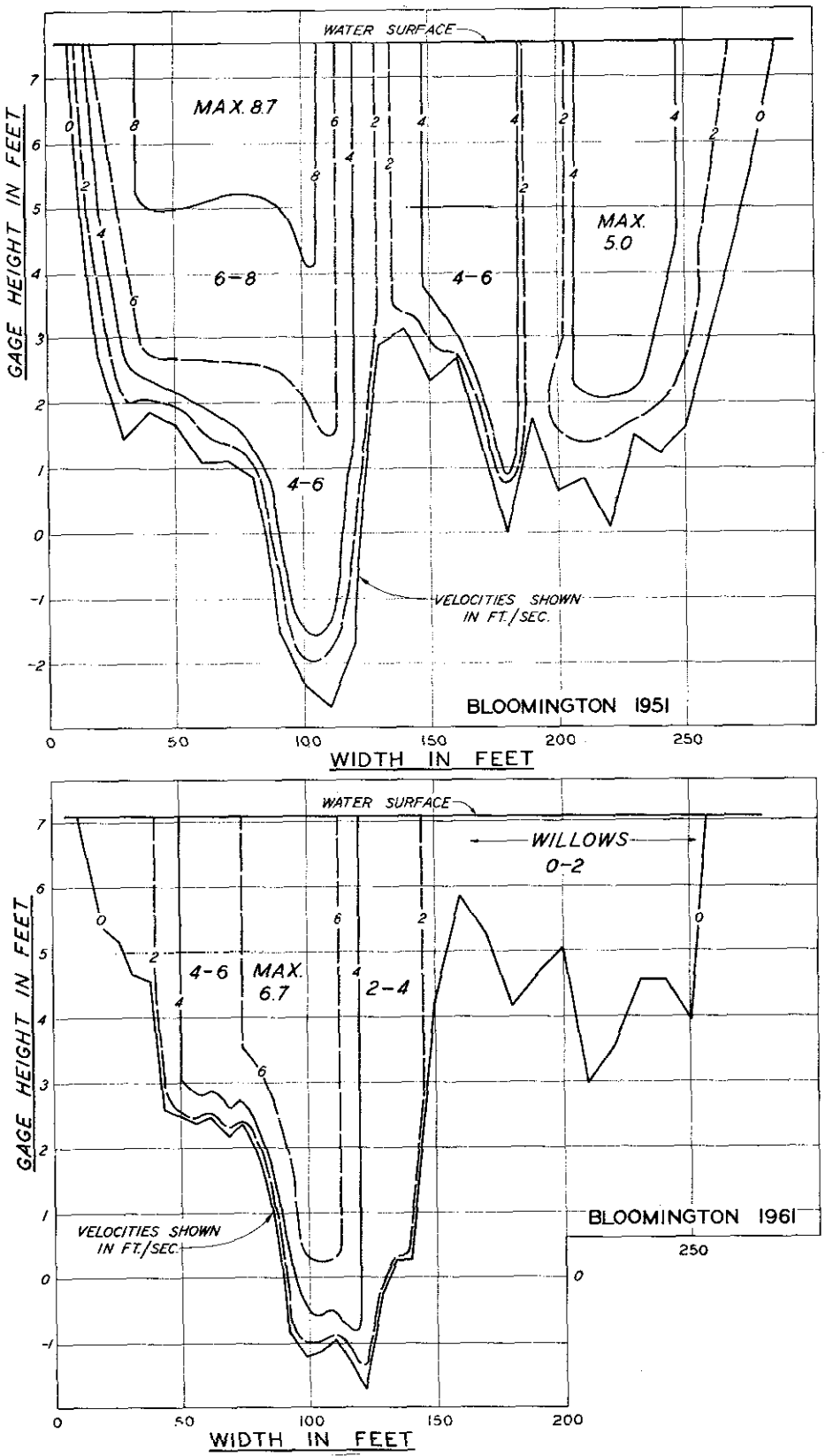


FIGURE 3. — Measured velocities of Republic River near Bloomington, 1951 and 1961.

TABLE 1.—*Summary of degradation range surveys below Harlan County Dam*[1]

Range No.	River miles (1961)	Miles below dam	Bankfull area[2]					Half-bankfull area				
			1951	1957	Change	1961	Change[3]	1951	1957	Change	1961	Change
			Sq. ft.	*Sq. ft.*	*Sq. ft.*	*Sq. ft.*	*Sq. ft.*	*Sq. ft.*	*Sq. ft.*	*Sq. ft.*	*Sq. ft.*	*Sq. ft.*
1–2	237.9	1.5	5,027	5,365	+338	.....	.....	1,469	1,555	+86	.....	.....
3–4	236.5	3.9	4,221	4,326	+105	.....	.....	1,587	1,629	+42	.....	.....
5–6	235.2	4.2	3,248	3,150	−98	.....	.....	1,304	1,213	−91	.....	.....
7–8	232.8	6.6	5,477	5,613	+136	.....	.....	1,832	1,824	−8	.....	.....
9–10	231.7	7.7	3,234	2,770	−464	.....	.....	1,003	728	−275	.....	.....
11–12	229.3	10.1	2,595	3,088	+493	.....	.....	1,645	1,776	+131	.....	.....
13–14	227.2	12.2	3,339	3,508	+169	3,455	−53	1,512	1,746	+234	1,673	−70
15–16	224.6	14.8	3,720	3,531	−189	3,429	−102	1,859	1,646	−213	1,486	−160
17–18	221.9	17.5	3,533	4,166	+633	4,093	−73	659	1,053	+394	1,070	+17
19–20	215.0	24.4	3,077	2,864	−213	.....	.....	1,086	886	−200	.....	.....
21–22	208.5	30.9	2,458	2,371	−87	.....	.....	800	717	−83	.....	.....
23–24	190.6	48.8	3,508	3,115	−393	.....	.....	1,484	1,033	−451	.....	.....
25–26	177.7	61.7	3,848	4,250	+402	.....	.....	1,283	1,520	+237	.....	.....
27–28	160.1	79.3	2,176	2,568	+392	.....	.....	688	826	+138	.....	.....
Total	.....	.....	49,461	50,685	1,224	.....	.....	18,211	18,152	−59	.....	.....
Average	.....	.....	3,533	3,620	+87	.....	.....	1,301	1,297	−4.2	.....	.....

[1] Dam at 239.4 river miles.
[2] Maximum depth used 12 feet.
[3] Change from 1957.

the main channel below Harlan County Dam. Prior to this time, the channel had remained essentially free of woody growth for many years, although these species have always grown in back channels and in many places near or above top of bank level. Little significance was attached to this new growth at the time, as it was assumed that the next flood would clean the channel. As it happened, a prolonged drought, depletion of streamflow by irrigation, and the initial filling of the two main stem reservoirs resulted in little or no flood activity along the

TABLE 2.—*Summary of degradation range surveys below Trenton Dam*[1] *keeping maximum depth down to 8.0 feet, which in several cases is below bankfull*

Range No.	River miles (1961)	Miles below dam	Bankfull area			Half-bankfull area		
			1949	1958	Change	1949	1958	Change
			Sq. ft.	*Sq. ft.*	*Sq. ft.*	*Sq. ft.*	*Sq. ft.*	*Sq. ft.*
7	366.1	8.4	2,200	2,071	− 29	471	408	− 63
8	363.8	10.7	3,341	2,567	−774	1,086	599	−487
9	360.2	14.3	1,992	2,094	+102	457	680	+223
10	358.4	16.1	2,255	1,760	−495	1,022	549	−473
13	354.4	20.1	2,758	2,806	+ 48	1,260	1,248	− 12
13a	353.9	20.6	1,880	1,770	−110	1,022	633	−389
14	348.8	25.7	1,602	1,498	−104	593	500	− 93
14a	347.1	27.4	1,492	1,380	−112	437	380	− 57
15	345.6	30.9	2,060	1,790	−270	548	420	−128
16	341.9	32.6	2,381	2,608	+227	578	612	+ 34
17	334.7	39.8	2,811	2,788	− 23	1,160	1,165	+ 5
18	330.5	44.0	3,259	2,560	−699	900	480	−420
20	323.9	50.6	3,127	2,605	−522	1,305	775	−530
22	317.7	56.8	2,596	2,414	−182	958	501	−457
23	313.6	60.9	2,882	2,694	−188	1,230	1,010	−220
23a	312.6	61.9	2,958	3,040	+ 82	785	866	+ 81
24	312.1	62.4	2,734	2,136	−598	1,080	625	−455
25	310.7	63.8	3,250	2,985	−265	1,345	1,087	−258
26	308.1	66.4	3,970	3,185	−785	1,355	1,100	−255
27	305.7	68.8	3,110	2,880	−230	1,225	1,045	−180
28	301.9	72.6	4,820	5,170	+350	1,990	2,110	+120
28a	298.0	76.5	3,973	3,753	−220	1,695	1,660	− 35
29	296.2	78.3	3,554	2,134	−1,420	1,240	510	−730
31	290.1	84.4	3,140	4,360	+1,220	1,233	1,380	+147
32	284.6	90.1	3,280	3,830	+550	1,595	1,975	+380
34	276.7	97.8	2,810	2,133	−677	920	706	−214
Total	.....	.....	74,235	69,011	−5,124	27,490	23,024	−4,466
Average	.....	.....	2,855	2,654	− 197	1,057	886	− 172

[1] Dam at 374.5 river miles.



DISCHARGE 50 C.F.S. 17 OCT. 1961 BLOOMINGTON STAGE 0.9 FT.



DISCHARGE 436 C.F.S.
BLOOMINGTON STAGE 2.6 FT.
24 MARCH 1951

REPUBLICAN RIVER

RIVER CHANNEL
WILLOW ENCROACHMENT

FIGURE 4. — Willow encroachment in the Republican River, 1951 and 1961.

river to the Kansas-Nebraska State line until 1957. By 1955, local residents expressed concern about the willow growths blocking the channel. At that time the growth had reached a height of 4 to 7 feet and the area of younger growth had extended from Trenton Dam to the downstream reaches in Kansas. By 1957, the older growth was 7 to 15 feet high and accretions of sand up to 3 feet in depth had formed in the growths thus increasing the height of the islands and bars. Much of the channel area is now covered by willows and cottonwoods of a size that could be dislodged only by the most extreme floods or by mechanical means. Deposit of sand in the larger growth has increased to 5 feet in depth at some locations. Fortunately, aerial pictures of the channel below Harlan County Dam were made in 1951, and between Trenton Dam and Harlan County Reservoir in 1958. In 1961, the reach below Harlan County Dam was photographed again. Measurements were made of the area of growth between high banks. Figure 4 shows a typical reach of channel in 1951 and how it has been changed by 1961. Figure 5 shows that 40 to 60 percent of the 1951 channel area has been occupied for a major part of its length by woody growth.

Bank Erosion

Comparisons of the 1951 and 1961 aerial photographs between Harlan County Dam and Clay Center show that the willow growth has had the overall beneficial effect of reducing bank erosion from 121 miles to 20 miles. This reduction in the extent of caving banks is attributed principally to the growth that was established on the banks during the 1950 drought period. Reduced current velocities due to growth in the channel have also been beneficial from this viewpoint, except in instances where deposits in the growth have increased velocities next to the banks. Instances may be found where the concave (eroding) side of river banks have filled in with willows and sand accretion. In tentatively establishing the outside limits for channel restoration purposes, a deviation from the 1951 channel has been made, when possible, to take advantage of the protective cover provided by recent willow growth. At many critical locations, local interests have successfully used

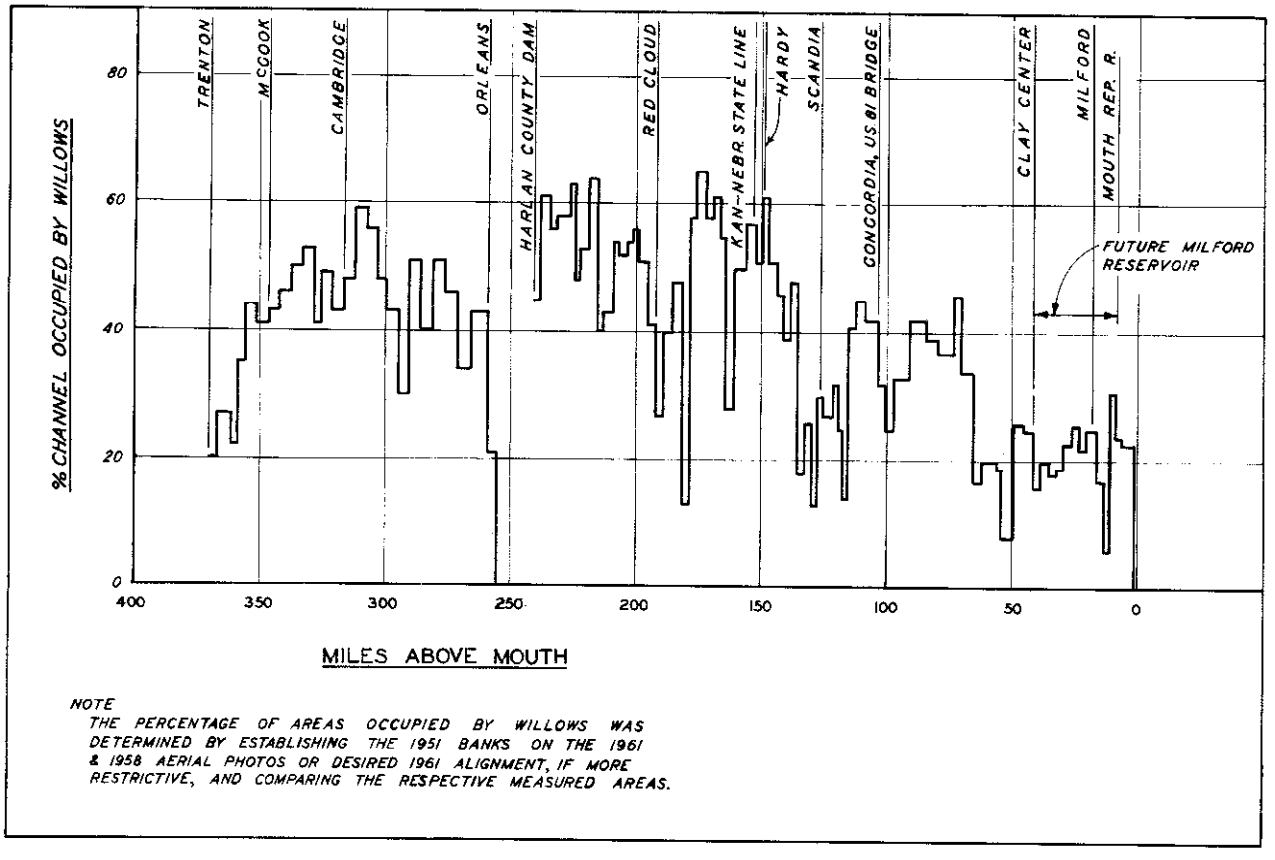


FIGURE 5. — Percentage of channel occupied by willows in Republican River.

old car bodies and other types of improvised light revetment to prevent further erosion and induce sedimentation. The areas of active bank erosion that remain will present a problem so long as corrective measures are not taken. A substantial part of seriously eroding bank areas can be greatly improved or eliminated by completing cutoffs that can be made at comparatively low cost. Some areas would be improved by removal of woody growth from the main channel. It is anticipated that light revetment such as car bodies, and continued encouragement of bank growth will be required in addition to cutoffs and channel clearing to completely eliminate undesirable caving banks.

Hydrologic History

Records

Fragmentary records of stream discharge of the Republican River are available dating back to 1894. Several gaging stations were established in the 1920's and good records are available since the mid-thirties. Major floods have often washed out gage installations, and actual current meter measurements of floods much in excess of flood stage were virtually impossible prior to construction of high bridges in the last 20 years. Although the quality of records has improved as access to stations during floods became possible, due allowances must be made in considering changes in channel capacity.

Flood Peaks and Runoff

The erratic hydrologic history of the Republican River is illustrated by the annual maximum discharges and annual runoff values given on figure 6 for the station at Bloomington. From the beginning of the record in 1929 through 1951, more than half of the peak annual maximum discharges at this station were in the 10,000 c.f.s. range, 4 years exceeded 15,000 c.f.s., and 2 years, 1935 and 1947, were 260,000 c.f.s. and 140,000 c.f.s., respectively. With a few exceptions, the annual runoff volumes varied about as the peak values. There was an abrupt break in the pattern after 1952 when the peak discharge varied from 1,000 to 5,000 c.f.s. Low discharges after this time were the result of Harlan County Reservoir regulation following initial filling of the conservation pool in 1957. The record low annual runoff volumes beginning with water year 1953 and extending through 1956 is explained by sustained drought conditions during this period.

Channel Capacities

The channel capacity that can be related to the gage at Bloomington is roughly approximated by the top line on figure 6 for the present state of valley development. Comparison with the peak discharges and runoff volumes also shown in the figure indicate a rough correlation with channel capacity. There was a gradual improvement in channel capacity from the 4,000 to 5,000 c.f.s. range prior to the 1935 flood to 16,000 c.f.s. after the 1947 flood. Subsequently, there was a comparatively rapid deterioration to 4,000 c.f.s. in 1957. A small, but discernible, improvement is indicated in the last few years, which had near normal volumes of flow. During the study of Republican River channel conditions, similar diagrams were prepared for other stations. The improvement between 1935 and 1947 is not so regular and pronounced at some of the other stations, but the deterioration between 1952 and 1957 is evident. A particular exception was noted for the reach below Trenton Dam where the improvement since 1960 was lacking. In fact, deterioration is still in progress at Cambridge about midway between Trenton Dam and Harlan County Reservoir. Analysis of records from 1917 to 1934 at Culbertson (near Trenton Dam) and Wakefield (near Clay Center) indicate that there had been a fairly stable channel during this earlier period. One investigator found evidence that a flood which may have been comparable to the 1935 flood occurred in 1826. If channel scour comparable to 1935 condition occurred in 1826, the 1917-to-1934 stabilized channel, with 4,000 to 5,000 c.f.s. capacity, would be indication of what could be expected to develop eventually without reservoirs to change the flow regimen.

Flow Duration

Duration studies were made for all stations with substantial records. Curves for three significant periods of record, 1934-52, 1953-56, and 1957-61, are shown in figure 7 for the station at Bloomington. These curves confirm the extremely low flows during the drought period 1953-56, and show that floods since 1956 are somewhat less than the normal experienced prior to 1952. The analysis of flows below Trenton Dam indicated the same tendency during the drought period in the 1950's. Whereas the drought period was also evident at the lowest station, Clay Center, the variation from normal was not as pronounced.

Pool Operation

Impoundment of flow in the conservation pools of several headwater reservoirs began about 1950. Harlan County Reservoir was placed in operation for conservation storage in October 1952, and Trenton Dam (Swanson Lake) began conservation operation late in the calendar year 1953. Lovewell, a small tribu-

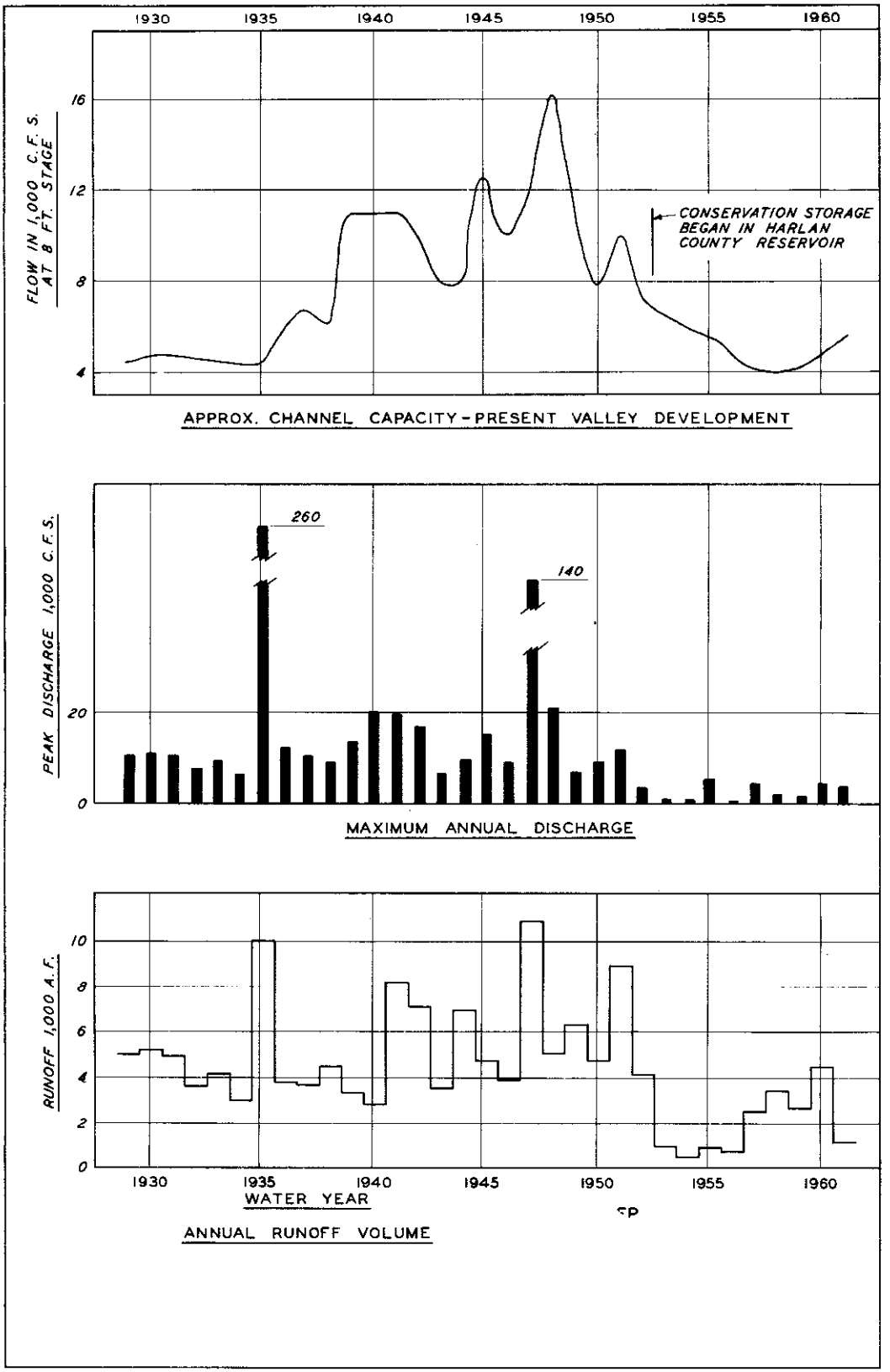


FIGURE 6. — Hydrologic history of Republican River at Bloomington, 1930-60.

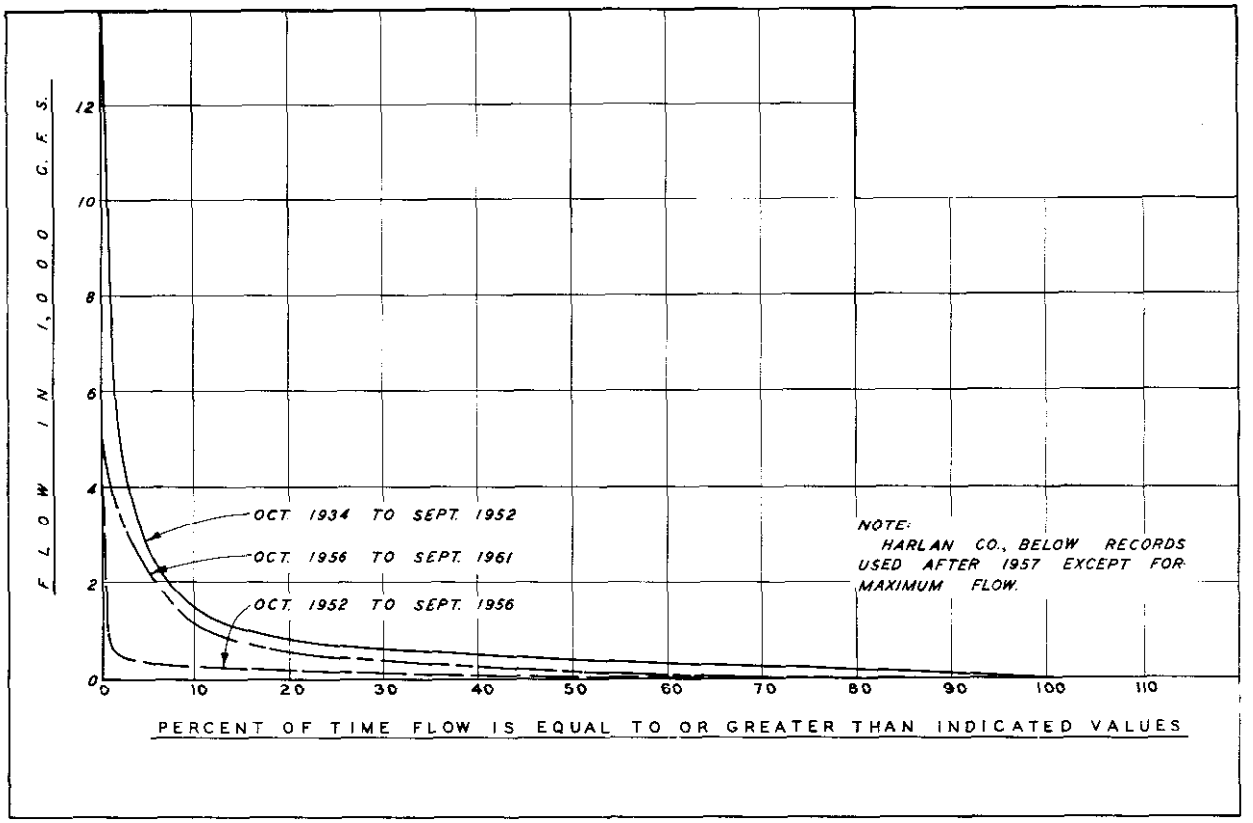


FIGURE 7. — Durations of flow below Harlan County Dam at Bloomington.

tary reservoir in the lower basin, began impoundment in 1957, and Milford Reservoir, now under construction, is scheduled to begin impoundment of conservation storage in 1966. Initial filling of Harlan County Reservoir and Swanson Lake was completed in the spring of 1957. Releases to the channel up to about 350 c.f.s. were made at both projects for irrigation during initial filling. Beginning in 1957, reservoir releases have been limited to about 2,500 c.f.s. at Trenton Dam and 4,000 c.f.s. at Harlan County Dam, due to reduced downstream channel capacities. Usually the channels are not used to full capacity, as some margin of safety is desirable for unpredictable local inflow, and some uncertainty usually exists as to the exact capacity that should be used. Substantially more volume of flood storage has been available to release from the Harlan County Reservoir since 1957 than from the Trenton Dam. Preliminary studies indicate a channel capacity of 7,000 c.f.s. and 10,000 c.f.s., respectively, below Trenton Dam and Harlan County Dam would be desirable for effective regulation of the Republican River reservoir system. Capacity, in addition to these values, would, of course, be desirable if a practicable method can be devised to make it available.

Sediment

Channel Characteristics

The Republican River is a naturally meandering sand-bottom stream with slopes gradually increasing from 3½ feet per mile at the mouth to 9 feet per mile at Trenton Dam. The river valley averages about 1 mile in width and is formed generally of sandy soil that is easily eroded. Streambanks are usually well defined and average about 8 feet in height between Trenton Dam and Harlan County Dam and about 12 feet in height downstream from Harlan County Dam. Banks well below the average, with adjacent low bottom land, are found at irregular intervals. The banks are prone to erode on the outside of bends when not protected by natural vegetation or by artificial measures. The channel has made many natural cutoffs, usually during extreme high water. The back channels normally are soon occupied by a growth of willows and cottonwoods, which accelerate the entrapment of sediment and closure of the channels to normal river flows. A few

natural islands are to be found, but for the most part all inbank flow is carried in one channel. Bottom widths average 600 feet below Trenton Dam and 450 feet between Harlan County Dam and the mouth. The sand bed is shaped into islands and shore bars that fluctuate in size and location from one flood to the next when not held in place by vegetation. Woody growth and related sediment accumulations on low islands and bars, as described previously, has impaired bankfull channel capacity and has in some instances accelerated caving banks in the remaining active areas.

Sediment Data

Sediment data have been collected on the main stem of the Republican River at eight locations. These data have been used to estimate sediment loads shown in table 3.

Streambed Material

Early in 1961, about 150 surface samples of the streambed material were collected along 200 miles of the river channel below Harlan County Dam. The samples were divided equally between (1) the low-water channel, (2) bars not exceeding 2 feet above low water, and (3) bars 2 to 5 feet above low water. Lenses of silt, which constitute roughly a fifth of the material in islands and bars built up by sediment deposits, are usually found at lower levels and are not reflected in the surface samples. As shown in table 4, the surface of the high bars and islands contain a significantly higher percentage of the fine sands. The bed of the low-flow channel was probed with a ½-inch rod 7 feet in length. The bottom of the sand was not reached except in some instances where the channel was adjacent to the hills that define the alluvial plain. In these instances penetration was limited to 4 or 5 feet. It appears from field observations that bed material between Trenton and Harlan County Dams is of a similar nature.

TABLE 3.—*Sediment load of Republican River at selected stations*

Station	Average annual load in tons			
	Before reservoirs		With reservoirs [2]	
	Suspended sediment	Bedload [1]	Suspended sediment	Bedload [1]
Trenton, Nebr.	1,230,000	.....	15,000	.....
Orleans, Nebr.	5,600,000	450,000	920,000	310,000
Harlan County Dam, Nebr.	.....	.....	17,000	.....
Bloomington, Nebr.	5,700,000	475,000	60,000	160,000
Guide Rock, Nebr.	.....	.....	390,000	200,000
Hardy, Nebr.	.....	.....	725,000	.....
Concordia, Kans.	.....	.....	1,420,000	.....
Clay Center, Kans.	8,323,000	500,000	2,580,000	380,000

[1] Estimated by Straub's equation in Missouri River 308 Report.

[2] These estimates are based on normal runoff below the dams and on trap efficiency of near 100 percent.

High Bar and Island Composition

Typical high bars and islands formed by sediment deposits in vegetative growth were sampled to the low-flow elevations at 10 locations, and sections exposed by erosion were observed at other places. The samples were found to be composed of sand similar to the surface samples previously described, except layers of silt were found at random locations in the vertical section. Most islands and many of the bars are protected by stranded driftwood that will require removal if a willow-clearing program is to be effective. Sieve analyses of the samples are summarized in table 5.

It was found expedient to sample and analyze the sand and silt layers separately. A field estimate indicates the silt layers constitute about 20 percent of the high bar and island material. The weighted average size-distribution shown above was based on this observation. While there were variations from place to place, no definite trend in size-distribution was established in relation to river mileage or other iden-

TABLE 4.—*Summary of Republican riverbed material samples, 1961*

Location of samples	Percentage of material retained by sieve sizes								
	¾ (20 mm.)	⅜ (10 mm.)	4 (4.8 mm.)	10 (2 mm.)	18 (1 mm.)	40 (0.44 mm.)	70 (0.22 mm.)	200 (0.070 mm.)	Less than 0.005 mm.
Low flow channel:									
Increments	0	0.4	1.9	7.1	14.5	46.7	25.6	3.8	0
Summation [1]	0	.4	2.3	9.4	23.9	70.6	96.2	100.0	100.0
Low bars:									
Increments	.1	.4	1.5	6.3	12.9	40.5	32.0	4.9	0.4
Summation [1]	.1	.5	2.0	8.3	21.2	61.7	93.7	98.6	100.0
High bars:									
Increments	.2	.1	0.4	2.0	4.5	30.0	44.0	18.6	0.2
Summation [1]	.2	.3	.7	2.7	7.2	37.2	81.2	99.8	100.0
Average:									
Increment	.1	.4	1.3	5.2	10.6	39.2	33.9	9.1	0.2
Summation [1]	.1	.5	1.8	7.0	17.6	56.8	90.7	99.8	100.0

[1] Summation is percent coarser.

TABLE 5.—*Particle-size distribution of sediment deposits of Republican River*

Sample	Percentage of materials retained								
	Fine gravel (4.8 mm.)	Sand					Silt		Clay
		Very coarse (2.0 mm.)	Coarse (1.0 mm.)	Medium (0.44 mm.)	Fine (0.22 mm.)	Very fine (0.075 mm.)	(0.05 mm.)	(0.10 mm.)	(0.005 mm.)
Average of sand samples:									
Increments	2.0	2.0	10.0	37.0	32.0	9.0	6.0	1.0	1.0
Summation[1]	2.0	4.0	14.0	51.0	83.0	92.0	93.0	99.0	100.0
Average of silt samples:									
Increments	.....	.....	.....	2.0	17.0	16.0	47.0	5.0	13.0
Summation[1]	.....	.....	.....	2.0	19.0	35.0	82.0	87.0	100.0
Weighted average: Sand, 80 percent Silt, 20 percent	.....	2.0	10.0	30.0	29.0	10.0	14.0	2.0	3.0

[1]Summation is percent coarser.

tifiable features. Schematic sections of a typical island are given on figure 8.

Willow Root System

Since the composition of the root system has considerable significance in determining methods that might be adopted for removal of objectionable islands and bars, a study was made to determine its nature. The principal woody

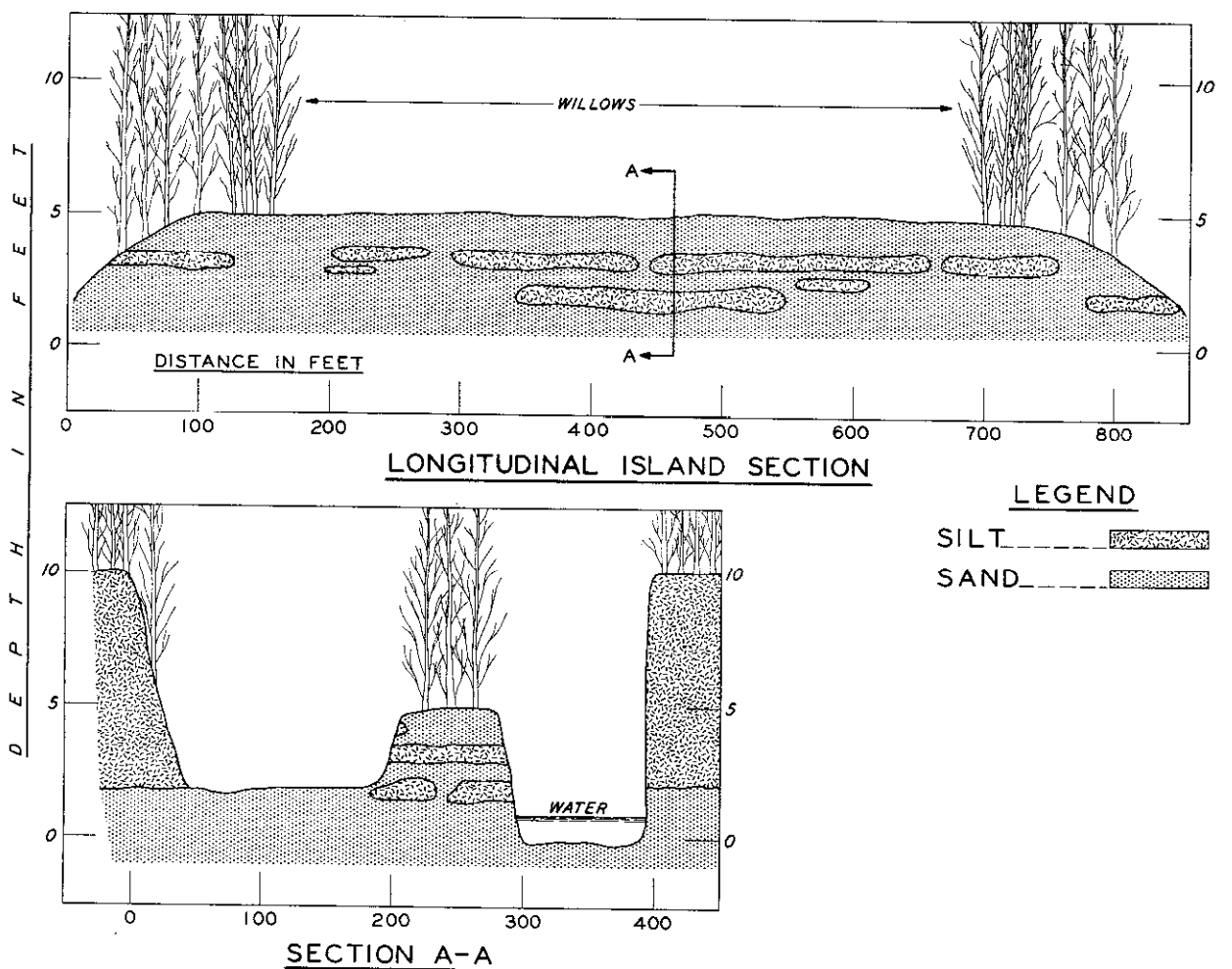


FIGURE 8. — Schematic sections of typical island in Republican River.

growth on the islands was found to be willows. The sides of selected islands were excavated from top to bottom to expose the root systems. The entire volume between low water and top of island was well-filled with roots varying from threadlike size to a half-inch in diameter. Many of the willows apparently started just above low water and the center stem extends upward to the surface with roots branching off at all levels, while others obviously started at higher levels and have extended small roots down to low water. Figure 9, showing typical root systems, was prepared from photographs of the exposed roots.

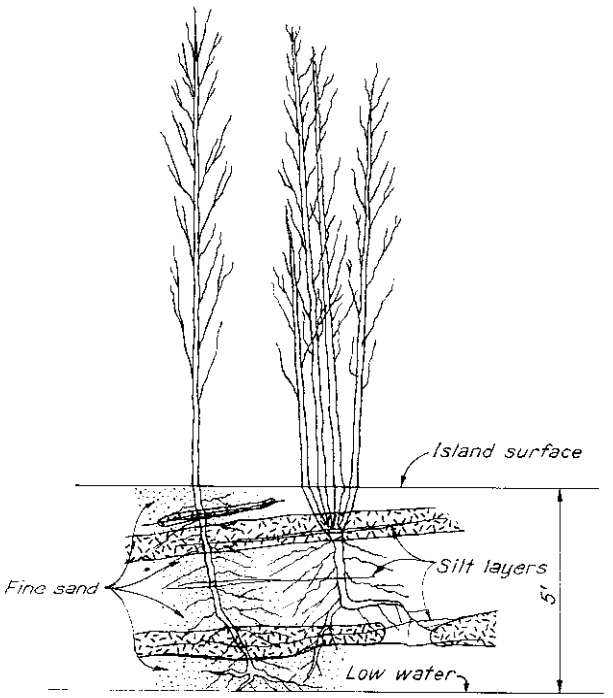


FIGURE 9. — Willow root development.

Roughness Coefficients

A typical 5½-mile reach of the channel below Harlan County Dam, extending from Bloomington downstream, was selected for field tests to evaluate channel-roughness factors, including the effect of willows that cover many of the islands and bars. The channel between high banks was about 60 percent occupied by willows. Occasional cottonwoods were noted. Twenty cross-sections were established, three of which were located at degradation ranges. Discharge measurements were made at six of the sections for regulated releases up to 3,500 c.f.s. Analysis of the discharge measurements and related stream slopes indicated average "n" values of 0.20 and 0.036 for willow-covered bars and for sand, respectively. The results of these tests were used to compute a rating curve at Bloomington for both actual conditions and with willows removed from the main channel, as shown on figure 10. This computation method does not include an allowance for the probable increased efficiency due to better flow configuration with the willows removed and the sand bed smoothed out by floodflows. It was not practicable to extend the tests to evaluate this factor.

The increase in channel capacity resulting from willow removal is dependent on willow area, channel area, and channel capacity. It appears reasonable to assume that the ratio of the percentage of increase of capacity in a reach resulting from willow removal to the percentage of willows in the reach is constant. On this basis, the results in the test reach can be applied to other reaches and the effects of willow removal for the entire Republican River can be evaluated.

Engineering Studies

General

Serious concern over Republican River channel deterioration began in the spring of 1957 when the reduced channel capacities became apparent. Several reconnaissance-type field studies and analyses of available office data were made during the period 1957-60 to evaluate the situation as it was related to reservoir regulation requirements. In December 1960 the problem was called to the attention of the Missouri River Division Board of Sediment Consultants [1] and suggestions of the board members were followed in the collection of field data and the analysis of available information, with a view to formulating recommendations for corrective action, if such should be indicated.

Findings

Principal factors related to the loss of channel capacity and possible future restoration as indicated by the studies are summarized as follows:

A. The loss of channel capacity appears to have been caused by three principal factors:

(1) Extensive use of former low wasteland since 1951 for farm crops from Trenton Dam to the mouth of the Republican River has lowered the maximum upper level of regulated release over 1 foot and accounted for roughly 50 percent of the loss.

(2) Partial blocking of the main channel, disruption of the flow pattern by permanent woody growth on former low bars and islands, and de-

[1] The Missouri River Division of Sediment Consultants Board is composed of H. A. Einstein, L. G. Straub, and Vito Vanoni.

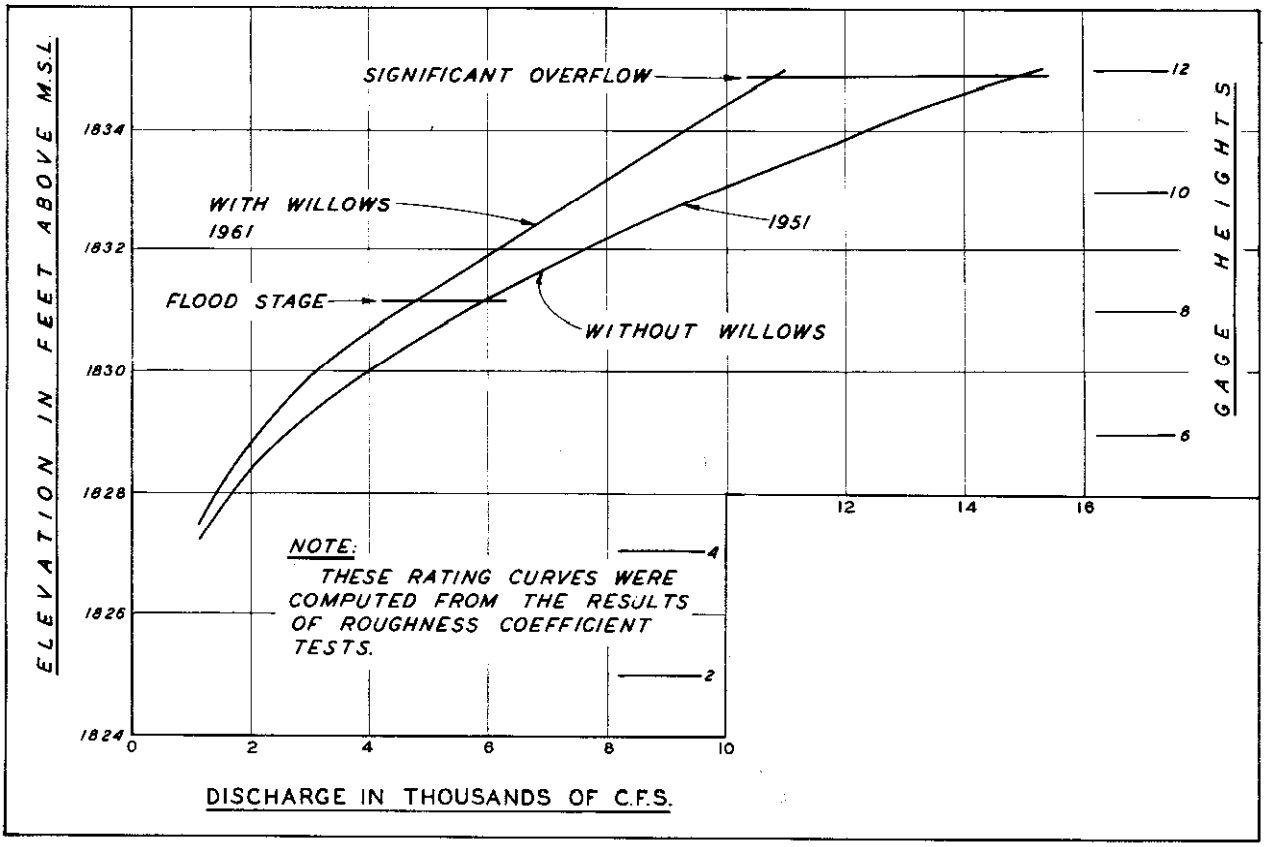


FIGURE 10. — Republican River stage-discharge relations near Bloomington, 1951 and 1961.

posits of bed material up to 5 feet in depth in the woody growth have accounted for the remaining loss of channel capacity.

(3) Initial filling and subsequent regulation of the Swanson Lake (Trenton Dam) and Harlan County Reservoir for flood control have eliminated or greatly reduced the natural flood peaks that served to flush the channel and in that way has contributed, to an undetermined extent, to the loss due to woody growth and deposits on the islands and bars.

B. There is evidence that the former enlargement of channel capacity to 15,000 c.f.s. or more below Harlan County Dam was due to a combination of the 1935 flood and the succeeding 12 years of a sustained wet cycle. It is likely that a repetition of the 1826-1934 climatic and streamflow cycle would have caused the channel to assume the same 4,000 to 5,000 c.f.s. capacity available before the 1935 flood. Essentially the same reasoning applied to the channel below Trenton Dam.

C. The grain-size of the sandbars and islands, the root structure of the willows, and the configuration of the bars and islands indicate that the channel would be readily eroded to former natural conditions by bankfull streamflow after clearing of the woody growth, decay of the roots, and the removal, where present, of snags or resistant obstructions at the heads of the islands and bars.

D. Much of the channel capacity, which was lost during the past decade, could be restored by removal of the woody growth and maintenance of the denuded areas.

E. Removal of woody growth from bars and islands between the high banks and completion of the more obvious cutoffs will alleviate existing caving banks to the extent that remedial measures can be taken with light treatment, such as plantings and dumping of car bodies.

Conclusion

Hydrologic data, sediment information, and aerial mapping accumulated prior to recognition of the channel deterioration proved invaluable. These data combined with the results of a short period of intensive field work made possible a fruitful engineering study and evaluation of the problem. This experience again emphasized the advisablity of analyzing in advance the potential channel changes that may be

expected when the flow regimen is changed. As a result of the studies, it was possible to recommend corrective measures in this particular situation. It is anticipated that the results of the study also will be of value in planning and administering water-control programs in the surrounding areas with similar channel, flow, and climatic characteristics.

Acknowledgments

Much of the material incorporated into this paper was prepared originally for the Republican River Review Report which is nearing completion in the Kansas City District. Many individuals in the Planning and Reports Branch, under the direction of Willis G. Fish, contributed to the studies. The writer is especially indebted to Alexander M. Gow and Charles A. Burgtorf for review of the text and many valuable suggestions.

DEGRADATION STUDY OF THE MIDDLE MISSISSIPPI RIVER, VICINITY OF ST. LOUIS

[Paper No. 48]

By THOMAS F. MAHER, *hydraulic engineer, U.S. Army Engineer District, St. Louis*

Synopsis

The results of a study of the relation in time between hydrographic surveys and stage and discharge observations in the vicinity of St. Louis, Mo., are presented. An attempt has been made to include as much basic data as are obtainable. Data for the analysis were obtained from the files of the U.S. Army Engineer District, St. Louis. Analysis of results shows accelerated degradation was brought about by works of man.

Degradation Study

St. Louis, Mo., lies on the right, or west, bank of the Mississippi just downstream of the confluence of the Missouri River. The drainage area above St. Louis is divided into two major river basins, the Missouri and upper Mississippi, which have a combined area slightly in excess of 700,000 square miles. Approximately 530,000 square miles lie in the Missouri River basin and the remaining 170,000 square miles in the upper Mississippi River basin.

The flood seasons in the Missouri and upper Mississippi basins occur during the period, March to July. Except for periods of drought, the low water season is generally in the winter months, December through February.

The mean discharge of the Missouri River is about 82,000 cubic feet a second with an estimated maximum of about 900,000 in 1844 and a minimum of about 4,000 in 1940. The mean discharge of the upper Mississippi River is about 95,000 cubic feet a second with an estimated maximum of 565,000 in 1851 and 1858. Minimum discharge occurred in 1948 with a discharge of about 8,000 cubic feet a second. The mean discharge at St. Louis, Mo., is about 175,000, with an estimated maximum of about 1,300,000 in 1844 and a minimum of about 20,000 in 1933.

Fragmentary and intermittent river stage records were kept in St. Louis, Mo., between 1837 and 1860. Complete gage records are available for the year of 1844, when the greatest flood of record occurred. The first continuous records of stage were initiated in 1861, when the present Market Street gage was established at mile 179.6 above the mouth of the Ohio River. Stage and flow records stated herein are referred to the Market Street gage.

The earliest streamflow measurements taken at St. Louis were made by the city engineer of St. Louis in 1866. However, flow measurements were estimated as early as 1837. Since 1866, streamflow measurements have been made by the city of St. Louis, Mississippi River Commission, U.S. Corps of Engineers, and the U.S. Geological Survey. Although discharge measurements have been taken at several localities in the area, the results are all referred to the Market Street gage.

Hydrographic surveys covering parts of the area under discussion, from which it is possible to chart the changes in bed and bank configuration, are available since 1837. The first comprehensive survey was made by Lt. Robert E. Lee in connection with his work on the St. Louis Harbor.

Data from historic maps reveal that the Mississippi River at Vine Street in St. Louis was 3,100 feet wide in 1803. Due to shifting and cutting of the Illinois channel currents eastward of Bloody Island, the river gradually enlarged to 3,200 feet in 1808, 3,700 feet in 1837, 3,900 feet in 1843, and reached a maximum of 4,200 feet in 1849. Because of the large commerce resulting from the steamboats of the era, it became necessary to preserve the channel in front of the city of St. Louis, which was deteriorating rapidly. The magnitude of the interests of this wealthy and rapidly growing city demanded that the river, throughout the

entire length of the 13 miles of the city river front, be made to flow in a permanent channel having stable banks. To effect this, the city and private corporations began work in 1838 on a series of dikes from the Illinois shore, intended to confine the river in a definite channel. The Federal Government recognized the magnitude of the work and initiated work based upon the plan proposed by Lt. Robert E. Lee. Limit lines finally adopted for the St. Louis Harbor were 1,500 feet at low water (L.W. 1863 = zero on gage) and 2,000 feet at bankfull (30 feet on gage). The river at St. Louis as it existed in 1837 is shown on figure 1. On figure 1 is also shown the river at the present time in dashed lines. It will be noted by comparison the contraction from about 4,000- to 2,000-foot width.

In order to show the change in channel regimen resulting from the contraction work, comparative cross sections are shown on figure 2. The cross sections are at approximate mile 180 just downstream at Eads Bridge. For the earlier sections the deeper water is on the Illinois side, whereas after completion of the contraction works in about 1875, there is a gradual and marked deepening of the Missouri channel. The Missouri channel was at elevation 380+ in 1837 and is at elevation 360− in 1963. To illustrate this bed lowering further, channel thalweg profiles are plotted and shown on figure 3. The channel profiles extend from about mile 175 to mile 185 and show the overall lowering of the bed from 1837 to date.

Individual cross sections or channel profiles are not always conclusive as to what is happening in a channel, because the surveys performed may not be phased with the phenomenon. Discharge measurements over a long period, referred to a permanent gage, will portray changes in channel regimen. On figure 4 is shown the stage for a constant discharge of 40,000 cubic feet a second from 1837 to date. In 1837, 40,000 cubic feet a second passed the St. Louis-Market Street gage at a plus 6-foot stage, whereas in 1963 this same discharge passes at stage of minus 6-foot. This is a lowering of 12 feet in about 120 years. The rate of lowering has been almost constant since 1875. Table 1 shows a list of selected discharge measurements.

It is not the intention of this paper to claim that there is no filling of the channel due to bedload movement. The purpose is to show the general long term trend. There have been numerous occasions in the past when the bed elevation has fluctuated markedly, the cause being attributed to hydrologic conditions in the upstream basins. One documented fluctuation occurred in the winter of 1951-52 after the great Kansas River basin flood of 1951. This flood carried tremendous amounts of bedload material, which was deposited not only in the channel but in the extensive overbank areas below the junction of the Kansas and Missouri Rivers. Due to the flatter slopes of the Mississippi in relation to the Missouri, large channel depositions occurred in the Mississippi below

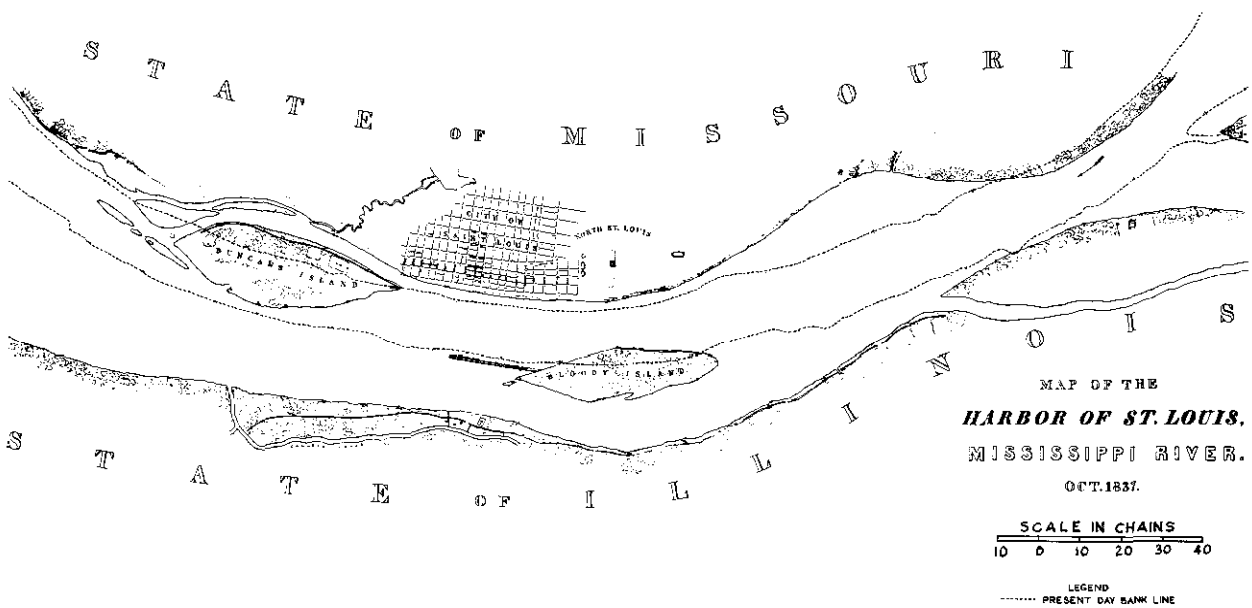


FIGURE 1. — Present-day and 1837 channel configurations.

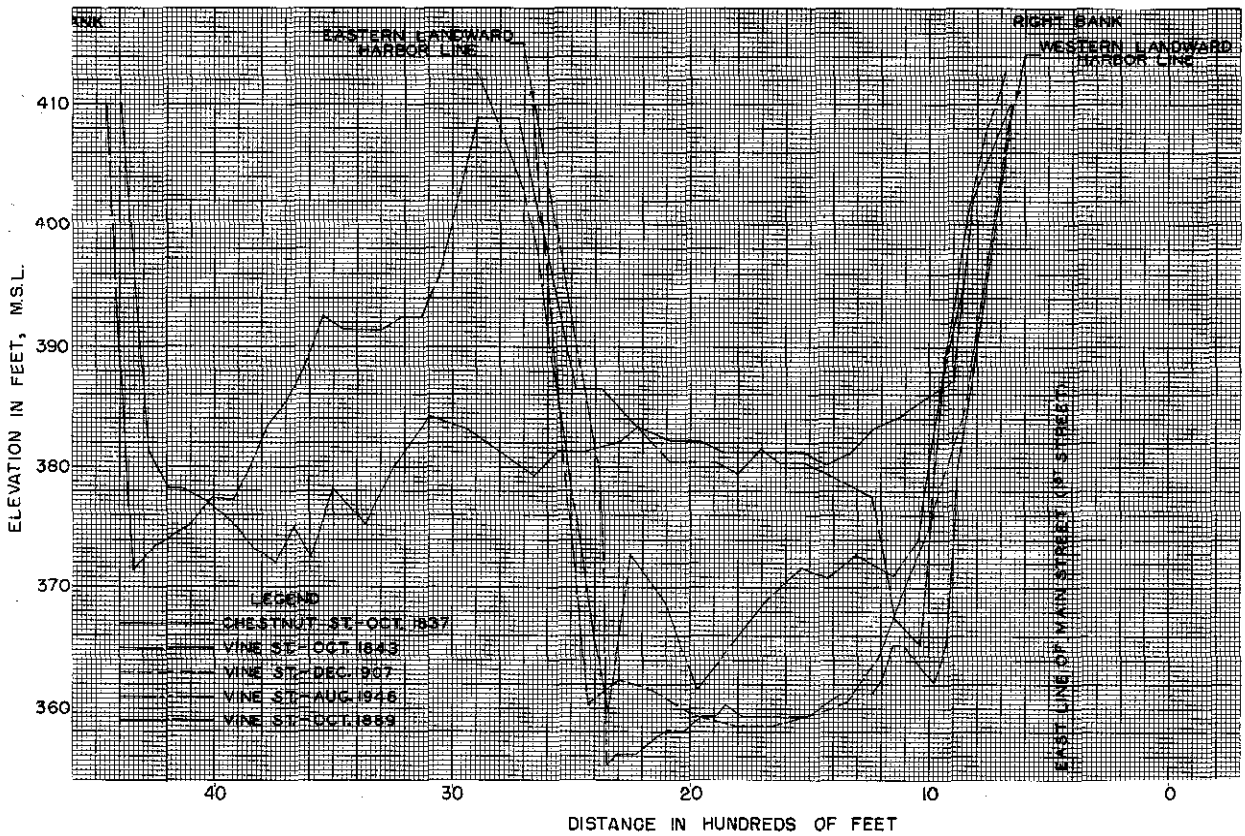


FIGURE 2. — Cross sections of Mississippi, downstream of Eads Bridge, 1837, 1843, 1889, 1907, and 1946.

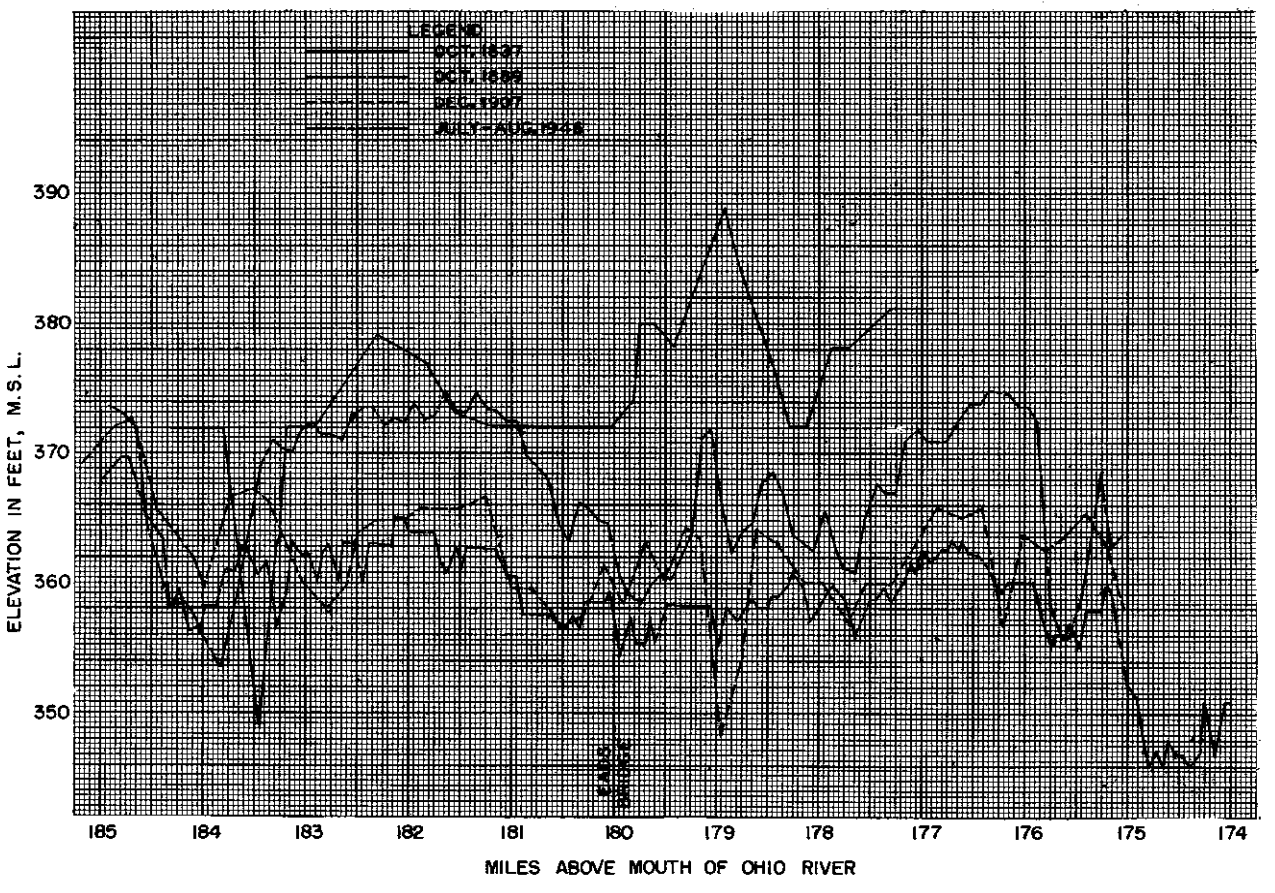


FIGURE 3. — Bed profiles of Missouri channel, 1837, 1889, 1907, and 1946.

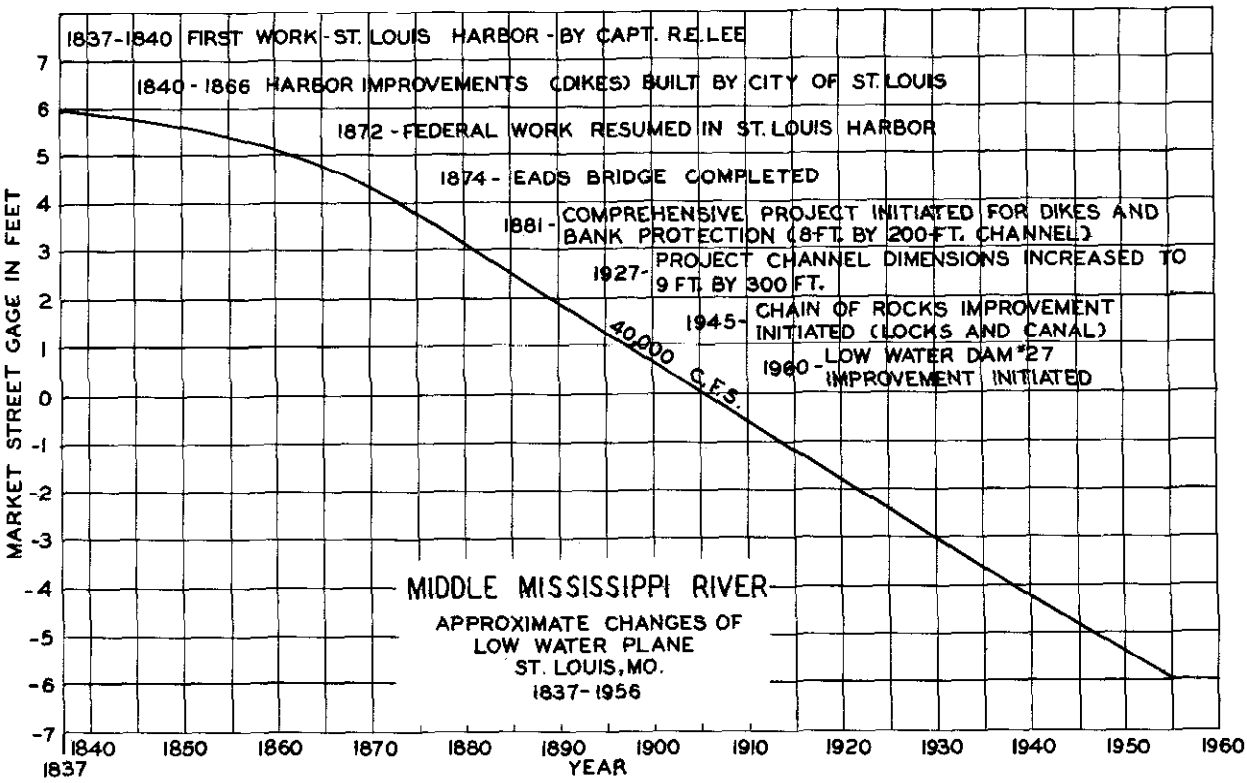


FIGURE 4. — Stage-time graph for 40,000 c.f.s., Mississippi River at St. Louis, 1837-1955.

the mouth of the Missouri. In the winter, generally, the upper Mississippi flow contribution at St. Louis is greater than the Missouri; consequently, in January 1952, this deposition moved through the St. Louis Harbor. Figure 5 shows comparative cross sections at mile 180.2, prior to and when the dome was moving downstream. The height of the dome was about 20 feet. A channel profile showing extent of and comparisons with previous profile is shown on figure 6.

In conclusion, it will be interesting to find out what affect the main stem Missouri basin reservoirs will have on the rate of long term change in the degradation of the channel at St. Louis, Mo.

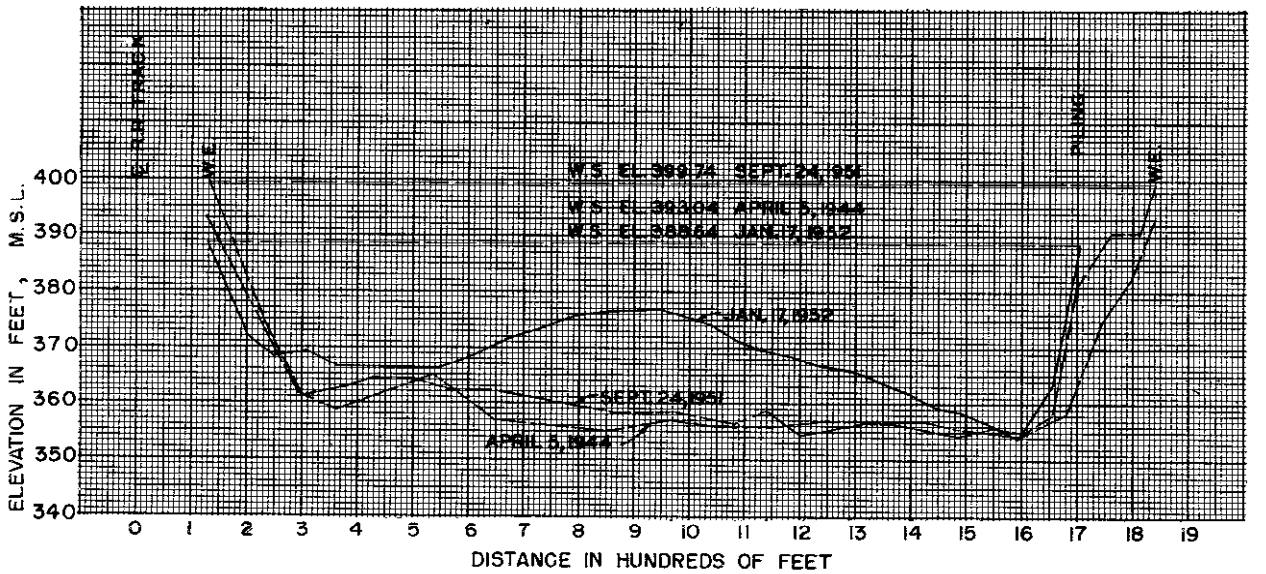


FIGURE 5. — Cross sections of Mississippi River at mile 180.2, prior to and after 1951 flood.

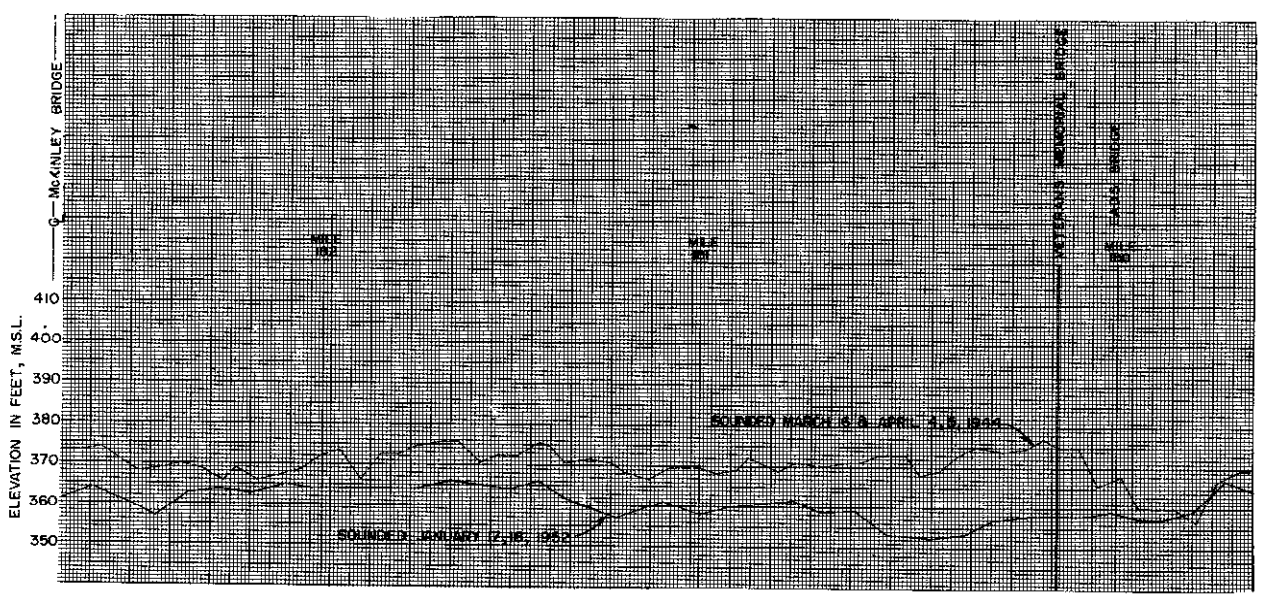


FIGURE 6. — Longitudinal channel profile, Mississippi River, 1944 and 1952.

TABLE 1.—*Discharge measurements, St. Louis, Mo.*

Date	Stage	Discharge[1]	Measurement method
	Feet	*1000 c.f.s.*	
14 Dec. 1880	5.4	51	Meter.
27 Jan. 1881	8.0	45	Meter (ice).
5 Feb. 1900	0.	35	Rod float.
7 Jan. 1901	.6	40	Meter.
16 Jan. 1902	1.3	44	Rod float.
22 Dec. 1903	2.3	51	Do.
21 Dec. 1904	.4	43	Do.
22 Dec. 1904	.2	39	Do.
3 Jan. 1905	1.5	54	Do.
16 Dec. 1910	−1.2	33	Do.
17 Dec. 1910	−1.5	34	Do.
31 Dec. 1910	.2	43	Do.
16 Jan. 1913	− .4	39	Do.
15 Jan. 1914	− .1	44	Do.
8 Jan. 1923	0.	48	Floats.
26 Dec. 1929	− .8	51	Do.
31 Dec. 1930	−1.3	41	Do.
3 Jan. 1931	−1.5	39	Do.
4 Jan. 1931	−1.0	42	Meter.
21 Oct. 1932	− .1	55	Floats.
1 Jan. 1934	−3.21	42	Meter.
31 Aug. 1934	−3.0	40	Do.
31 Aug. 1936	−4.0	38	Do.
16 Dec. 1937	−4.7	38	Do.
18 Dec. 1937	−3.5	40	Do.
2 Oct. 1939	−3.5	40	Do.
5 Feb. 1940	−3.3	34	Do.
13 Feb. 1940	−4.7	38	Do.
4 Feb. 1948	−2.5	50	Do.
28 Jan. 1954	−3.9	46	Do.
27 Jan. 1956	−4.4	45	Do.
8 Jan. 1959	−3.9	48	Do.
31 Jan. 1961	−3.9	49	Do.

[1]Discharge per foot about 5,000 to 6,000 c.f.s. at extreme low stages.

EFFECT OF A RIVER CONSTRICTION

[Paper No. 49]

By SAM SHULITS, *Professor of Civil Engineering, The Pennsylvania State University*

Abstract

The Susquehanna River at Middletown, Pa., is constricted considerably by a long embankment. The resulting downstream deposition has caused recorded progressive changes in the river course. Almost a mile of riverbank has already suffered substantial accretion that is expected to extend further downstream for miles with subsequent changes in the course of the river.

The Constriction and the Prior Condition of the Susquehanna River at Middletown

The Susquehanna River is constricted at Middletown, Pa., by a runway (levee) completed in 1957 (fig. 1). The total width of the river at Sassafras Island, originally 4,610 feet, is now 4,050 feet, which represents a narrowing of about one-eighth. The width of the channel between the left, or north, bank and the island has been reduced from 2,260 to 1,700 feet, corresponding approximately to a 25 percent reduction in width of that channel. The conditions in the slack water region just downstream from the constriction are the topic of this investigation. The study area is shown in figure 1.

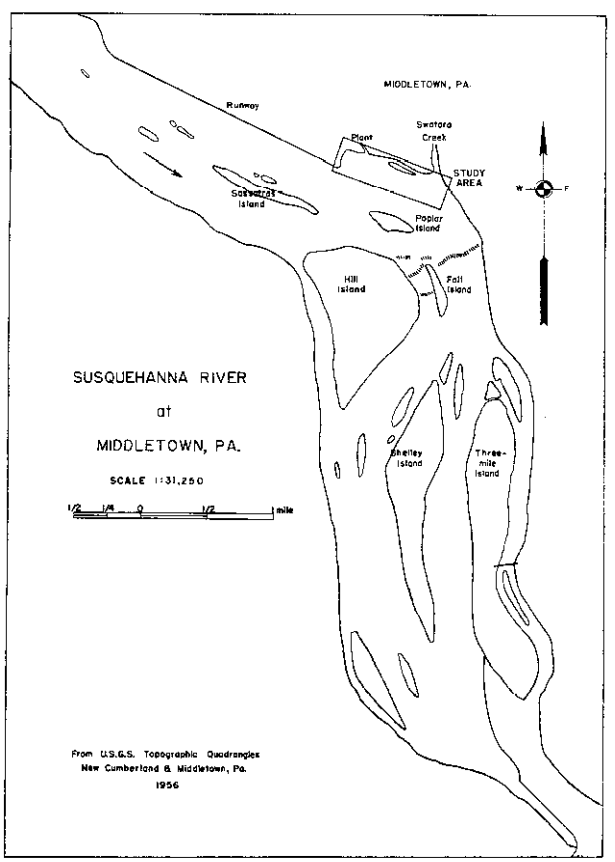


FIGURE 1. — Plan of Susquehanna River at Middletown, Pa., 1956.

The first aerial photograph of record is for the year 1937 (fig. 2). Noted should be the solitary island at the left bank between the intake and discharge channels of the electric generating station, and Swatara Creek (at the right of the photograph). This island will be named either "The Battery" or "Original Island" hereinafter. Subsequent aerial photographs taken in 1947, 1950, 1955, and 1956 show no discernible change in the river. Just upstream from the mouth of Swatara Creek in figure 3, there is a bar formation that should be borne in mind for subsequent discussion.

The incomplete constriction (white) appears first in figure 4, which was taken at a rather low flow (3,160 c.f.s.) on September 5, 1957. Here can be detected the first change in the slack water area; namely, a shoal about 200 or more feet riverward from the upstream end of the original island. The island and the bar at the mouth of Swatara Creek appear connected as part of the same river-developed formation also visible on figure 3, which was taken in 1950 at a flow of 6,780 c.f.s. This formation will play a role in the rationale to be developed later about the probable future configuration of the Susquehanna River.

Conditions Subsequent to the Constriction

The rapidity of the deposition in the front of the plant and just downstream from the levee after its completion is definitely demonstrated by figure 5, photographed in 1959, at a discharge of 3,950 c.f.s. As this is not an accurate aerial survey photograph, the magnitude of the accretion may be surmised from the 200-foot width of the runway at the lower left. The shoal mentioned above as barely appearing in figure 4 is now new land attached to the head of The Battery. The discharge from the plant, now trapped between the accreted land, the island, and the original shoreline, flows toward Swatara Creek. Formerly, this discharge flowed away from the shore out into the Susquehanna River. In figure 5 can be seen the bar extending



FIGURE 2. — Condition of Susquehanna River, November 23, 1937; discharge, 32,800 c.f.s.

from the toe of the embankment. Somewhere between this bar and the deposition will be the final or fully developed future left bank line of the river.

The photographic record of figure 5 is substantiated by surveys determining the riverbed elevation in the deposition area along transverse ranges (fig. 6). The first surveys in 1957 were not along these ranges, but by 1961 the ranges were established as a basis for future measurements. The ranges T–0 to T–26 became necessary when it appeared certain that the left or north bank was changing at least as far as Swatara Creek, a distance of about four-fifths

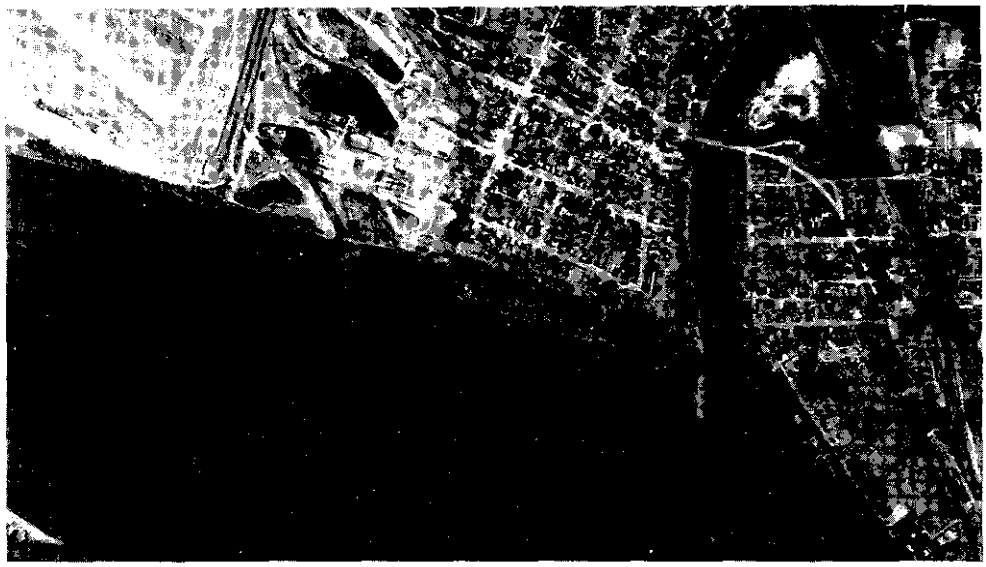


FIGURE 3. — Susquehanna River, August 13, 1950; discharge, 6,780 c.f.s.

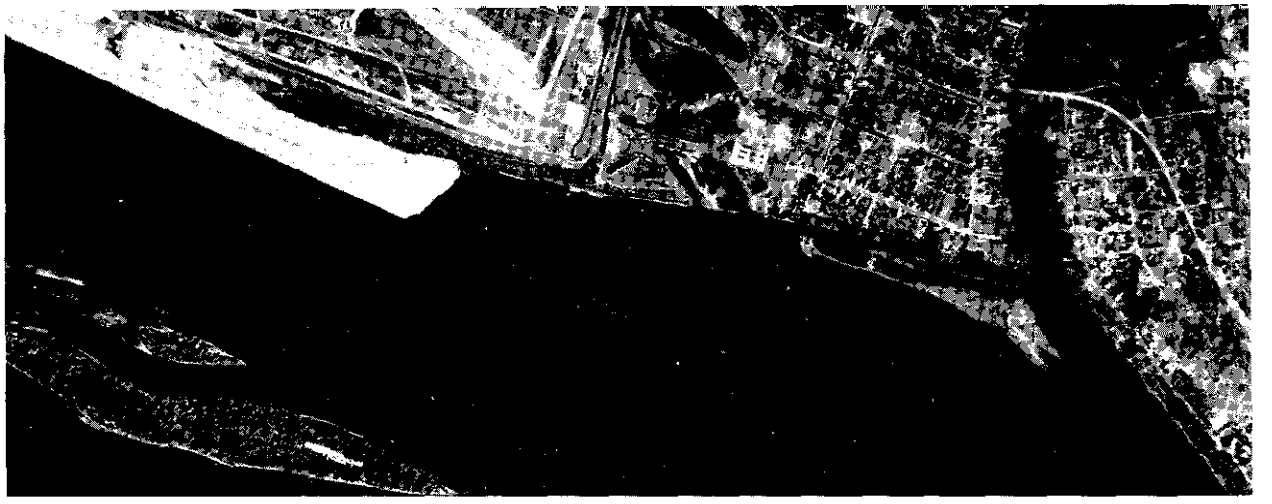


FIGURE 4. — Susquehanna River, September 5, 1957; discharge, 3,160 c.f.s.

of a mile. The T-ranges were used for the first time in August 1962.

The changes in bed elevation are exemplified by transverse profiles G and I (fig. 7). The earliest bed elevations of record are for April 1923. Systematic measurements of the deposition started in 1957. It is possible to reason from profiles G and I that a trough exists between 600 and 800 feet from the original shore and that this trough might not fill in. As there are only about four annual measurements as evidence of this trough, its permanence is not yet assured.

As the most data are available for ranges A to I, the area enclosed by these served as a basis for computation of the magnitude of the sediment accumulation. The mean total height of the deposit over this area is 0.15 foot for the period 1923-57, or an average annual deposit of only 0.004 foot. For the period 1957-62 the average accumulation is a total of 2.52 feet, or a mean annual deposit of 0.51 foot. The annual rate of deposit after the appearance of the constricting levee is about 130 times the mean rate for the 34-year period preceding the constriction.

The rapid occupation of the riverbed in front of the power plant by the accumulating sediment is represented graphically in figure 8, which shows the riverward advance of the shore line. Between ranges A and D the 1962 shore line is 550 feet riverward from the original shore line, and on range K it is 500 feet from the original shore line. This is a yearly advance of approximately 100 feet.

The measurements just described are not only a record of the effects of the constriction, but might serve two other purposes. First, although a prediction of the probable future course of the Susquehanna River will be ventured later, obviously only a model study could yield a truly reliable ultimate picture and for this purpose the measurements are indispensable. Second, it should be possible to relate the amount of sediment deposited to that transported in suspension by the Susquehanna River. It is reasonable to postulate that the sediment is deposited during the receding spring or snowmelt floods.

Probable Future of the River at Middletown

Before venturing to predict the future configuration of the river, particularly of its left bank, one should compare figure 2 portraying the preconstriction river and figure 9 depicting the river in June 1962. The shape of the deposition between the levee and the original island resembles closely the kind occurring downstream from river contraction works, such as spur dikes erected to induce deposition to protect or build up a riverbank. The shrubs on the bar at Swatara Creek are making it into a permanent island as is clearly evident on figure 10. That this bar could be suspected of acting as a spur dike is obvious in figure 11, where there are two smaller bars downstream from it. The Middletown Rapids stretch across the bottom of the photograph from the left bank to Hill Island (refer to fig. 1).

The foregoing conditions are made clearer by figures 12 and 13. The runaway width (200 feet) affords a sense of scale, but again it must be noted that these photographs are not accurate aerial survey photographs. The condenser

discharge water from the power plant flows between the original shore and the island, toward the bar at Swatara Creek.

With the aid of figure 14, which area extends eastward from the constriction to Swatara Creek and then southward to the Middletown Rapids, one may venture a prognosis of the long-range effect of the constriction on the pattern the river will follow. The shoreline in front of the plant will advance riverward at a decreasing rate, but probably not as far as the bar (fig. 5) extending downstream from the levee toe. The trough between the advancing shoreline and this bar could remain open. The riverward edge of the deposition in front of the plant might be a line from the toe of the embankment to the downstream tip of The Battery (original island).

Just as the bar at the toe of the levee showed shrub growths in the summer low-water of 1962, it is likely that the two long bars to the south in figure 14 will eventually trap some sediment when a lengthy low-water period happens and shrubs will develop on them. It is not too farfetched to conceive a chain of almost connected islands from The Battery to the bar formation at Swatara Creek to the two islands at the south. The discharge water will join Swatara Creek and flow southeast between the left or east bank and the suggested chain of islands.

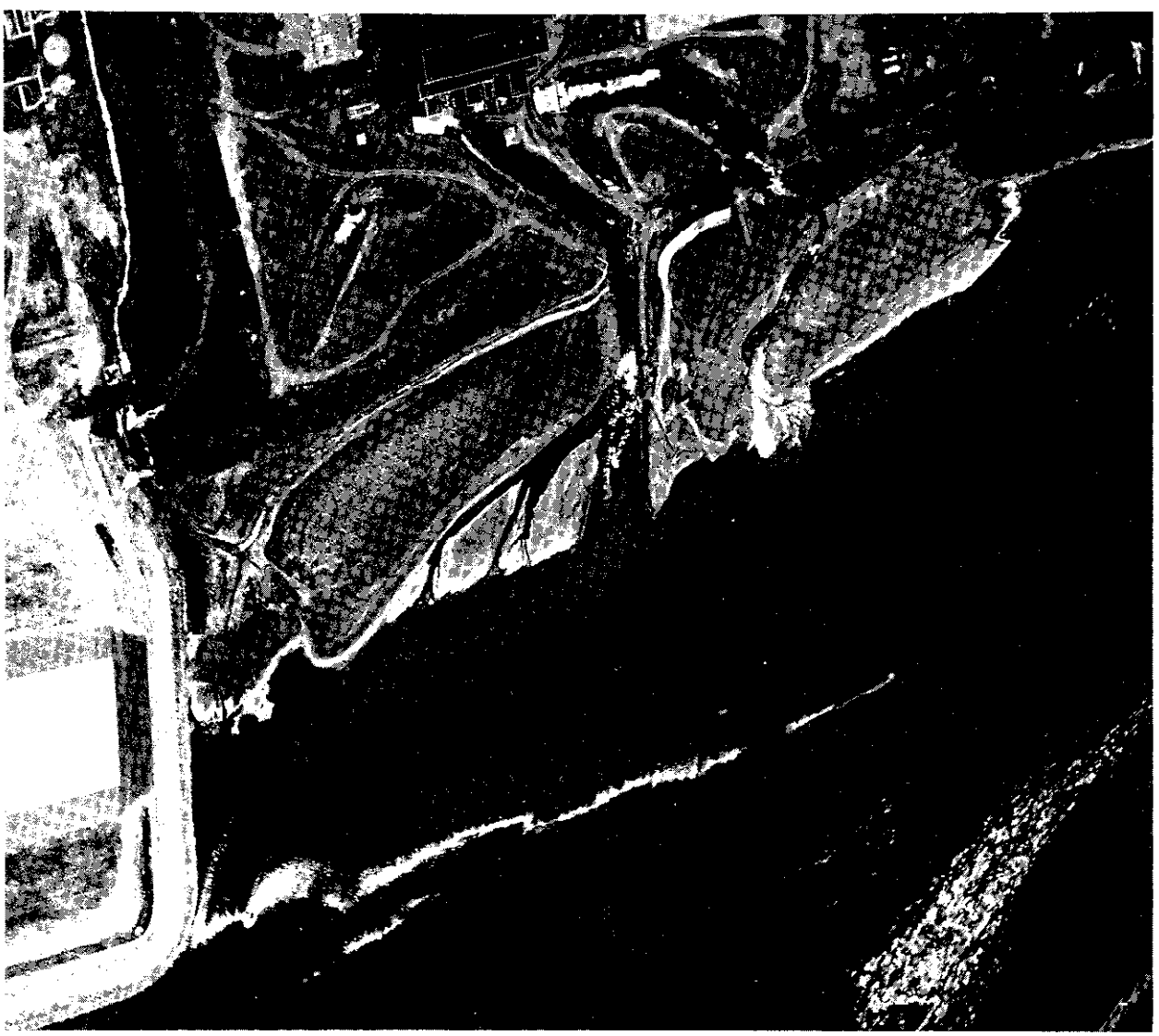


FIGURE 5. — Deposition at Middletown, August 3, 1959; discharge, 3,950 c.f.s.

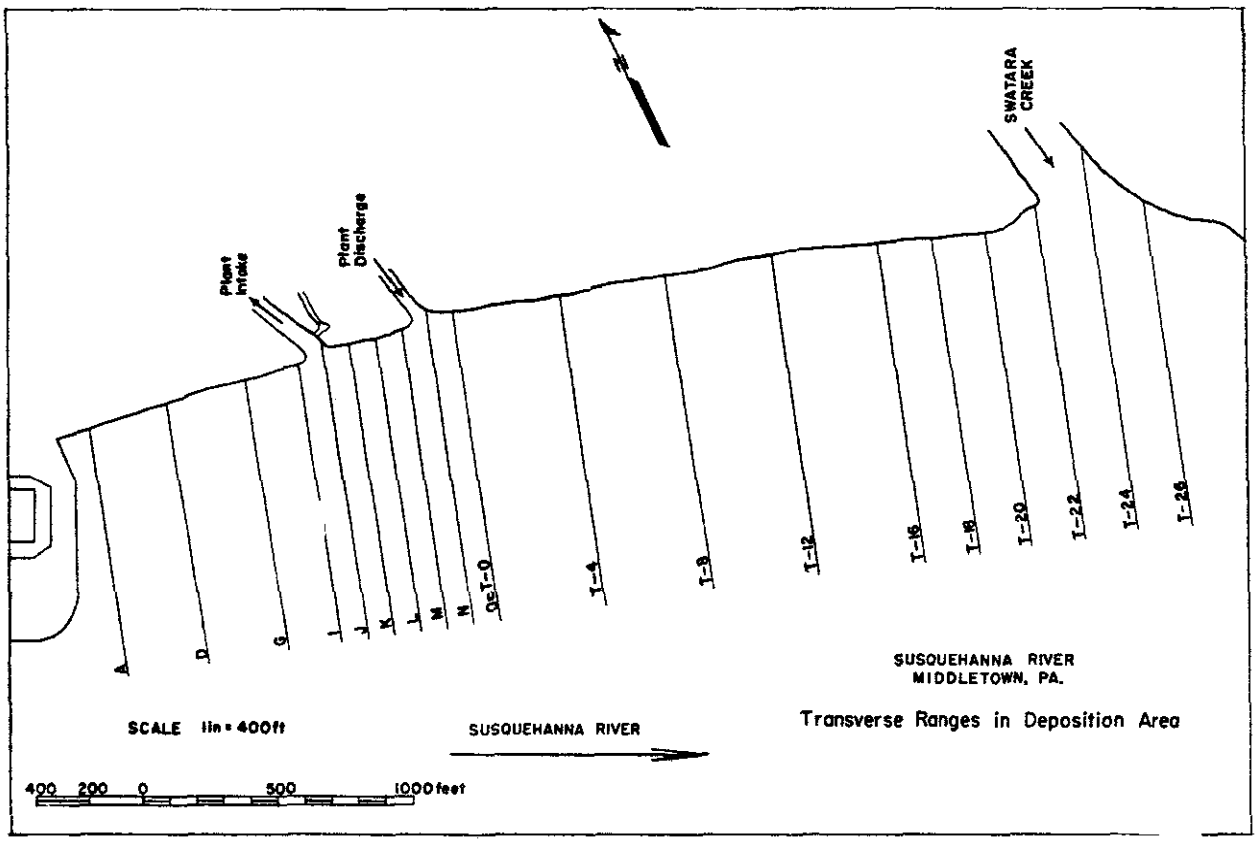


FIGURE 6. — Susquehanna River, Middletown; transverse ranges in deposition area.

The most conjectural part of this rationale is what the discharge water will do to the Swatara silt load. It is imaginable that the bar at Swatara Creek will grow northward to join the original shore, so that the discharge water would flow into the Susquehanna River at the downstream (east) tip of The Battery. Then Swatara Creek would have its own channel between the chain of islands and the cast bank.

The prognosis of island formation extends only to the Middletown Rapids where the flow is accelerated and below which no investigations have been undertaken. Nevertheless, that the effect of the deposition at Middletown is reaching surprisingly far downstream is substantiated by the local observation that the main river current during the last 2 or 3 years seems to be between Hill and Shelley Islands (see fig. 1), whereas it was formerly between Shelley and Three-Mile Islands; that furthermore and consonantly the northwest side of Shelley Island is eroding. The reach between Hill and Shelley Islands is about 2½ miles around the bend from the downstream end of the constricting levee. As would be expected, the constriction itself is causing erosion of the bank of Sassafras Island facing it.

The separation phenomenon associated with the outer or concave side of a bend is obviously also an important factor in the deposition process. The river bend at Middletown has a central angle of about 130° (see fig. 1). The entire reach of the left or concave bank from the plant to the rapids upstream from Fall Island is in the separation zone below the runway where an intricate and connected system of slowly rotating eddies or rollers will provide a region for sediment deposition. That the constriction has enlarged the width of the separation zone is clear.

River Data

As an appendix some river data will be given. The slope of rock bottom is about 0.05 percent, or 0.05 feet in 100 feet. At a Susquehanna River discharge of 5,020 c.f.s., the water surface slope on September 26, 1961, was approximately the same, though some slopes of 0.03 feet in 100 feet were observed. The water surface slopes were measured out in the river just beyond the bar at the toe of the levee.

For the period 1957–62, the Susquehanna low water discharge in the summer varies from about 2,500 to 9,000 c.f.s. The peak of the spring snowmelt floods varies from about 170,000 to 376,000.

The 42-year average discharge of Swatara Creek is 572 c.f.s. For the water year 1960–61, the mean is 625 c.f.s. with a maximum of 9,340 and a minimum of 84.

The plant usage of water ranges from a minimum of 38.4 to a maximum of 366 c.f.s. The maximum usage is an inappreciable 15 percent of the lowest Susquehanna River summer discharge of 2,500 c.f.s. A condenser discharge

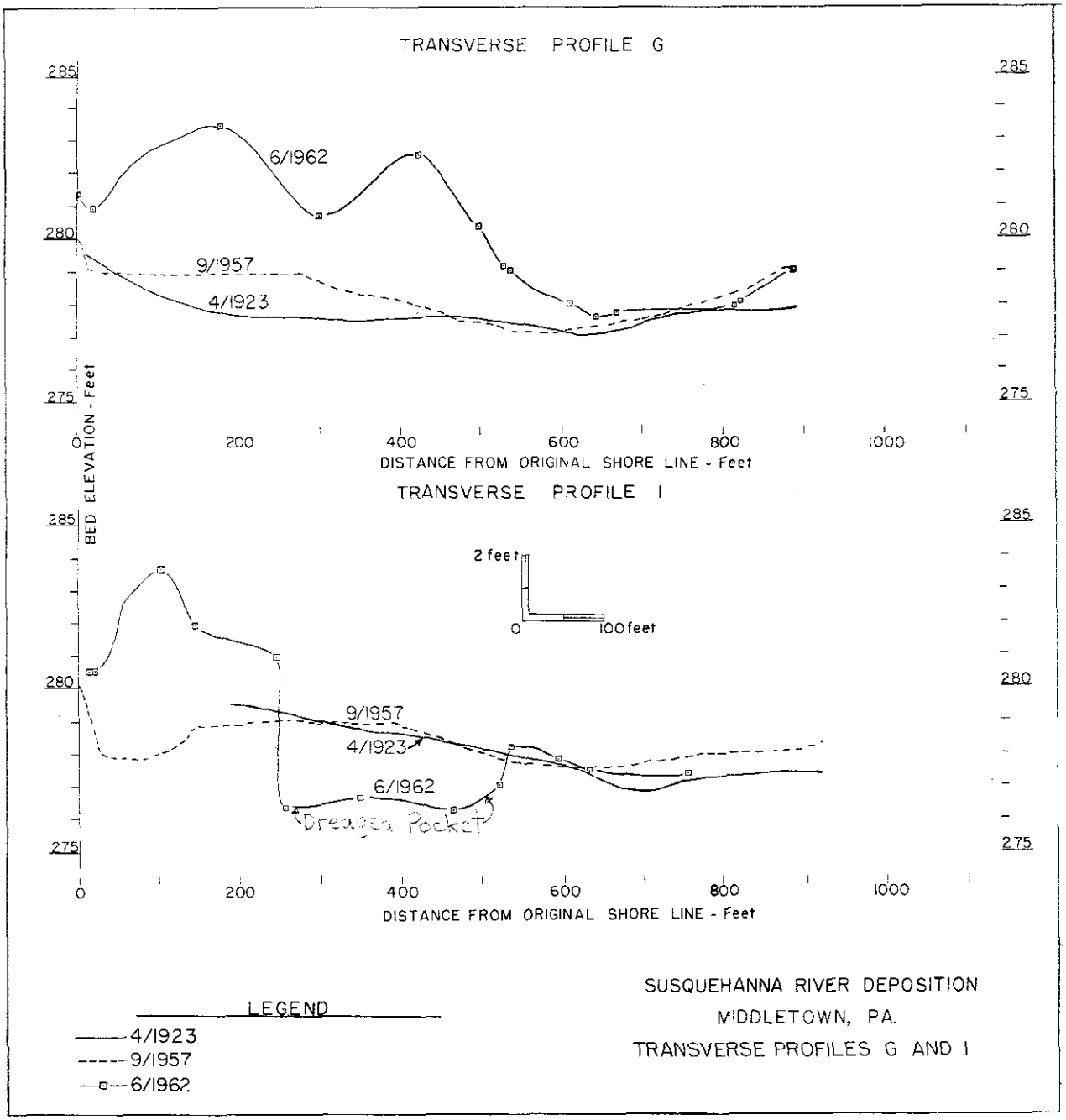


FIGURE 7. — Susquehanna River deposition at Middletown; transverse profiles G and I.

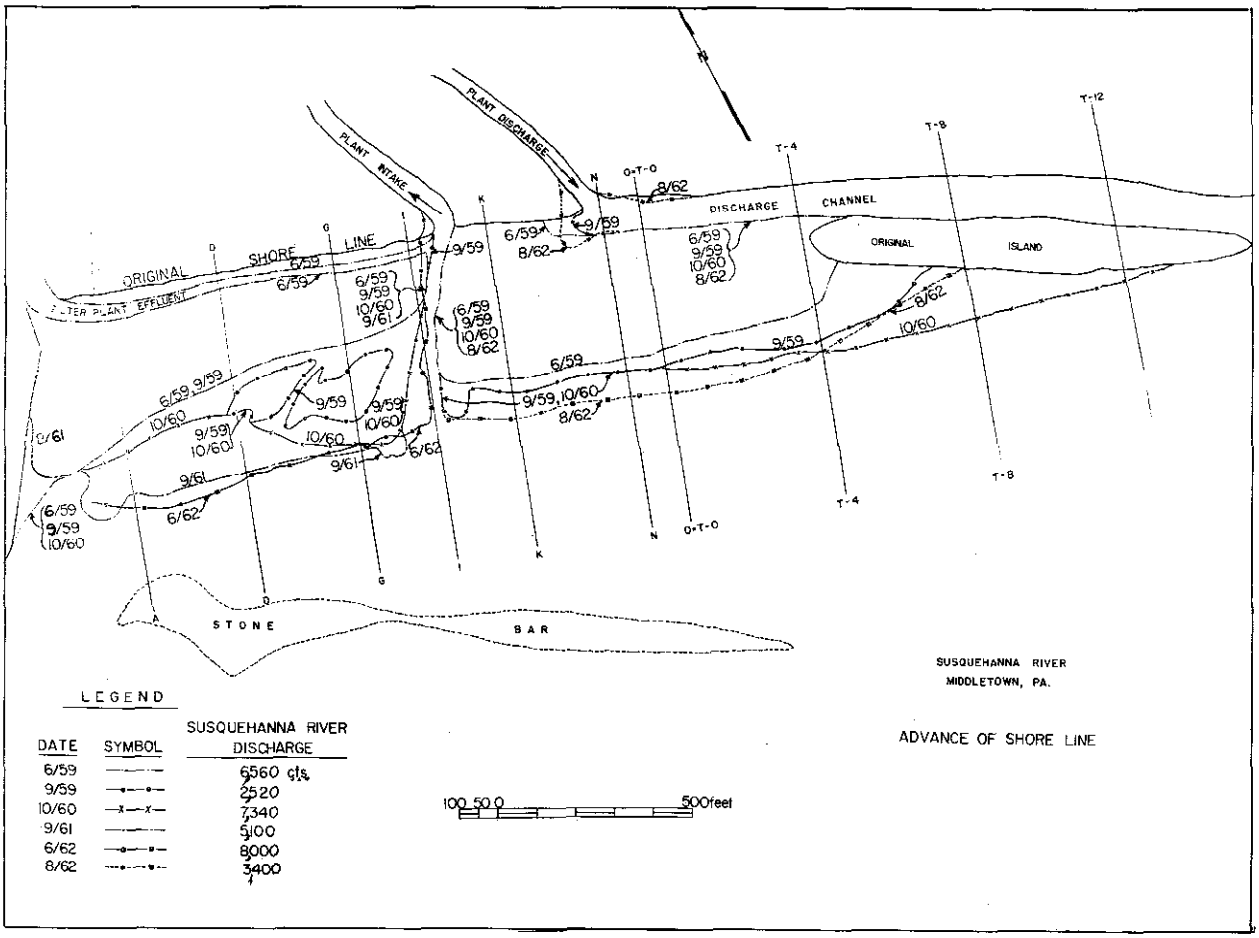


FIGURE 8. — Susquehanna River, Middletown; advance of shoreline.



FIGURE 9. — Susquehanna River, June 26, 1962; discharge, 5,860 c.f.s.

water flow of 366 could have considerable influence on the Swatara Creek.

The Susquehanna River suspended load measured at Harrisburg, Pa., ranges from approximately 7,000 tons per day at a discharge of 45,000 c.f.s. to 650,000 tons per day for 400,000 c.f.s. Swatara Creek at Middletown carries an approximate average of 660 tons per day.

Acknowledgments

J. J. Molloy, district engineer, Surface Water Branch, U.S. Geological Survey, Harrisburg, Pa., and his successor, R. E. Steacy, were very helpful in placing at my disposal discharge data for the Susquehanna River at Harrisburg.

J. R. George, geologist-in-charge, Quality of Water Branch, U.S. Geological Survey, Harrisburg, Pa., gave me important information on the suspended loads of the Susquehanna River and Swatara Creek and the geology of the area.

H. E. Bodden, superintendent of the Crawford Station of Metropolitan Edison Co. at Middletown, and D. G. Steck rendered me very substantial assistance at the deposition area by measurements, advice, and a steady flow of information; they were actually my watch on the river, and I would have accomplished much less without their continuous and enthusiastic cooperation.

Students of the Civil Engineering Department of The Pennsylvania State University assisted in the investigations; in particular, P. Pisaneschi, J. Smyth, and C. W. Lubold, Jr.

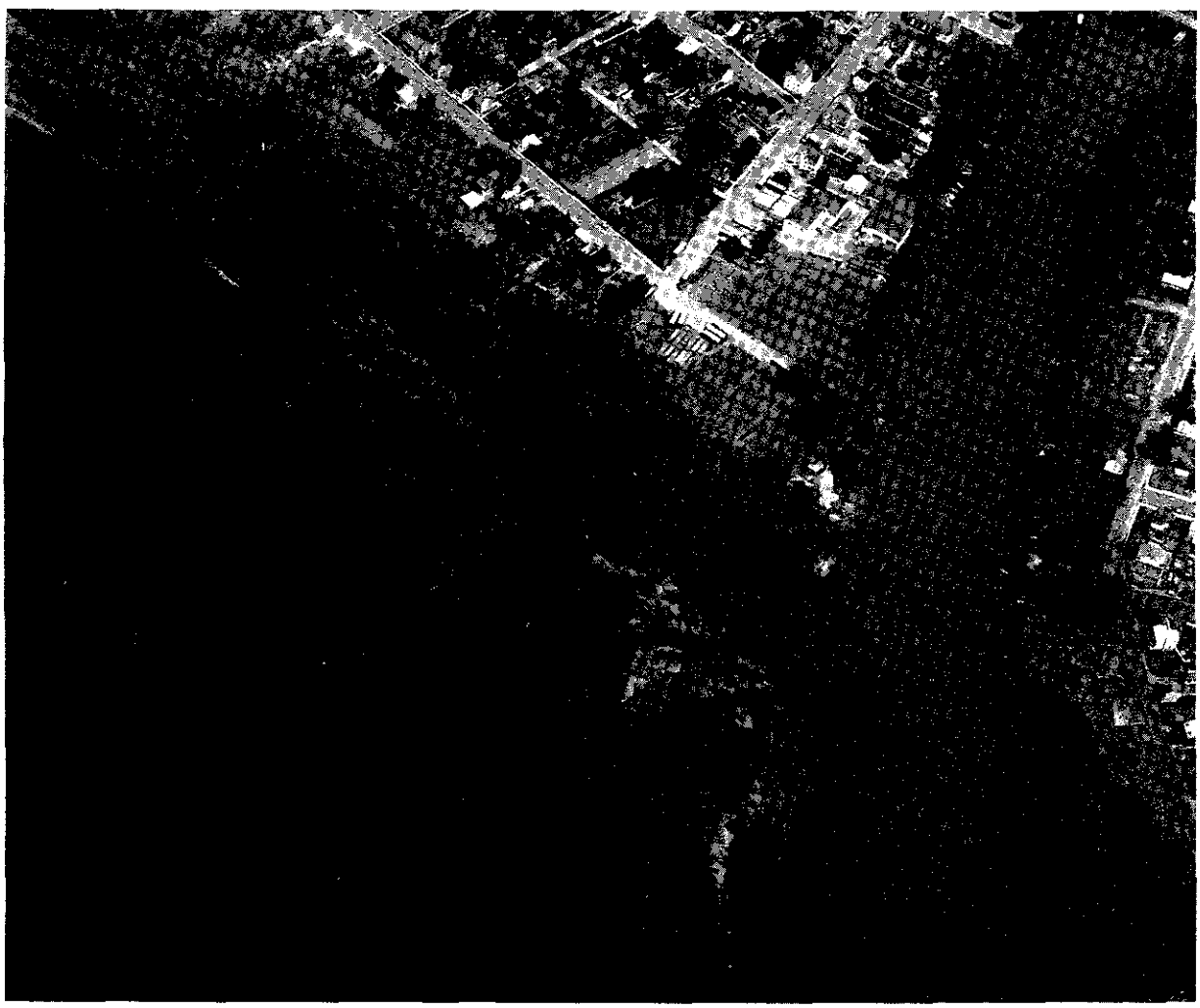


FIGURE 10. — Closeup of bar at Swatara Creek on Susquehanna River, June 26, 1962; discharge, 5,860 c.f.s.

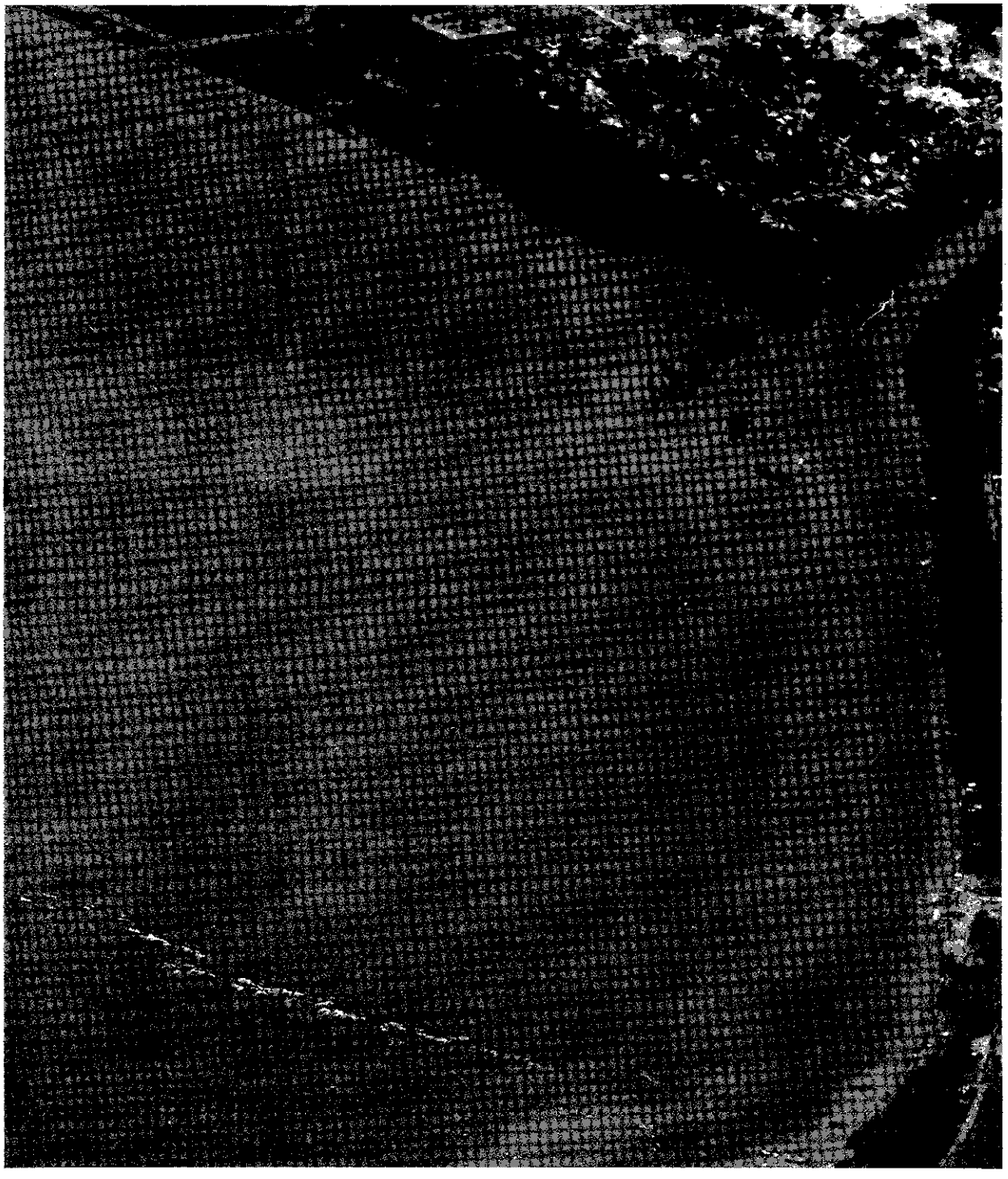


FIGURE 11.— Susquehanna River, September 15, 1961; discharge, 5,640 c.f.s.



FIGURE 12. — Deposition at Middletown, September 23, 1959; discharge, 2,670 c.f.s.

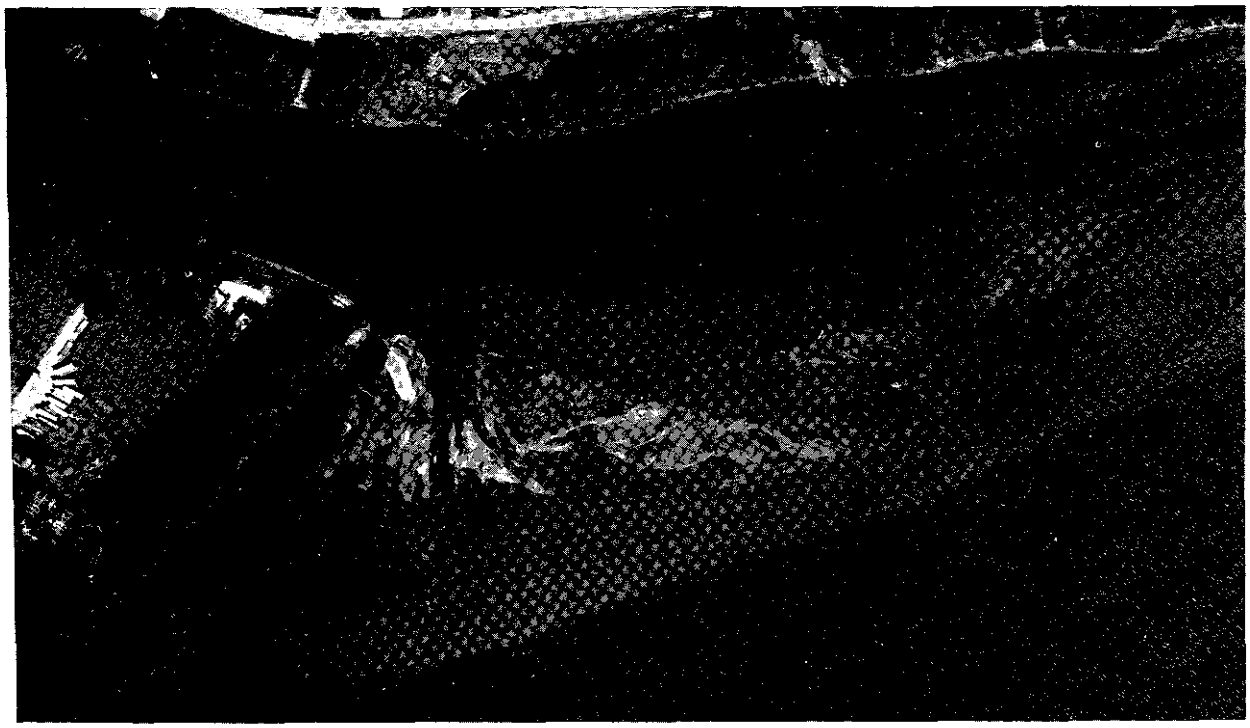


FIGURE 13. — Bars at Swatara Creek, September 15, 1961; discharge, 5,640 c.f.s.

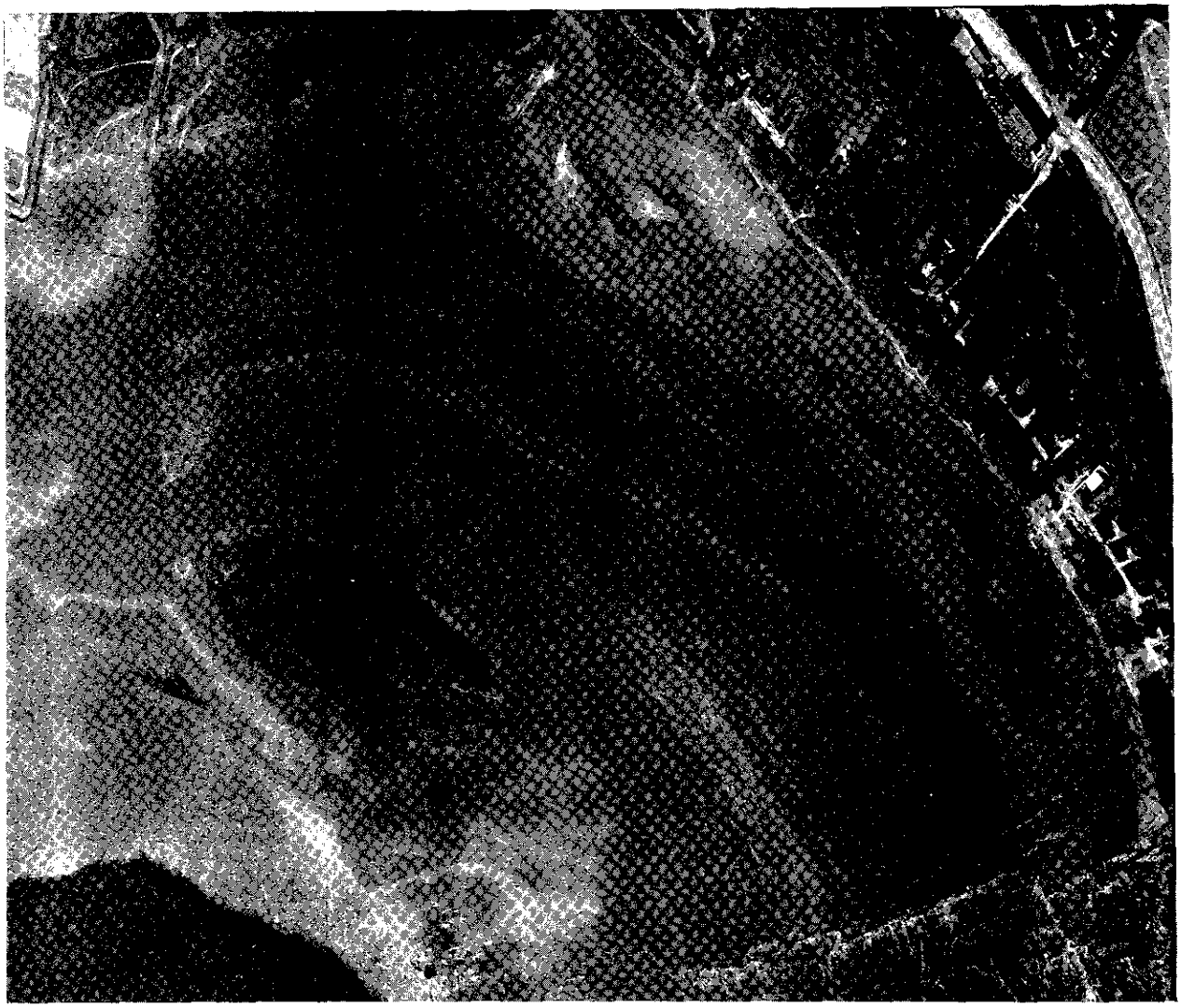


FIGURE 14. — Left bank of River susceptible to change, September 23, 1959; discharge, 2,670 c.f.s.

SOME ECONOMIC CONSIDERATIONS IN RIVER CONTROL WORK

[Paper No. 51]

By PAUL A. OLIVER, *chief, Division of Construction and River Control, Region 3, Bureau of Reclamation*

The name "Colorado River" and the term "muddy water" have long been linked in the minds of many Americans. When translated from the Spanish and applied to the description of rivers, the name "Colorado" means "reddish in color." Many will appreciate the humor of the expression "too thick to drink and too thin to plow" as often applied to the Colorado River. Prior to the closure of Hoover Dam these and many other terms justly described the river from its source to its mouth. However, since the closure of Hoover Dam in 1935 and the subsequent completion of Parker and Imperial Dams in 1938 and Davis Dam in 1953, these terms can no longer be justly applied to the river below Hoover Dam (fig. 1).

The changed regimen of the river resulting from the construction of this stairway of dams has been dramatic. For many years prior to the construction of Hoover Dam the river carried great quantities of sediment brought in from the drainage areas during the periods of floodflow. Some of this material was deposited in the river channel; some was carried to the delta located at the upper end of the Gulf of California. During the seasons after floodflows, part of the bed material was picked up and moved to deposits farther downstream. The net result was a gradual rise in the riverbed elevation in the lower reaches. Subsequent to the construction of Hoover Dam, the sediment load from the upper river has been trapped in Lake Mead and clear water released below the dam. This water, freed of the sediment load, rapidly scoured the deposit that was laid down during the earlier years. After the construction of the dams downstream, additional depositories for the scoured materials were formed and correspondingly more reaches of the river became subject to the erosive action of the clear water. The dramatic effect of the changed regimen upon the downstream sediment load can be expressed well in tabular form. Table 1, prepared from data given in Geological Survey papers, shows flows and sediment transport for two stations on the river. The Grand Canyon station, 267 miles above Hoover Dam, represents the near virgin condition, while the Yuma station, some 300 miles below Hoover Dam and located near the head of the Colorado River Delta, reflects the altered regimen.

To describe all economic aspects of river control for the entire reach of river between Hoover and the Delta would require more space than allowed here; however, some aspects are worthy of consideration. In this report, remarks will be confined to aspects of river control in the Needles area, the Blythe area, and in the Yuma area.

TABLE 1.—*Suspended sediment load of the Colorado River at Grand Canyon and at Yuma*

Water year	Grand Canyon		Yuma	
	Discharge, Acre-feet ×1,000	Load, Tons×1,000	Discharge, Acre-feet ×1,000	Load, Tons×1,000
1926	14,400	225,000	12,200	130,800
1927	17,300	398,000	17,100	242,500
1928	15,600	172,000	12,800	114,800
1929	19,400	480,000	17,500	289,000
1930	13,400	235,000	10,600	183,900
1931	6,700	68,800	4,800	52,100
1932	16,000	261,000	14,200	184,200
1933	10,000	178,000	8,000	92,200
1934	4,650	50,100	2,400	17,800
1935[1]	10,210	122,000	4,000	21,600
1936	12,300	158,000	3,500	14,000
1937	12,400	191,000	4,000	15,800
1938[2]	15,600	232,000	4,200	12,500
1939	9,620	86,300	6,600	13,000
1940	7,440	75,400	5,400	3,500
1941	16,900	270,000	11,700	14,100
1942	17,300	230,000	10,500	8,800
1943	11,400	58,800	7,300	3,000
1944	13,500	97,800	8,600	4,800
1945	11,900	83,600	6,500	4,400
1946	9,100	66,000	3,800	1,700
1947	13,700	136,000	4,200	2,000
1948[3]	13,900	144,000	6,300	6,700
1949	14,400	119,000	6,600	7,800
1950	11,100	59,800	3,500	3,500
1951	9,840	48,700	2,800	1,100
1952	18,200	148,000	9,200	10,500
1953	8,900	48,800	4,100	1,900
1954	6,200	40,700	3,200	1,300
1955	7,580	83,100	2,100	830
1956	8,700	76,100	880	260
1957	17,500	155,000	1,170	660
1958	14,500	132,000	2,950	3,900
1959	6,900	23,600	940	560
1960	9,580	37,500	700	137
1961	7,080	46,138	680	135

[1] Closure of Hoover Dam.
[2] Closure of Parker and Imperial Dams.
[3] Initial closure of Davis Dam.

River Problems, Needles Area

The problems in the Needles, Calif., area arising out of the changed regimen of the river are worthy of note, not only because they were the first to occur but because they were the most dramatic and costly to date. The Bureau of Reclamation studies indicate that from the time of closure of Parker Dam in 1938 until October 1944 a total of 84 million cubic yards of sediment eroded below Hoover Dam. Surveys showed that 67 percent of the total originated in the reach between Hoover and the site of Davis Dam and that the yearly contribution of

this reach averaged 500,000 cubic yards by 1944. Although a large part of the material eroded was deposited in Havasu Lake, some 48 percent of the total eroded up to December 1944 was deposited in the flood plain between Needles and Topock.

By 1941, however, the backwater arising from the filling of Havasu Lake had formed a pool over the flat bottom lands upstream of Topock. With the reduced velocities, the rate of deposition increased and, by the progressive creation and exposure of sandbars and growth of phreatophytes, the deposition was accelerated until by 1944 an almost impenetrable swampy jungle existed in the 12-mile reach between Needles and Topock.

As the backed-up waters reached the vicinity of Needles, lands and homes in the riverfront section of the town within the levee were inundated first by seepage and finally by backwater encroachment. Early in 1944 it became evident that the levee in front of Needles would soon be overtopped. Had this happened, inundations would have reached a stage endangering the municipal water supply, the California Pacific Utility Light and Gas Plant, and the Santa Fe Railroad Division Point facilities including the terminal, shops, roundhouse, powerplant, iceplant, and some 12 miles of double track main line.

To cope with this emergency, the Congress appropriated funds for raising the levee and for the purchase of land and homes that had been inundated in the city of Needles. Under the provisions of the Interior Department Appropriations Act of 1945, a total of $340,000 was made available, of which not to exceed $100,000 was to be expended for the purchase of land subject to seepage or overflow and for improvements thereon.

The history of changes in water surface elevation at Needles and at Topock, some 12 miles below Needles, is shown on the hydrograph, figure 2. The hydrograph shows that the rise that had been taking place at Topock prior to the construction of Hoover Dam continued at a gradual rate until Havasu Lake was formed. Within a short time thereafter the elevation of the water surface rose an additional 6 to 10 feet to about elevation 448.0 feet. On the other hand, the rise at Needles was halted almost completely after the closure of Hoover Dam and the level remained constant until 1941, when a rapid rate of rise began. This rapid rate continued at an average of 1.2 feet per year until the end of 1944 and elevation of 475.5 feet was reached. The level then proceeded to drop slowly until June 1951, when it was about 2.7 feet lower than at the end of 1944.

In 1942, an effort was made to scour the bars upstream from Topock by lowering the level of Havasu Lake. It was found, however, that although obstructions in the upper reservoir were removed during the lowering of the lake, little or no effect was noted on the gages above Topock and in the Needles area. The density of the growth in the swamp area had already reached such proportions that it was considered uneconomical to hold Havasu Lake at a low elevation long enough to reestablish a natural channel in the river. This decision was also reinforced by the fact that the gages at Needles responded directly to any change in release out of Hoover, and consequently it could be expected that additional sediment would act to reduce the effect of lowering Havasu Lake.

In an engineering study, completed in 1944, it was determined that the most practicable solution of the problem at Needles would be to dredge a channel and to open the way for the river to flow from Needles to Topock at a slope designed to provide for transport of all sediment. The completion of the dredged channel in 1951 between Needles and Topock and the subsequent adjustment of the river to this new route has resulted in the lowering of the water surface at Needles about 5 feet. In the years 1951–60, the dredge worked upstream and completed the channelization from Needles to a point some 8 miles downstream from Davis Dam. Concurrently with the dredging of the channel a program of bank line revetment and stabilization was accomplished. As a result of this dual program the erosion of material along the new channel is largely confined to that in the bed of the river. The dredging operation was conducted so as to take advantage of the larger deposits of gravel in the cut to establish sills for grade control, and the bottom is gradually becoming armored in other reaches with heavier material as the finer sediments are removed by the river.

Studies made in support of an extension of channelization upstream from Needles indicated that a total of 112,261,000 cubic yards of sediment would be eroded from this reach of river if left in its natural state. Continued deposition of this material in the upper 12 miles of Havasu Lake would have caused a restoration of high-water conditions at Needles. As a result of channelization, the total suspended load, in tons per day at 15,000 c.f.s., has decreased at the Needles station on an average from 14,000 tons per day in 1956 to 5,000 in 1962.

In order to reduce the possibility of a later buildup of sediment downstream at Topock and to reduce the amount of material carried into Havasu Lake, it was decided to create a settling basin immediately upstream from Topock in the newly dredged channel and to station a small

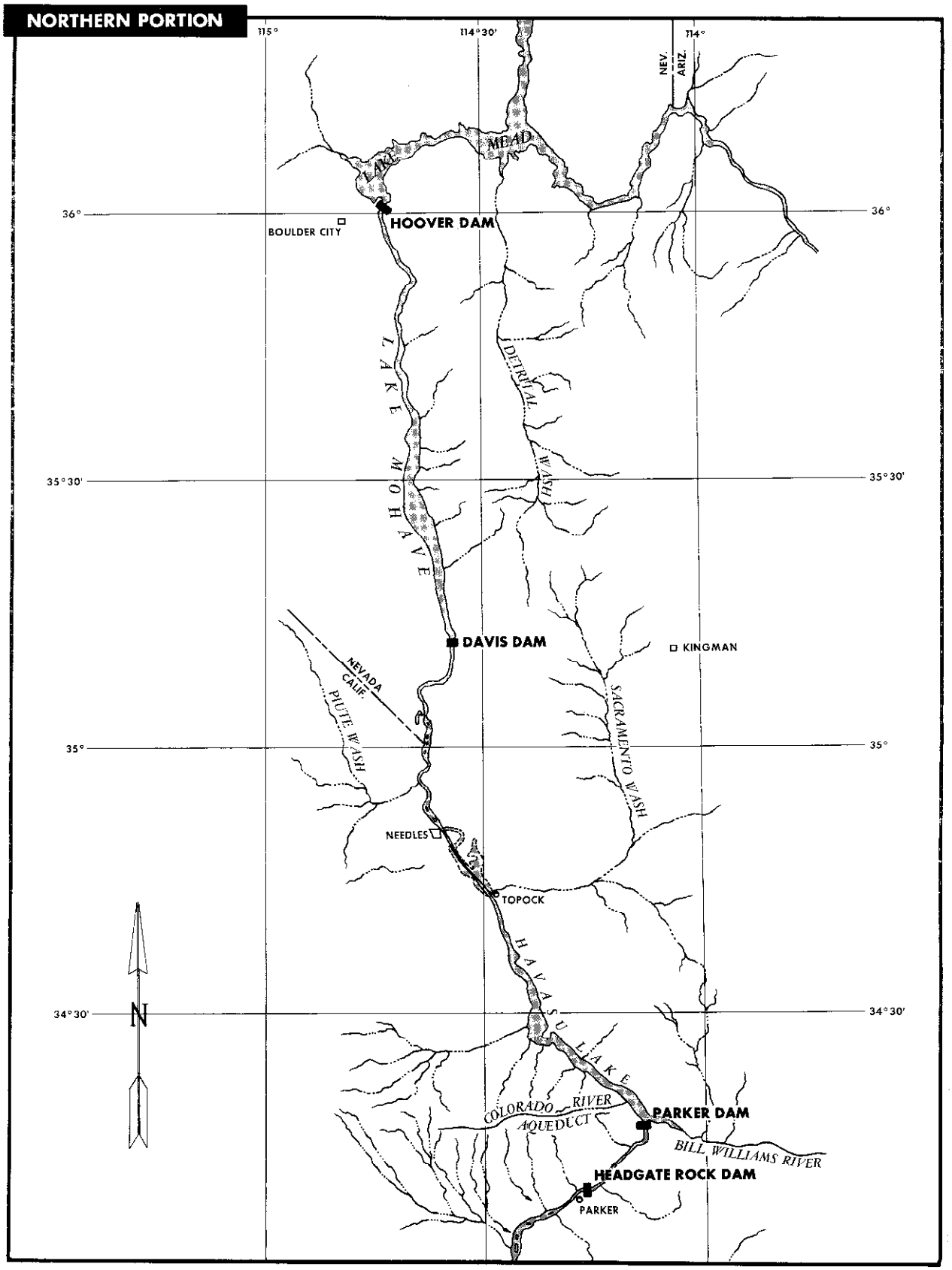
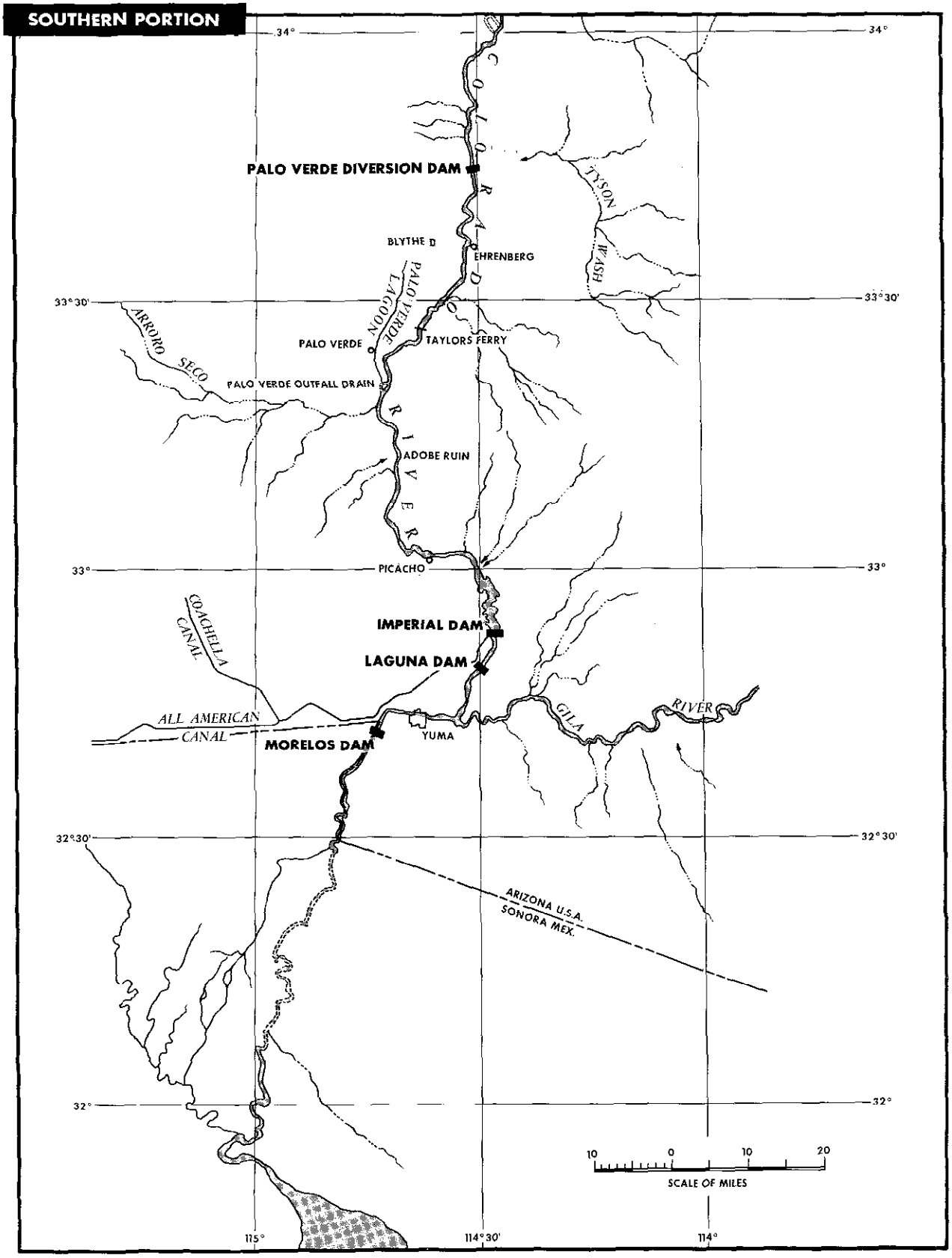


FIGURE 1.—Location map



of Lower Colorado River.

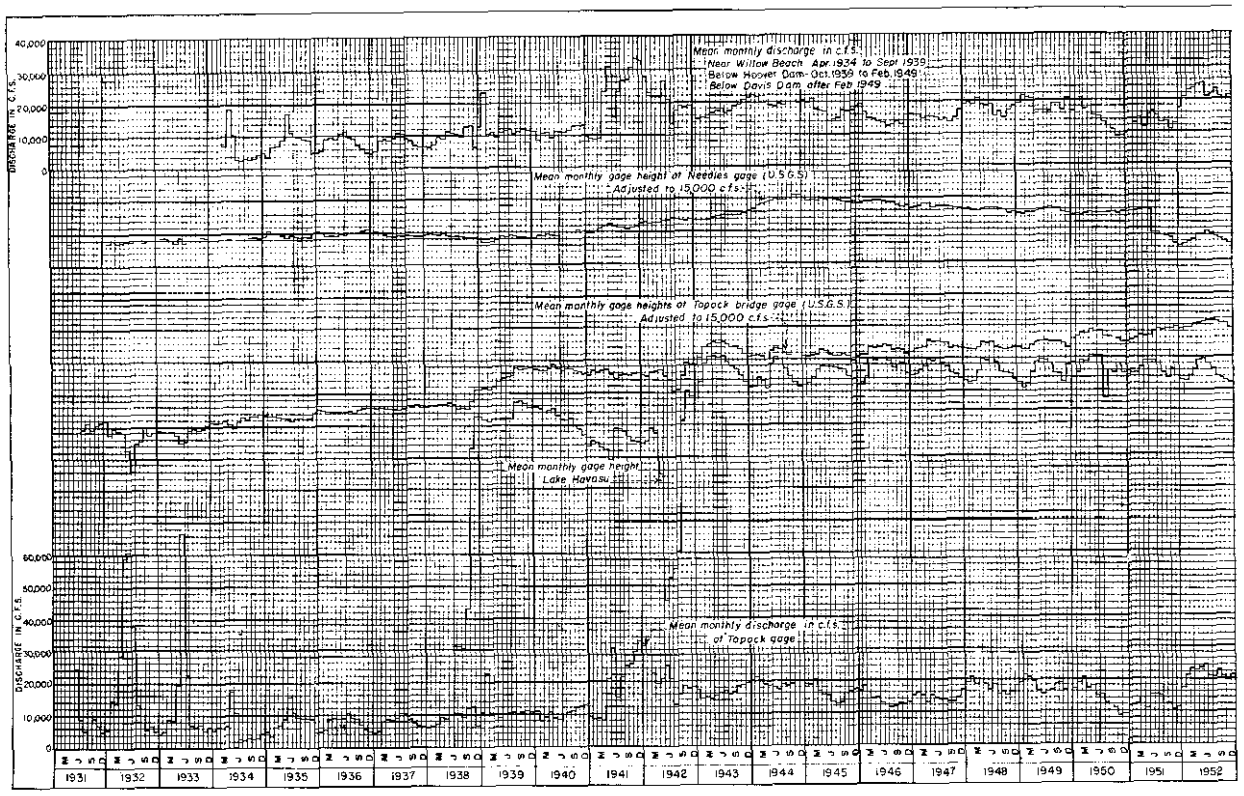


FIGURE 2. — Discharge and water surface

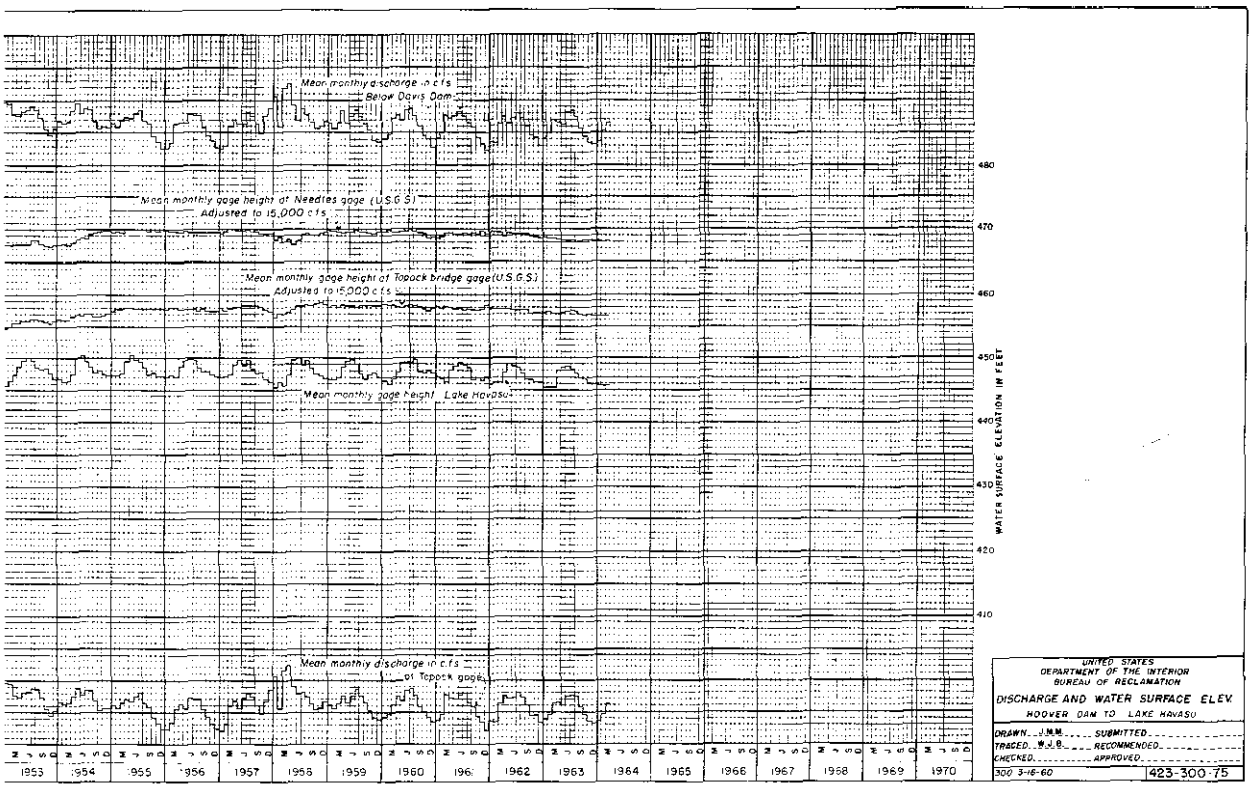
dredge at this point for removal of sediments arriving at that point. A 12-inch dredge was acquired in 1957, and since that time it has been working back and forth in this settling basin, removing the sediments arriving therein. Material removed is being deposited in the swamp area where it acts to reduce to some extent the open water area and tule growth. By 1965 reduction in sediment transport should allow the moving of this dredge to immediately below Topock, where a new program directed to channelization can be initiated.

It is expected that completion of the work in this region will produce a channel slope from Davis Dam to Havasu Lake that will be adequate to carry the relatively clear water flow anticipated at that time. Work to date in the stream between Davis Dam and Topock has resulted in the estimated annual salvage of some 65,000 acre-feet of water, and cost of the rechannelization has totaled $7,500,000. The estimated cost of channelization through Topock Gorge is $5,000,000, and the additional salvage in water should total another 60,000 acre-feet per year.

The studies mentioned previously predicted a cost–benefit ratio for work for the area between Davis Dam and Topock as summarized below. Recognition was given in these supporting studies of an inseparable relationship between work in this area and work to be done in the Topock Gorge.

Period:	*Benefit-cost ratio*
100-year	2.18 to 1
50-year	1.78 to 1

A revised program has since been developed for the Topock Gorge (below Topock), and a new overall cost–benefit ratio remains to be determined. In the original benefit study, no consideration was given to flood control and sediment reduction, as it was found that the benefits from other factors greatly exceeded the expected project costs, even though the relatively conservative values for irrigation use were applied to the salvaged water. It is expected that use of values attributable to municipal and industrial use will be considered in the later studies, and, if needed, the benefits from sediment reduction and flood control will be evaluated. Although flood benefits were not considered, an idea as to the threatened cost of damage to the Santa Fe Railroad should be mentioned. If the uncontrolled rise in river levels opposite Needles had been left to continue and thereby to force the relocation of the Santa



elevation at Needles and Topock gate.

Fe Railroad facilities, the cost of the relocation, estimated in 1947 by the chief engineer for the company, would have been somewhere between 10 million and 12 million dollars.

Not only has much of the agricultural area that was inundated by the high water been redeveloped, but there have been ancillary benefits arising from expansion of the recreational use of the riverfront lands. With the advent of clear 50° to 60° temperature water below Davis Dam, the fish and wildlife interests have introduced trout into the stream and a new type of settlement pressure has been the result. Where previously the lands were utilized only for farming, several full sections have been subdivided into smaller acreages for homesite and trailer park development.

The increased recreation pressure has resulted in the development of several bypassed inlets and lagoons along the new channel into small boat and trailer park marinas through State, county, city, and private enterprise. The mild winter climate that prevails has also had its influence. All these factors have acted to increase the rate of development and accordingly the value of lands along the river. Records in the assessor's office attest to the increase in prices paid for river frontage. Prices rose from approximately $10 to $20 per front foot in the mid-1950's to more than $100 per front foot by 1960.

River Problems, Blythe Area

The town of Blythe, Calif., and its principal source of economic support, the Palo Verde Irrigation District, lie about midway in the 150-mile reach between Parker and Imperial Dams. After the closure of Parker Dam in 1938, difficulty opposite in nature to that at Needles developed at the Palo Verde Irrigation District intake, 65 miles downstream from Parker Dam. Degradation of the riverbed lowered the water surface to such an extent that by 1944 gravity diversion into the District's canal was no longer possible. In 1944, the Congress approved funds for construction, operation, and maintenance of a temporary rock weir designed to restore operating levels at the intake. This weir, which was completed in 1945, initially required a total of 40,000 cubic yards of rock to raise the water surface to the required level for gravity diversions.

However, before the weir was replaced in 1957 with a permanent structure, the original total of rock placed had been increased by nearly 200,000 cubic yards as a result of need for re-

pairs and reinforcement. The permanent structure consisting of a rockfill dam, headworks, and a levee and drain system was constructed in accordance with the provisions of a repayment contract between the United States and the Palo Verde Irrigation District. Under this contract the United States agreed to construct the diversion dam, spillway, and intake works at a cost not to exceed $4,538,000 and to loan the District $500,000 for use in revising its distribution system to conform to the new diversion works. In addition to furnishing the necessary right-of-way and agreeing to operate and maintain the completed works, the District agreed to repay $1,175,000 in addition to the $500,000 as its share of the cost.

As a result of the combination of degradation and aggradation and the sill effect of the Palo Verde Diversion Dam, a degree of stability now exists in the riverbed for the 150-mile reach between Parker and Imperial Dams. The reduced slopes and the exposure of armoring beds of coarse gravel in the upper reaches have served to greatly reduce scour, while the decreased erosive force of sediment laden water has been the principal factor in reduction of scour in the downstream part.

Despite the quasi stability of the bed, an active erosion and sedimentation problem of considerable magnitude exists in this 150-mile reach. Degradation of the upper 90 to 100 miles has resulted in rather severe entrenchment, and, as the clear water is continually attacking the high banks, the sediment contribution is rather large. Water loss is also quite high in this 150 miles as a result of the excessive amount of braided, shallow, and excessively wide channel areas existing through the reach. Water losses are magnified in the downstream part where the river is contributing to high-water tables and excessive growth of phreatophytes.

Reconnaissance studies recently completed indicate that reduction in sediment transport and salvage of water in this reach can amply justify a channel improvement program, which it is estimated could cost $16,000,000. As a result of comprehensive studies, part of this program is already underway and further studies necessary to firm up the estimates for additional works are in progress. The complete program is expected to halve the 3 plus million cubic yards per year total of sediment now moving from this reach into the Imperial Dam reservoir and into the sediment control facilities of downstream irrigation districts. The confinement of the flow in a properly designed channel with stabilized banks and a gradient that will provide improved drainage of irrigated acreages should reduce the present average annual water loss by approximately 90,000 acre-feet.

Recreational and senior citizen groups are already taxing the marina and trailer park facilities in this area, and plans for the expansion of these facilities are being pressed by State, county, and private interests. It can be expected that upon the completion of the programed river control improvements, further expansions and attendant benefits to the local economy will result.

River Problems, Yuma Area

The Yuma area now marks the end of the line for the Colorado River. The last available water is diverted from Imperial Dam 17.5 miles upstream from Yuma for the irrigation of about 800,000 acres of land lying in the United States and from Morelos Dam 8.3 miles downriver from Yuma for the irrigation of about 400,000 acres in Mexico.

After closure of Imperial Dam in 1938, releases from Hoover Dam exceeded the downstream diversion requirements for several years and, as a result, the clearer water leaving Imperial Dam caused a degradation of the riverbed opposite the intake to the Alamo Canal, which supplied water to Mexico. By 1944 it became impossible to divert the required amounts, and service to Mexico was supplied by diversion from the All-American Canal to the Alamo Canal under special arrangements each year until 1950, when Morelos Dam was put into service by the Mexican Government in conformity with provisions of Article 12a of the February 1944 Treaty with the United States.

In the low-water years beginning 1953, a serious river regulation situation has developed. Referring again to the table on page 1, the average annual flow in the river at Yuma has dropped below a million acre-feet per year. This flow, plus returns to the river at points downriver, makes up to the guaranteed 1.5 million acre-feet that this country delivers to Mexico under the Treaty of 1944. Since 1953, in only 2 years have flows passed the Yuma gaging station in sufficient quantity to cause degradation of the channel. Stored water has been insufficient in the remaining years to provide regulatory flow sufficient to move downriver all sediment returned to the river below Imperial Dam and general aggradation of the riverbed resulted.

Since 1950 the Alamo Canal immediately below the inlet works at Morelos Dam has been maintained as a settling basin and cleaned periodically by dredging. By 1956, however, the spoil area had been built up so high that the 12-inch dredge maintaining the desilting basin

could no longer economically pump the silt to the designated area and the Mexicans have since deposited some 61,000 cubic yards of dredged material in a relatively short stretch of river below Morelos Dam.

Efforts to remove the channel constricting deposit hydraulically by means of large releases of stored water have met with only limited success and such efforts have been abandoned. Negotiations are now in progress to have the material removed by mechanical means and for the Mexicans to seek other spoil areas on land for future operations.

The limited releases from Imperial and the practically nonexistent flows below Morelos have resulted in an alarming decrease in carrying capacity of the floodway below Imperial Dam. An attempt this winter (1962–63) to improve the channel by release of some 270,000 acre-feet of stored water in regulatory flows has met with only nominal success, and a program of channel rehabilitation is planned by both the Bureau of Reclamation and the International Boundary and Water Commission.

This program will include the stationing of a 12-inch dredge, now under construction, in a swampy area below Imperial Dam where a desilting basin is planned. The silt released below Imperial will be trapped in the basin and deposited by the dredge in the adjacent swamp area. Clearing of the floodway will be initiated in the next fiscal year, and, in the following years, an overall program will be directed toward construction of a low-flow channel and a well designed floodway between Imperial and Morelos Dams. The International Boundary and Water Commission, United States Section, is negotiating with the Mexican Section to provide an extended international program of channel clearing and floodway maintenance in the 40-mile reach of river below Morelos Dam.

The daily scheduling of water to arrive at Imperial Dam for delivery to the 1,200,000 acres of land in the United States and Mexico is being subjected to considerable scrutiny in view of the recent history of low runoff, the impending closure of Glen Canyon Dam, and the need for filling Lake Powell. The fact that the final control for flows reaching Imperial is at Parker Dam 150 miles away along a badly deteriorated channel causes continuing concern to water-scheduling people. In 1961, the best year of operations, delivery of 7,000,000 acre-feet of water was made at Imperial Dam and the flow deviated only 5,000 acre-feet from the amount scheduled for arrival.

Cause for concern arises from the fact that even with this close limit on delivery, some 300,000 acre-feet of water over and above that ordered by Mexico under the Treaty was delivered at Morelos. As most of the yearly overdelivery generally consists of cutbacks in American orders and unforeseen changes occurring in the 3 days required for flow to reach Imperial Dam from Parker Dam, reregulation at Imperial is imperative. It is believed that this can be accomplished through construction of an offstream storage reservoir in a side canyon 1½ miles upstream from Imperial Dam. Feasibility studies show that a 15,000 acre-foot storage reservoir, having a 500 c.f.s. pumpback storage facility, will economically salvage a substantial part of the 300,000 acre-feet of overdelivery presently being wasted to the Gulf of California. The estimated cost of construction of this offstream reservoir and pumping plant is $7,000,000.

The economic expansion of the southwestern part of the United States is directly related to the availability of a dependable supply of water. Although the limit of supply of stored water in the Colorado River Basin is near, the fact is recognized that there is a vast possibility for economies in the use of this resource. This paper describes some potential sources of salvage; however, the Bureau of Reclamation estimate of the total potential salvage along the river and in dependent irrigated areas is in the realm of 1,000,000 acre-feet. This total could no doubt be augmented somewhat in practice if a sincere effort is directed, at all points from storage dams to the last irrigator on the lateral, toward economic use of water. The Bureau of Reclamation, as representative of the Secretary of the Interior in custodianship of the Colorado River, welcomes the responsibility of leading an all-out effort toward the salvage of water now being lost.

SEDIMENT DEPOSITS ON THE CONE AREA OF BIG SAND CREEK, AT VALLEY HILL, MISS.

[Paper No. 52]

By FRED BAYLEY, JR., *U.S. Army Engineer District, Vicksburg, Miss.*

Synopsis

Hill tributaries of the Yazoo River system in Mississippi carry a tremendous sediment load. This tends to be deposited near the foot of the hills in large cone-shaped fills as the streams enter the flat alluvial valley. Big Sand Creek in north-central Mississippi is typical of these. It affords a rare instance where estimates of sedi-

ment rates can be based on actual field measurements of deposits. Sediment deposits on the cone area have been computed from a comparison of range profiles obtained in 1920 with those made in 1947 and 1957. This paper describes the watershed, the range profiles, and the amount of sediment deposits. Between 1920 and 1947, these deposits amounted to nearly 1,000,000 cu. yd., or 583 acre-ft. per year, equivalent to over 5 acre-ft. per square mile per year.

General

The line of hills along the eastern side of the Mississippi Delta rises abruptly about 150 ft. in elevation in a very short distance. The streams, heavily laden with sediment, enter the alluvial valley where the flat slopes drastically reduce the sediment-carrying capacity. Thus, a major part of the sediment load is dropped near the foot of the hills to form the cones peculiar to the area. These cones rise considerably above the surrounding land and fan out in a westerly direction. Sediment deposits on the cone area of Big Sand Creek, a hill tributary of the Yazoo River, are typical. The magnitude and rate of these deposits were computed from range profiles made in 1920 and those obtained 27 years later, in 1947, and 10 years thereafter, in 1957. This paper briefly describes the watershed, the range profiles, and the amount of sediment deposits. Figures 1 and 2 show the sand bottom and the caving banks of Big Sand Creek. Figure 3 shows typical sand deposits around cabins.

Watershed Conditions

Big Sand Creek lies in Carroll and Montgomery Counties, Miss., and flows in a westerly direction, entering the Yazoo River at Greenwood, Miss. Figure 4 shows the drainage basin. The hill area of the stream, which lies east of



FIGURE 1. — Channel of Big Sand Creek 2.5 miles west of Carrollton.



FIGURE 2. — Bank of Big Sand Creek near Valley Hill.



FIGURE 3. — Typical sand deposits.

Valley Hill, is about 18 miles long and contains 110 square miles. Elevations in this area vary from 150 ft., mean sea level, near Valley Hill, to 520 ft., mean sea level, at the northeast divide. Soils in the upland part are within the loess resource area. The loess is quite deep in the western part and decreases in thickness to the east.

The streams and large gullies have eroded through the loess into the sands and clays of the underlying Coastal Plain formation. The easy erodability of these soils is a large factor in the huge amount of sediment deposits in the delta area. Practically the entire upland watershed is affected by sheet and gully erosion, which is most severe in the area east of Carrollton.

The delta area extends west of the bluff line near Valley Hill and contains the alluvial cone on which the range profiles were made. The local drainage district constructed a channel and levee system in the delta part in 1922–23 in an attempt to carry flows from the hills to the Yazoo River. The carrying capacity of these levees rapidly deteriorated because of large sediment deposits in the floodway, until at the present time they are crevassed by flows exceeding about 5,000 c.f.s. The levees are con-

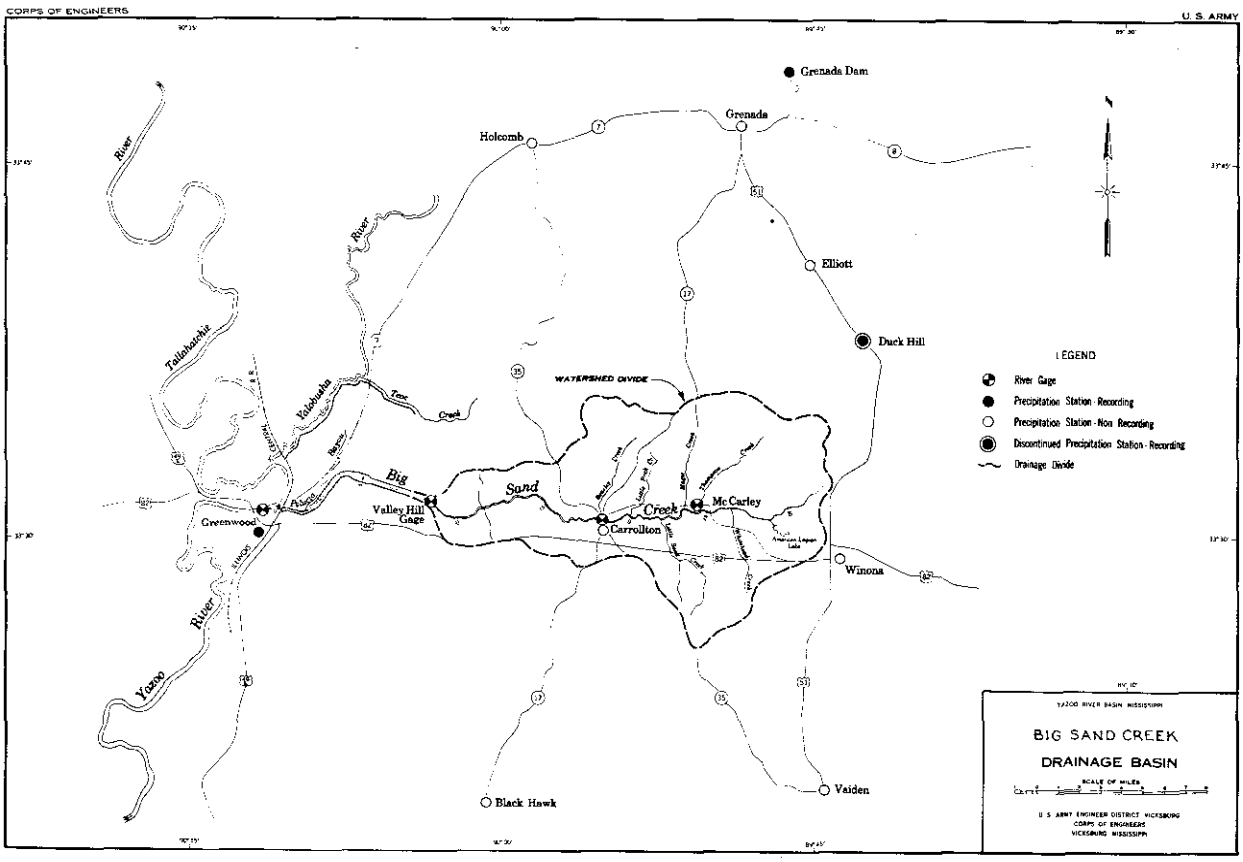


FIGURE 4. — Drainage basin of Big Sand Creek.

structed of sandy material and crevasses occur almost every year, with several in some years. The poor soil cover in the upland area is conducive to rapid runoff, with factors as high as 90 percent occurring during winter and spring rains. Floodflow slopes in the hill area range from 5 to 60 ft. per mile. The maximum flow of record is estimated as 33,000 c.f.s. at Valley Hill, the western edge of the hill area. The average annual precipitation is 52 in. The maximum hourly rainfall at Greenwood is 2.7 in., and the maximum 24-hour amount is 11.8 in.

Range Profiles

During a drainage survey for local interests in 1920, the Elliott and Harman Enginecring Co. of Peoria, Ill., obtained ground profiles in a north-south direction on four section lines in the delta area of Big Sand Creek. Figure 5 shows the location of these profiles. During field surveys in 1947 for the Greenwood Protection Works, the Vicksburg Engineer District resurveyed these ranges. Figure 6 shows the deposition between the 1920 and 1947 surveys. These deposits were, to a large extent, the result of sediment dropped by crevasse flows through numerous and widespread breaks in the Big Sand levees near the hill line during this period. The four ranges were profiled again in 1957. The 1957 profiles are not shown in figure 6, since they did not differ greatly from the 1947 survey.

Sediment Deposits

Table 1 gives the amounts of sediment deposits on the cone area as computed from the range profiles and field investigations. Amounts

TABLE 1.—*Sediment deposits on cone area*

Time and location	Total	Sediment deposits per year		Per year per square mile
	Cu. yd.	*Cu. yd.*	*Acre-feet*	*Acre-feet*
1920–47 (27 years):				
Upper area.....	13,151,000	487,000	302	2.75
Lower area.....	12,254,000	454,000	281	2.55
Total.....	25,405,000	941,000	583	5.30
1947–57 (10 years):				
Upper area.....	810,000	81,000	50	0.45
Lower area.....	0	0	0	0
Total.....	810,000	81,000	50	0.45

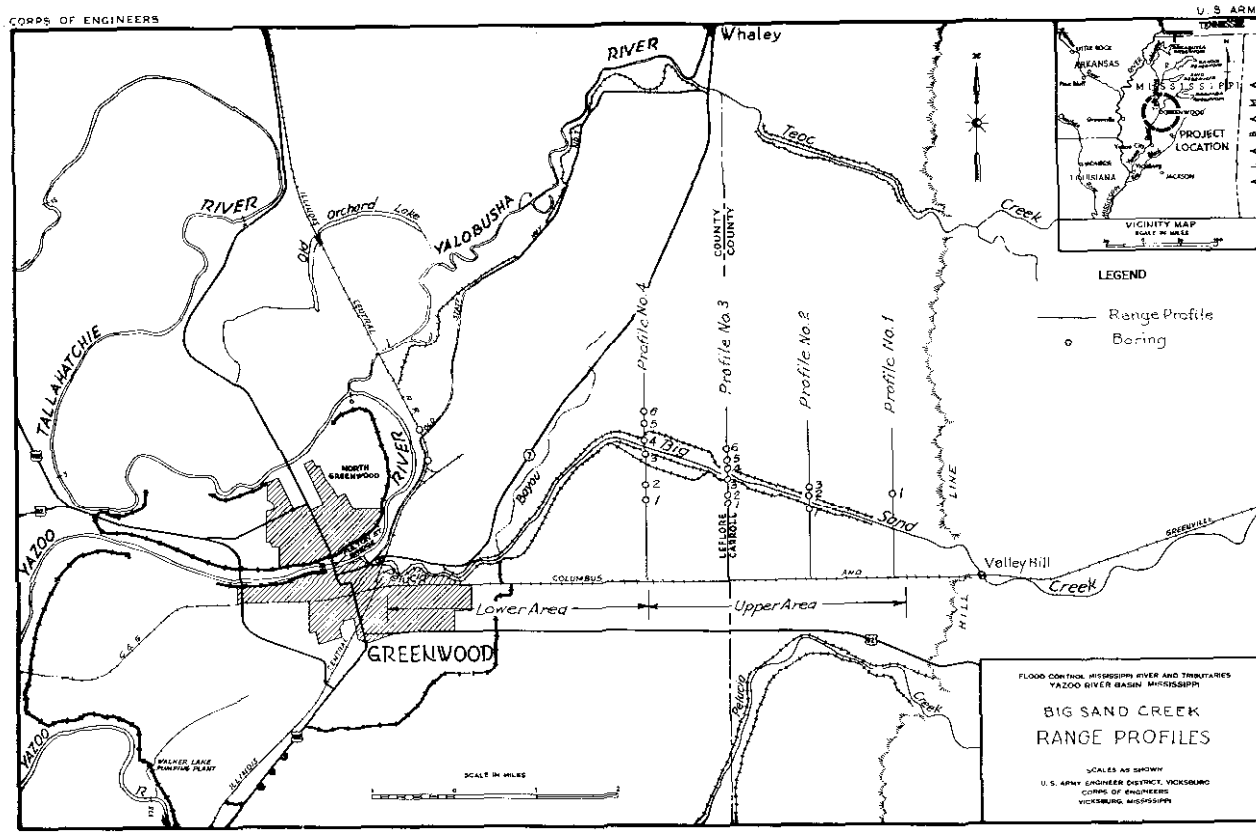


FIGURE 5. — Location of range profiles in delta area of Big Sand Creek.

of sediment deposits shown in table 1 for the upper area were computed directly from the range profiles. Amounts for the lower area were computed from depths of deposits obtained by field reconnaissance. Deposits on the cone are composed mainly of fine sands, silts, and clays. No estimates of deposits in the lower area were obtained in 1957; however, deposits in this area for the period 1947–57 are considered to be minor.

A major part of the difference between the rates of deposits on the cone area for the two periods is attributed to the improved maintenance of the levee system. During the latter period crevasses were limited in extent and were closed as soon as possible, thereby preventing long periods of flow through the crevasses and thus most of the sediment was transported to the Yazoo River. Evidence of this is indicated by comparative thalweg profiles of the Yazoo River in figure 7, which show considerable fill between 1940 and 1962. However, Big Sand Creek is only one of many small tributaries that, in addition to the major tributaries, contribute to the sediment load of the Yazoo River.

Estimates made in connection with the Mississippi River and Tributaries Project Review by the Soil Conservation Service in a report for the Mississippi River Commission entitled, "Report on Present and Anticipated Floodwater and Sedimentation Conditions, Yazoo Headwater Project, Part II, Big Sand Creek," dated August 1957, agree closely with the estimates made by the Vicksburg District from actual measurements. The estimates were based on empirical formula, analytical computations, and experience. The following tabulation shows the source of sediment and annual delivery rate to the delta area of Big Sand Creek as estimated in the Soil Conservation Service report.

Source [1]	Estimated gross erosion	Estimated delivered to delta area
	Acre-ft./yr.	*Acre-ft./yr.*
Sheet	648.5	129.7
Gully and roads	331.8	199.1
Streambank	392.3	392.3
Total	1,372.6	721.1

[1] Does not include any material from streambed since channel beds are believed in equilibrium.

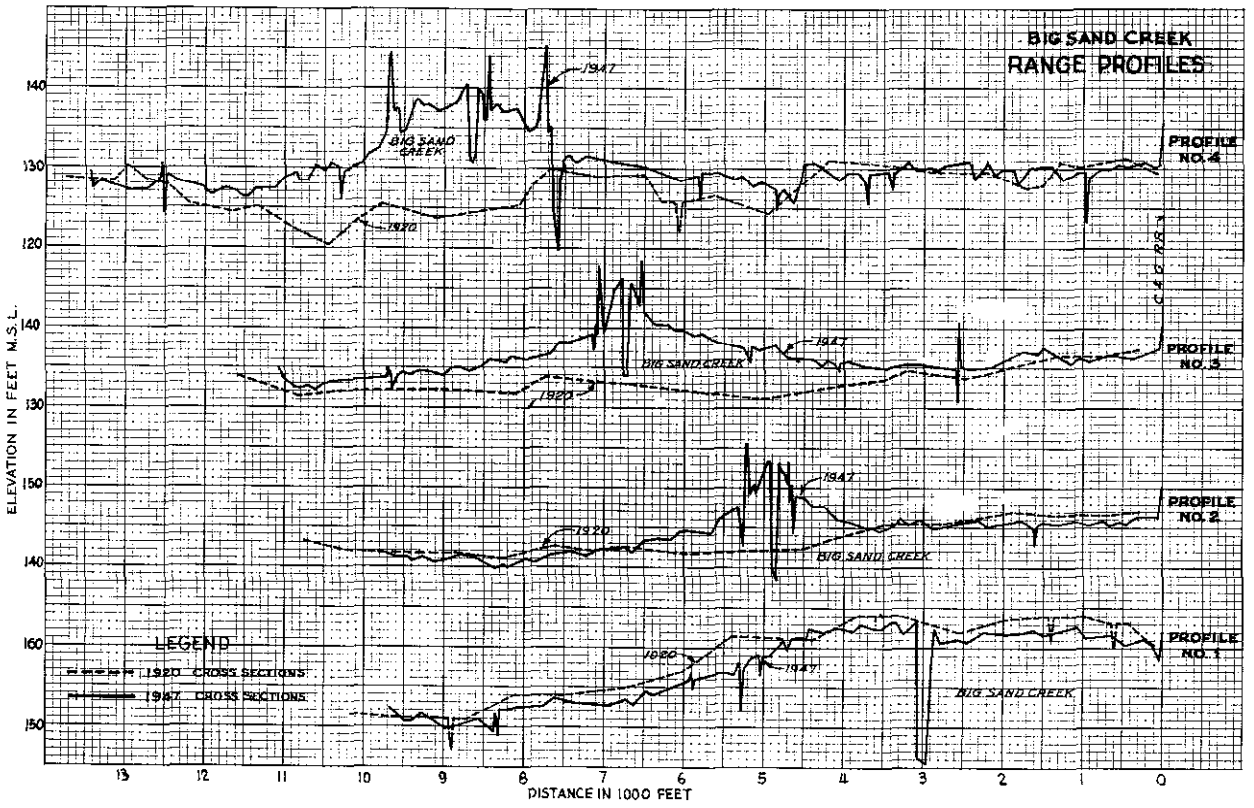


FIGURE 6. — **Range profiles of Big Sand Creek, 1920 and 1947.**

Summary

Big Sand Creek is one of the few areas where it was possible to obtain actual measurements of deposits on the ground as a basis for estimates of sediment loads. Although the estimates obtained are not highly accurate, they are reasonable and certainly serve to indicate the magnitude of the problem. The reduced rate of deposit in the cone area during the 1947–57 period is attributable to the fact that levees were maintained to reduce crevasse flow greatly and the opportunity for deposit in the cone area. Thus, a far greater part of the sediment load was transported to the Yazoo River than was the case during the earlier period. This has contributed to the filling in the Yazoo River in the 50-mile reach below Greenwood, as shown in figure 7.

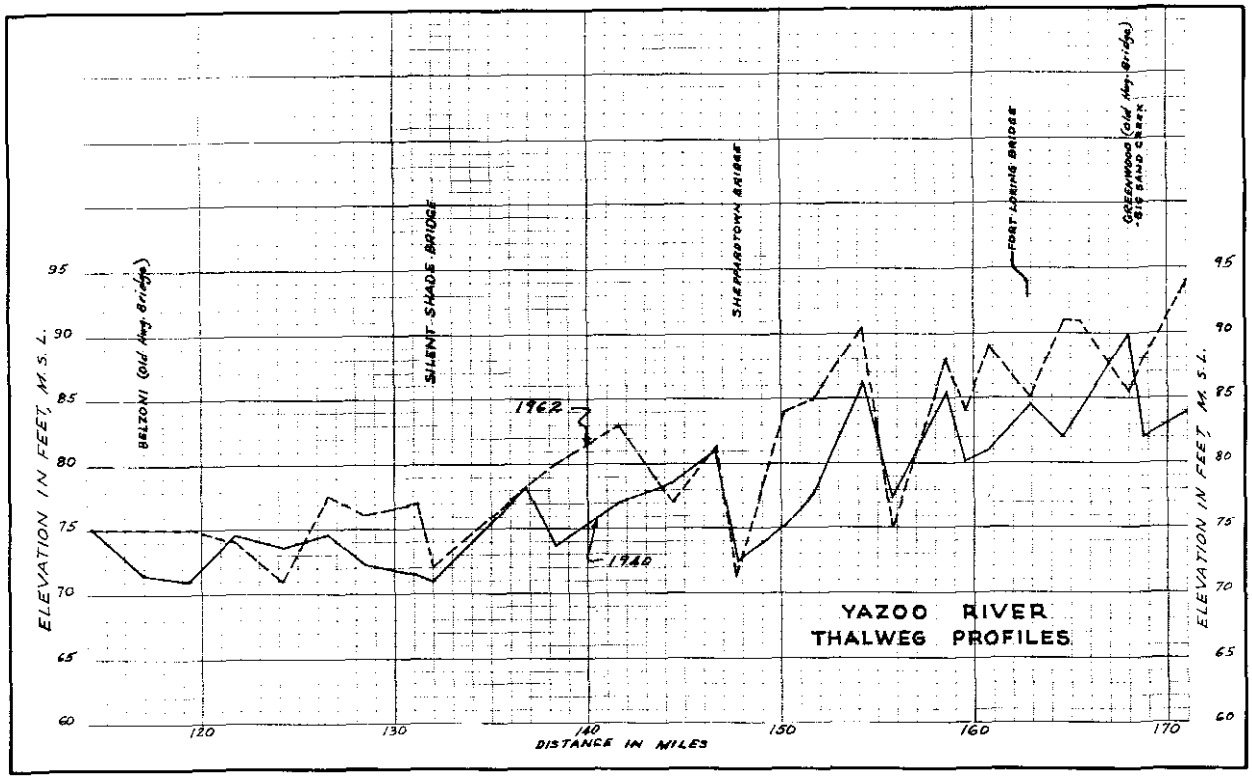


FIGURE 7. — Thalweg profiles of the Yazoo River, 1940 and 1962.

IMPROVEMENT OF THE NAVIGABILITY OF THE COLUMBIA RIVER BY DREDGING AND CONSTRICTION WORKS

[Paper No. 53]

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Synopsis

The Columbia River navigation channel from its entrance at the Pacific Ocean to the Portland-Vancouver area is maintained to dimensions of 35 by 500 feet by the removal of 12.2 million cubic yards of sediment each year. Constant improvement of the navigability of this 100-mile waterway has been made by the construction of permeable groins, removal of sediment by hopper and pipeline dredges, and use of dredge spoils for river constriction works. The controlling depth of the river was 12 feet in 1885; it is now 36 feet.

This paper reviews some work performed between 1957 and 1961 without giving details of earlier works on the river. The information is factual, not theoretical, and is based on hydrographic surveys of the river. The use of permeable groins and dredged sand in redirecting the river and improving channel depths on one bar is reviewed. The use of advance maintenance dredging to guarantee the availability of project depth all year, rather than just after completion of dredging, is considered. The success of this program in obtaining greater channel depths on a year-round basis is reviewed.

Several problems bearing on the maintenance of this navigation project are presented.

Introduction

The Lower Columbia River navigation channel from its mouth at the Pacific Ocean to the mouth of the Willamette River in the Portland-Vancouver area is an authorized project 35 feet deep by 500 feet wide. The channel is 98.5 miles long and dredging of approximately 9,800,000 cubic yards of river sediment is required annually through 26 river bars whose total length approximates 50 miles. In addition to this, an average of 2,400,000 cubic yards is

dredged from the Columbia River entrance bar each year. However, the term "river sediment" is slightly misleading, as the material dredged from the lower Columbia River consists almost entirely of fine to medium sand with very little silt to no clay. Absolute density of the material averages 2,700 grams per liter. The average length of the 26 river bars is 8,200 feet; the shortest is 2,000 feet and the longest is 14,000 feet. The other 48.5 miles of the channel under discussion is self-maintaining to river depths greater than 35 feet for a width of at least 500 feet. The river in its natural state had a controlling depth of 12 feet at St. Helens Bar (mile 86) in 1885.

River Constriction Works

It is recognized that the shoaling pattern on any river bar is a result of the interrelation of many factors. Principal among these factors in the Columbia River are:

(1) Characteristics of the annual river runoff, or "freshet," and its effect on the amount of bank erosion upstream.

(2) Action of river constriction works upstream and their tendency to pass material onto the bar being investigated.

(3) Action of river constriction works on the subject bar and the resultant shape and area of the river cross section.

(4) Placement of the navigation channel in relation to the thalweg of the natural river.

(5) Placement of the channel in relation to bends in the river.

(6) Type of sediment being transported by the stream.

It is not intended to present information where all the above variables have been held constant. However, by holding certain of these variables constant and changing others, a radical change in the channel over several years undoubtedly will be attributable to recognizable causes.

For purposes of comparison of shoaling, the years 1957 and 1961 were used because of the similarity of the flood pattern. Figure 1 shows the river hydrograph at Vancouver, Wash., for these 2 years. For purposes of comparison, since the freshets were similar, factor (1) mentioned above can be considered to have been constant when comparing shoaling in 1957 and 1961. No constriction works were installed for a distance of 20 miles upstream of St. Helens Bar between 1957 and 1961, so factor (2) was considered to be constant. Since the channel alinement was not changed and there is little change in the nature of the material from year to year, factors (4), (5), and (6) were likewise constant. Therefore, any change in shoaling on St. Helens Bar between 1957 and 1961 is primarily attributable to the change in river cross section caused by the addition of river constriction works.

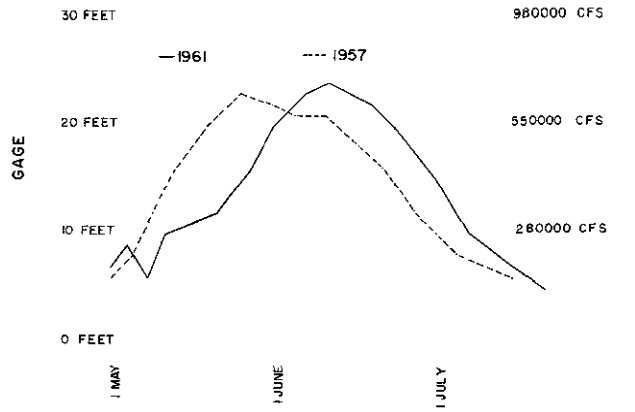


FIGURE 1.—Comparative flood-stage-discharge hydrographs, Columbia River at Vancouver, Wash., for 1957 and 1961.

Figures 2 and 3 are aerial photographs of St. Helens Bar, taken in 1957 and 1961. Through the years several permeable groins (i.e., pile dikes) have been constructed on this bar. In 1959 three additional groins totaling over 1,000 lineal feet were constructed, and in 1959 and 1960 hydraulic fill totaling 1,250,000 cubic yards was placed to supplement the permeable groins on the right bank in redirecting the river. Although this sand closure was breached the following year (see fig. 3), the effect of it on the navigation channel has been considerable. In 1957 the infill of sand onto this bar during the freshet was 856,000 cubic yards; in 1961, it was 364,000 cubic yards. It is believed this reduction in infill was a direct result of constructing the additional length of permeable groins and placing the 1,250,000 cubic yards of sand over and between these groins in the intervening years. As can be seen from the aerial photographs, the river cross section at this point was narrowed considerably and the river diverted completely by the groins and fill rather than partially diverted, as was the case with permeable groins only. Unfortunately, the hydraulic fill was only raised to elevation 22 feet and was topped by the freshet in 1961. It was breached in several places but was still effective in 1962. These breaches will be filled and the entire closure raised to flood-free elevation as material is available from maintenance dredging.

Slaughters Bar at mile 66 is 20 miles below St. Helens Bar. The intermediate bars between these two contain 30 permeable groins located so that a great deal of material is believed to pass through these bars. Slaughters Bar represents the natural settlement place for some of

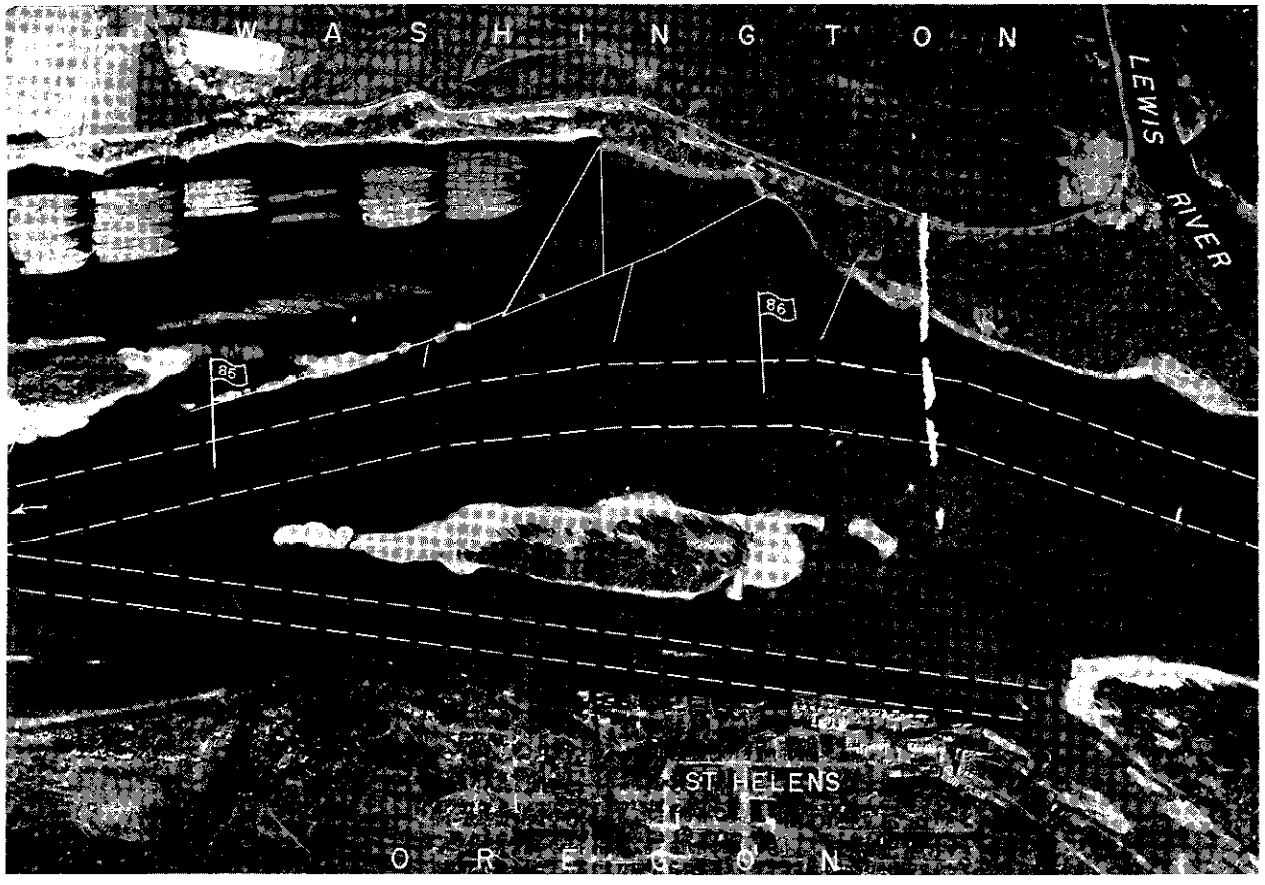


FIGURE 2. — St. Helens Bar, Columbia River, November 1957.

the material because its cross section is too wide to support a 35- by 500-foot channel. As shown in figure 4, comparison of a naturally maintained section at the head of this bar at mile 67.0 with a middle section at mile 66.0 with shoals above 35 feet discloses that at least 300 feet of excessive width should be eliminated from the section at mile 66.0 to make it self-maintaining. If this were accomplished, the increased velocities generated thereby would cause the bottom to scour to a pattern similar to that shown for mile 67.0. However, due to strong objections from local interests, it has not been feasible to narrow this section by creating a middle ground or island built from dredge spoils and stabilized by permeable groins. While bars above Slaughters Bar are improving each year, it is significant that the trend on Slaughters Bar is toward worsening conditions. Table 1 demonstrates the trend of shoaling of a section of this bar.

It can be seen that as other reaches of the river are improved by constriction works, it may have an adverse effect on the navigability of unimproved areas downstream subject to shoaling. It is assumed this is due to the increased inflow of bedload. It is not known if a reliable quantitative relation can be established between the amount passing through an upstream bar and the amount of material deposited on a lower bar under the conditions present in the Columbia River. The important question here is whether or not an increased bedload made up of material passing through upstream bars requires a much greater velocity to keep it moving. In other words, a bedload equal to X

TABLE 1.—*Shoaling on a section of Slaughters Bar*

Year	Shoal area after freshet [1]	Dredging depth	Infill into channel during freshet
	Sq. ft.	*Ft.*	*Cu. yd.*
1957 [2]	258,000	37	225,000
1958	710,000	37	[3]
1959	1,050,000	37	[3]
1960	1,347,000	37	[3]
1961 [2]	1,577,000	37	320,000

[1] Meaning total area within the navigation channel that is shallower than 35 feet.
[2] Similar freshet pattern.
[3] Not calculated.

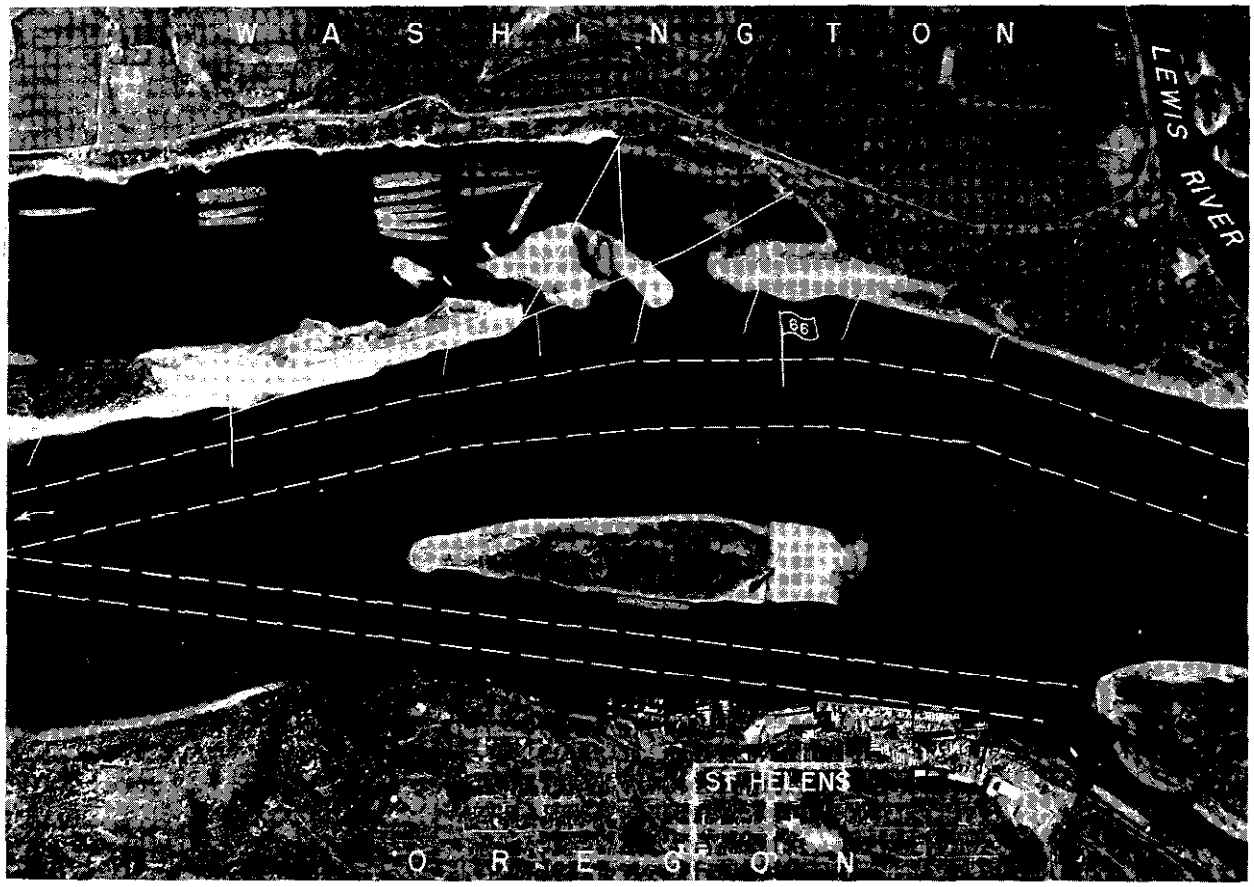


FIGURE 3. — St. Helens Bar, Columbia River, November 1961.

is passed through a bar by action of velocity V. When X is increased, will V necessarily have to be increased to prevent further shoaling on the downstream bar? The answer to this is not clear for the Columbia River.

Advance Maintenance Dredging

Experience indicates that nearly all of the shoaling on the Columbia River takes place dur-



FIGURE 4.—Comparison of river cross sections at middle and head of Slaughters Bar, Columbia River.

ing the several weeks of the spring runoff or freshet. This means that after the river level returns to normal there is nearly 50 miles of river shoaled above project depth. This presents a critical situation from the standpoint of navigation, as the deeper draft ships are forced to delay arrival and sailing times so high tides can be utilized. During low river stages tides affect the project channel approximately 8 feet at the seaward end and 2.5 feet at Portland. Dredging capacity of the district is taxed immediately following the freshet in order to deepen the channel in the shortest possible time.

It has been customary to dredge project depth plus 2 feet allowable overdepth, or to 37-foot depth on the Lower Columbia River. Maximum shoaling during the average freshet is 6 to 8 feet, leaving a controlling depth of approximately 30 feet on some bars. So that the channel may be at project depth the year around and in order that dredging may be scheduled on a year-around basis, Portland District, during the last several years, has endeavored to perform what is called advance maintenance

dredging. This means that rather than dredging to 37 feet, the channel is excavated to a depth of 40 or 42 feet; i.e., project depth, plus 3 to 5 feet of advance maintenance dredging plus 2 feet of allowable overdepth dredging, to allow for the inaccuracies of the dredging process.

It can be seen under this plan that after 6 to 8 feet of infill, there would still be left the project depth of 35 feet, and all shoaling would have taken place below project depth. The advantages of this plan are obvious. A good example of this practice is maintenance of Willow Bar at river mile 97. Table 2 shows results of the advance maintenance program from 1957 to 1961, inclusive: Table 3 shows conditions at bars after freshets of 1957 and 1961.

TABLE 2.—*Willow Bar maintenance by advance dredging*

Year	Dredging depth previous year	Shoal after freshet	Controlling depth at centerline	Number of soundings at depth [1]					
				29	30	31	32	33	34
	Feet	*Sq. ft.*	*Feet*						
1957[2]	37	1,205,000	31–	6	4	12	17	33	47
1958	37	1,835,000	31–	7	5	16	19	51	84
1959	40	335,000	31–	1	6	3	4	11	22
1960	42	44,000	35	0	0	0	0	0	2
1961[2]	42	48,000	34	0	0	0	0	1	7
1962	40	197,000	33	0	0	0	1	6	9

[1] Each sounding represents approximately 7,500 sq. ft.
[2] Similar flood patterns.

TABLE 3.—*Columbia River bars—comparison of condition after freshets of 1957 and 1961*

Bar	1957		1961		Remarks [1]
	Shoal area	Controlling depth on centerline	Shoal area	Controlling depth on centerline	
	1,000 sq. ft.	*Ft.*	*1,000 sq. ft.*	*Ft.*	
1. Desdemona	200	35	480	36	Bars 1–9 are hopper dredged. Shoal areas may remain after dredging due to inaccuracies of the dredging method.
2. Flavel	100	35	150	37	
3. Upper Sands	690	34	60	35	
4. Tongue Point	610	33	330	34	
5. Miller Sands	1,810	33	2,080	33½	
6. Pillar Rock	180	35	325	30	
7. Brookfield	710	33	900	31	
8. Skamokawa	490	35	280	32½	
9. Puget Island	250	35½	200	35	
10. Wauna	2,080	30½	680	35	Dredged to −40 feet in 1960.
11. Westport	710	33	0	38	Dredged to −42 feet in 1960.
12. Eureka	750	29½	150	33	Do.
13. Gull Island	200	34	120	37	.....
14 Stella-Fisher	3,350	29½	420	36	Dredged to −42 feet in 1960.
15 Walker Island	1,420	29½	340	36	.....
16. Slaughters	1,930	33	2,920	31½	.....
17. Lower Dobelbower	1,540	31	310	35	Dikes extended in 1959.
18. Upper Dobelbower	1,390	31	310	31½	Do.
19. Kalama	2,120	31	750	33½	New dike in 1960; also dredged to −42 feet in 1960.
20. Lower Martin Island	120	35	70	34	.....
21. Upper Martin Island	700	33½	1,380	29	.....
22. St. Helens	2,240	30	100	39	New dikes in 1959; also dredged to −42 feet in 1960.
23. Warrior Rock	140	37	160	36	.....
24. Henrici	780	34	1,320	30½	Not dredged in 1960.
25. Willow	1,210	31	50	34	Dredged to −42 feet in 1960.
26. Morgan	120	32½	120	35	Not dredged in 1960.
Average	994	32.8	539	34.2	Reflects effect of both dikes and advance maintenance dredging.

[1] All bars from Nos. 10 to 26 were dredged to 37 feet the previous year unless otherwise noted. Bars 1 to 9 were dredged to a maximum of 37 feet with minor shoals of 34 feet remaining at the completion of dredging.

It is apparent from the preceding tabulations that advance maintenance dredging accomplishes its objective of making greater depths available on a year-around basis. Although it is recognized that there is an obligation to maintain this channel to authorized depth, it is of interest to compute the benefit-to-cost ratio of making the increased depths available. It is obvious that overdepth must be originally dredged and its cost amortized over the life of

the project. Calculations show that approximately 10 million cubic yards of advance maintenance dredging would need to be accomplished initially so that all shoaling would take place below project depth. Based on an assumed cost of 22.5 cents per cubic yard and on a proposed life of 30 years, this dredging would cost about $75,000 per year (no interest included). In addition to this, it seems reasonable that annual dredging costs would increase due to the greater shoaling taking place; i.e., because velocity equals flow/area, it follows that an increase in cross section will bring about increased shoaling due to the decreased velocities. Means of estimating this increased shoaling are not presently available for this project. It has been determined that, in general, 5 feet of advance maintenance dredging will increase the total river cross section at a typical bar by about 5 percent. For purposes of economic evaluation, it is assumed that the shoaling will be increased by that amount.

Experience shows that an expenditure of $1,800,000 (1957 dollars) is required to keep the channel to depth of 35 feet for 215 days per year and to depths of 29 to 31 feet for 150 days per year. The additional cost to keep the channel to 35 feet all year is as follows:

(1) Initial cost amortized at 30 years, with no interest:

10,000,000 cubic yards at 22.5 cents per cu. yd. = $75,000

(2) Increased annual dredging costs:

5 percent × $1,800,000 = $90,000

Total increased annual cost to obtain 35-foot project depth year around..... $165,000

It is assumed that various increments of this sum would maintain channel depths as follows:

Depth all year (feet)	*Annual cost of dredging in excess of $1,800,000*
30	$0
31	33,000
32	66,000
33	99,000
34	132,000
35	165,000

The determination of the benefits to navigation largely depends on how much time deep draft ships are required to wait for the tides to aid in traversing the project during the 150 days after the freshet. No detailed study was made of subsidiary benefits, such as eliminating the necessity to stop off in other ports, as the reason for the study was only to ascertain whether advance maintenance dredging is economically justified.

On the Columbia and Lower Willamette Rivers project, deep-draft vessel traffic during the 5 months in calendar year 1961 when project depth was not available, was as follows:

Draft at stern of vessel:	*Approximate number of vessel trips during 5-month period*
33–34	5
32–31	151
30–29	173
28–27	160

As was stated above, controlling depth is 30 feet for 5 months per year when only $1,800,000 is spent for dredging.

It is further estimated that under these conditions each vessel noted above loses 3 hours per trip awaiting a favorable tide during the 5-month period. It is also assumed that reduced ship draft of 2 feet for squat and 2 feet for safety were suffered. If this wait of 3 hours, as developed above, is eliminated, benefits will accrue as shown in table 4.

TABLE 4.—*Benefits accrued from maintaining channel depths of Columbia River*

Draft, including squat and safety factor (Feet)	Vessels in 5 months	Awaiting favorable tide, hours at 3 hours per vessel	Benefits (hours×$200)[1]
	Number	*Hours*	
31 and greater plus 4 feet	156	468	$93,600
29 plus 4	173	519	103,800
27 plus 4	160	480	96,000
Total dollars			293,400

[1]Vessel hour assumed to cost $200.

Figure 5 shows these values plotted. Also plotted is the cost to attain these various depths as described above. Dredging advance maintenance at all depths up to 35 feet is justified. The benefit/cost ratio to maintain 35 feet all year is 293,400/165,000 or 1.78:1.

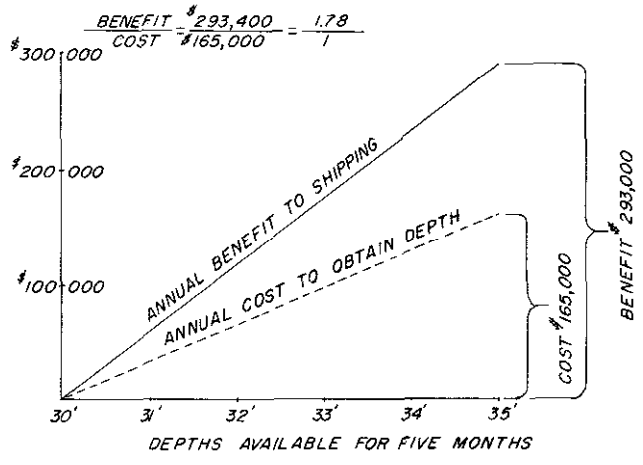


FIGURE 5.—Benefit cost ratio to obtain 35-foot depth year around.

Advance maintenance dredging in addition to the benefits to navigation is advantageous. A large pipeline dredge requires a relatively deep infill bank for efficient dredging. Figure 6 demonstrates the reason for this. To assure an efficient dredging program, the pipeline dredge should work against the heaviest possible bank.

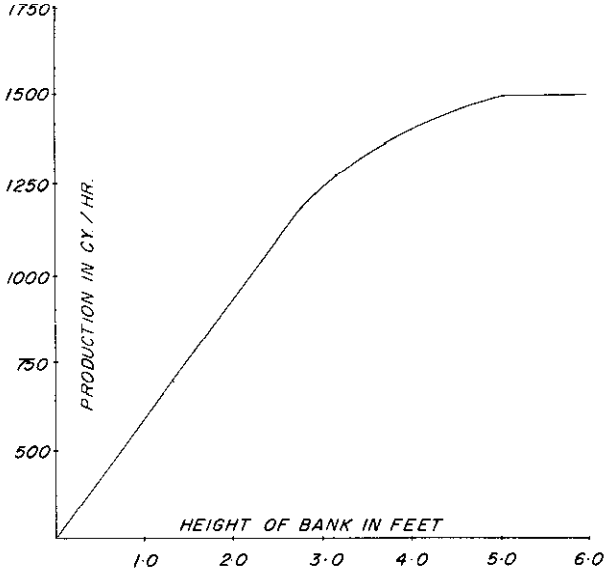


FIGURE 6.—Production curve for 30-inch-pipeline dredge *Clackamas* pumping through 2,000-3,000 feet of discharge line, Columbia River.

One means of doing this is to dredge advance maintenance and allow bars to shoal for 1 or more years. That is, a bar which normally shoals 2 to 3 feet per year could be dredged to 6 feet of overdepth every other year. The cross section would be slightly increased, which would in turn increase the tendency of the bar to infill the following year. However, dredge production may be increased greatly or at least enough to more than offset this increased shoaling. An increase of dredging bank from 3 feet to 6 feet increases production of the dredge *Clackamas* from 1,250 to 1,550 cubic yards per hour (fig. 6). Since dredge cost is virtually constant, regardless of production, this would reduce unit costs by 24 percent.

From the standpoint of dredging efficiency, it is unfortunate that shoaling does not take place by consistent and level buildup on river bars. The shoaling is mostly in sand waves and, therefore, a 32-foot sounding may be followed closely by one of 42 feet. This is clearly shown in the reproduction of a section from a fathogram shown in figure 7. It is obvious that this situation does not lend itself to pipeline dredging. The trailing suction hopper dredge is one method of eliminating these waves. It is believed that, if available, trailing suction hopper dredges can be utilized to great advantage to remove the scattered high spots in a channel. This allows the rest of the bar to build up toward a heavier bank, which in turn assures later efficient use of pipeline dredges.

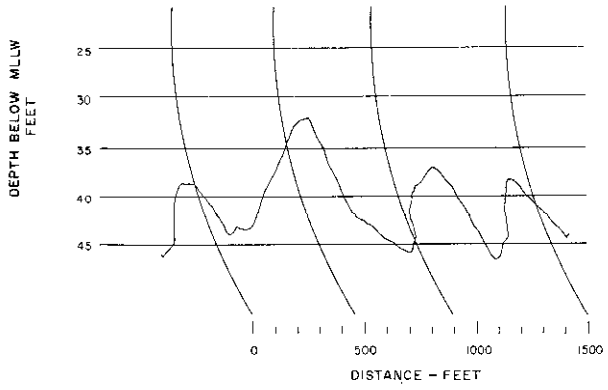


FIGURE 7.—Fathogram along centerline navigation channel, showing sand wave shoaling.

Use of River Sediment

We tend to lose sight of the fact that river sediment can be and in many places is a most valuable natural resource. Effectual usage of dredged sand depends upon the location and the foresight and planning of the engineers dealing with the sedimentation problem. In the Columbia River, dredge spoils are utilized for river control and constriction works and for development of recreational areas.

The use of sand for river control and constriction works certainly represents the highest and best use of sediment on the Columbia River project. The use of sand for constriction works has generally been in conjunction with the permeable groin building program. Permeable groins of untreated piling are used almost exclusively for river training in this area. Several permeable training groins have been built on river bends, where permeable spur groins are not sufficient to turn the river.

Normally, in constructing these groins, two rows of piling are driven with staggered pattern, so the effective spacing is 2½ feet. The two rows are separated by a waler which is securely bolted to all piling. The cutoff elevation varies between +6 and +10 feet on Columbia River datum. A rock blanket is placed after the piling is driven to protect the structure from scour. Because of the permeability of these groins, it is almost always desirable to fill between the groins with dredge spoils to supplement natural accretion. The ultimate effect is like that of a long training groin following the desired curvature of the river and moving the

bank line channelward. This is believed to be the most economical solution to the problem of dredge spoil disposal. However, when the spur groin method is used on sharp bends, excessive scouring and erosion of the toe of the slope of the sand fill presents a problem.

Constricting the river by placing sand fill without the benefit of spur groins has been fairly successful in some areas. However, to be fully effective, sufficient sand must be available to bring the fill to floodfree elevation and must be placed continuously channelward from normal high waterline. Even then, the toe of slope and the side of slope is endangered during subsequent freshets. The fill at St. Helens Bar (fig. 3) is thought to be a good example of the combined use of permeable groins and dredge spoils to obtain river control.

The use of river sand for recreational uses has been limited to establishing beaches or nourishing existing beaches for use by fishermen, picnickers, and boaters. Cooperation with State, county, and municipal groups has resulted in numerous public fishing and recreational beaches being created in past years in those reaches where use of dredge spoils is adaptable to this activity.

Conclusion

The Portland District has made great strides in improving the navigability of the Columbia River during the past years. River constriction structures and the use of dredge spoils as well as advance maintenance dredging have brought about a marked improvement to the ship channel.

There are several major problems whose solution is not readily apparent but must be accomplished to insure continued improvement of the Columbia River navigation channel. These problems are:

1. Determination of a low-cost method for removing the peaks from the sand waves that develop in the ship channel and interfere with navigation.
2. A sound, harmless method of determining percentages of material that reenters the navigation channel under various dredging disposal methods to allow logical derivation of costs of various dredging disposal procedures.
3. Determination of how much shoaling increases when a channel is overdredged in depth but river width remains constant.

Solution of these problems will enable the Portland District to continue improvement in the navigability of the lower Columbia River.

CHANNEL ARMORING BELOW FORT RANDALL DAM

[Paper No. 54]

By ROBERT H. LIVESEY, *hydraulic engineer, U.S. Army Engineer District, Omaha*

Synopsis

One of the major problems encountered in the design of outlet facilities for reservoir projects, particularly those planned for alluvial streams, is the determination of tailwater levels for optimum operating conditions. Generally the degradation estimates are based upon various methods of analytical study involving numerous related variables, many of which have not as yet been formulated into adequate mathematical terms, or upon engineering judgment derived from observation and study of experienced conditions. The variables in this problem include the transport relation for the fine riverbed material, the amount of coarse material available to armor the bed, the presence of such fixed degradation controls as rock outcrops and clay ledges, the pattern of reservoir releases, and the degree to which the banks are stabilized to prevent the channel from widening or meandering.

This paper is directed toward the phenomenon of channel bed armoring. It describes results that occurred from the segregation of a small quantity of gravel on the surface of the Missouri River streambed below Fort Randall Dam in South Dakota. Extensive sampling of the riverbed surface, plus deep exploration of the subsurface strata of both the channel and the overbank areas, prior to construction of the dam, indicated only a small percentage of gravels down to the probable lower limit of degradation. Yet a one-grain thickness of gravel has accumulated over the entire bed surface for a distance of several miles, overlaying erodible sands, to halt significantly the desired degradation trend. This paper describes the natural characteristics of the Missouri River reach below Fort Randall Dam and conditions after 10 years of degradation. Detailed photographs of the bed armoring layers will be of interest. The tailwater degradation trend is presented. The armor layers and degradation trends obtained in one series of flume studies compare with the observations below Fort Randall.

Introduction

Seldom does an engineer engaged in hydraulic design work enjoy the latitude of having a complete and factual picture of the basic criteria

necessary for the proper analysis of a problem. When the analysis encompasses any sedimentation phenomena, particularly on alluvial streams, the problem immediately becomes further magnified due to the many interrelations that exist between numerous variables, which to date have been neither adequately identified nor formulated into usable mathematic terms. In addition, even adequate knowledge of several of the basic equilibrium factors is somewhat vague and unpredictable, or perhaps indiscernible altogether. To the young or inexperienced engineer these limitations might seem a ponderous block that necessitates extensive research and study of the many facets before attempting to resolve the problem. However, the answers necessary to establish design criteria for proposed projects cannot wait. They must be based upon the best analytical methods available at the time and upon judgment derived from the observation and study of experienced conditions.

This paper is focused upon the problems encountered in the design of outlet facilities for the Fort Randall Reservoir project on the Missouri River in South Dakota and specifically upon the phenomenon of the channel bed armoring that occurred. At Fort Randall the determination of tailwater levels for optimum operating conditions included an evaluation of future degradation trends and probable limits. Variables in this problem included the transport relation for the fine riverbed material, the amount of coarse material available to armor the bed, the presence of such fixed degradation controls as rock outcrops and clay ledges, the pattern of reservoir releases, and the degree to which the banks are stabilized to prevent the channel from widening or meandering.

The final design of the powerhouse and outlet works are based upon a detailed analytical study, which was supervised by eminent consultants in the sedimentation field and included extensive but limited field investigation surveys. This study indicated that an ultimate 15-foot lowering of the tailwater levels would occur from the expected regulation of flows. Since the closure of Fort Randall Dam in July 1952, the total tailwater lowering experienced to date has been only about 3.5 feet, with 2 feet occurring during the first 2 years. A small layer of gravel, one grain in thickness and overlying the erodible sands, has segregated out of the degrading bed and accumulated upon the bed surface for a distance of several miles to cause this significant reduction in the desired degradation trend. The following description of the characteristics of the Missouri River in this reach plus an account of the degradation phenomena observed since closure of Fort Randall Dam in 1952 may be of general interest to the engineer faced with the problem of predicting degradation.

The Fort Randall Dam and Reservoir project is located on the Missouri River in the south central part of South Dakota (fig. 1). It is operated as one unit of a comprehensive plan of development of the Missouri River basin. The reservoir has a capacity of 6,300,000 acre-feet and extends 111 river miles upstream to the Big Bend Dam and Reservoir project. Diversion of the river through the outlet works was accomplished in July 1952. Storage to modify significantly downstream discharges began in about June 1953.

Missouri River Below Fort Randall Prior to Closure of the Dam

The Missouri River below the Fort Randall Dam site is confined to a narrow valley by chalk bluffs spaced fairly uniformly about 1.5 miles apart. The valley is alluvial, with the channel bed composed primarily of sand that is constantly in a state of movement. Gravel pockets exist at infrequent and irregular intervals in the underlying stratum, with occasional boulder outcrops in the vicinity of local valley wall drains. The channel banks and valley floor are composed of sandy silt with occasional areas of clay that were deposited in overbank pockets during floods. The river channel itself is alined in a semimeandering shape, with an alternating series of straight reaches adjacent to the chalk bluffs and intermediate crossing reaches where the channel shifts diagonally across the valley to the opposite bluff contact. The channel averages about 2,000 feet in width and is generally divided into more than one channel by large wooded islands or low, shallow sandbars. Due to the extensive bluff contacts in this particular reach of the river, the channel alinement has not changed materially since 1892.

The natural flow pattern in this reach was characterized by a definite annual pattern. During the winter months the flows were greatly reduced, due to freezing weather and the accumulation of ice that eventually formed a solid cover over most of the reach. Usually in March or April the warmer temperatures induced surface runoff of the uplands snow pack, which caused ice breakups to occur. Normally this spring runoff would be relatively moderate in volume, but frequent ice jams would produce extensive flooding and record maximum stages. After the spring breakup the river would subside to below bankfull stages and maintain an irregular pattern dependent upon the frequency of the spring rains. A second rise, usually greater in volume, occurred about the first of June and was governed by the availability of runoff from mountain snowmelt. Frequently this rise

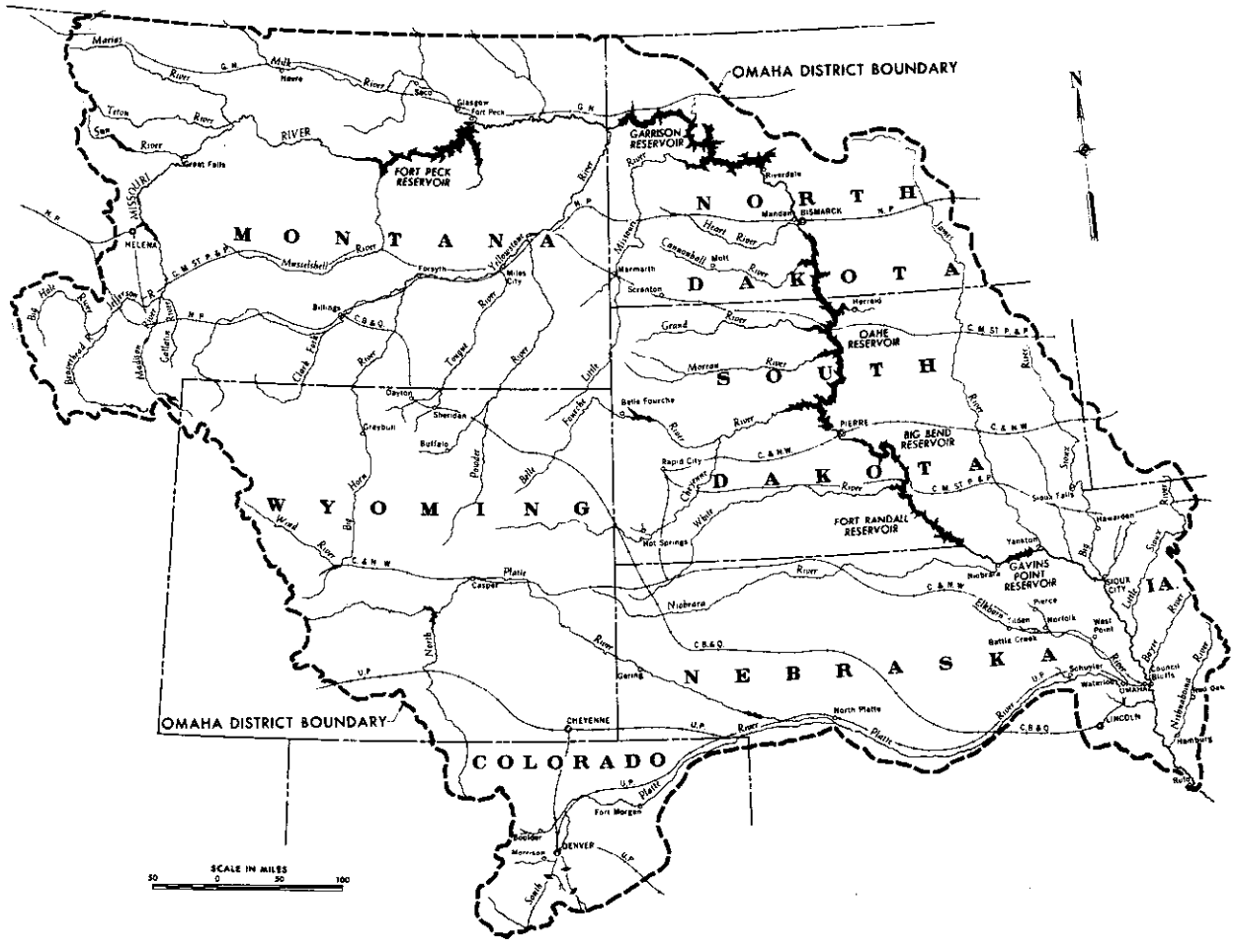


FIGURE 1. — The Omaha District and Fort Randall Reservoir.

was augmented by severe early summer thunderstorms. After the recession of this June rise, the flows continued at a reasonably steady but slowly diminishing rate through the late summer and fall until freezeup again occurred to repeat the cycle. The maximum flood of record at Fort Randall was 448,000 c.f.s., which occurred in April 1952, just prior to closure of the dam. The normal summer flows generally ranged from 16,000 to 70,000 c.f.s., and the winter flows between 9,000 and 15,000 c.f.s.

The general shape of the Missouri River channel, below the valley flood plain, is rectangular with an average width of 2,000 feet and bank heights varying between 8 and 15 feet. The natural channel has a bankfull capacity of about 150,000 c.f.s. and an average gradient of 0.8 foot per mile. At normal summer flow stages the active channel width varied between 700 and 1,500 feet and the mean depth between 4 and 11 feet. Greater widths and shallower depths occurred in the diagonal crossings between opposite bluff contacts.

The Missouri River has long been noted for its capability to transport a large and variable quantity of suspended sediments. It is estimated that the average annual sediment load transported past the Fort Randall Dam site was 148 million tons for the period 1898–1952. Based upon the particle-size distribution observed from 1943 to 1952 at Yankton, S. Dak., it is estimated that 30 percent of this annual load was sand, 30 percent silts, and 40 percent clays. The material composing the streambed surface of the natural channel consisted of fairly uniform, fine sand with a median size of 0.20 mm. A small percentage of larger material, generally well rounded glacial particles varying in size up to 1 inch, could be found in the deep holes at the downstream ends of submerged bars.

The available data on the original bed composition of the Missouri River near Fort Randall

are summarized by the mechanical analysis curves in figure 2. Curve 1 on this figure, based

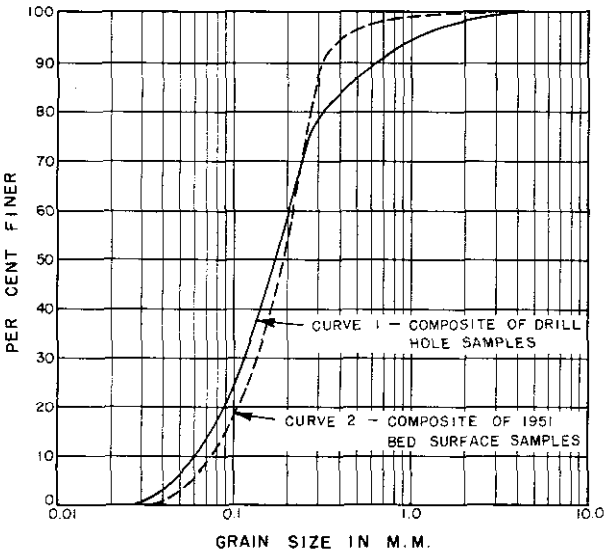


FIGURE 2. — Grain-size distribution of available bed materials as determined from drill hole and bed surface sampling.

upon drill hole samples, indicates the mean composition of the first 10 feet or so of material beneath the bed surface. Curve 2 is based on bed surface samples. The two curves up to the 80 percent finer size are similar, but above this point curve 1 shows significantly more coarse material than curve 2 for the bed surface. Applying a little hindsight at this point, it appears these curves may be somewhat misleading in that they probably show less coarse material than actually existed in the undisturbed conditions. For example, the BM–48 bed material sampler employed in obtaining many of the bed surface samples tended to be selective, due to its inability to always penetrate a gravel bed surface and retrieve the larger gravel particles. Second, neither the BM–48 nor the Straub bed material samplers are capable of obtaining samples that represent only the composition of particles forming the bed surface. These samplers bring up not only the surface layer of particles but also about one-tenth of a foot of the material underneath. Third, some gravel particles were probably lost from the churn drill samples between the bottom of the drill hole and the deck of the drill barge.

Reservoir Operation

The pattern of monthly average releases from Fort Randall Dam is shown in figure 3. The

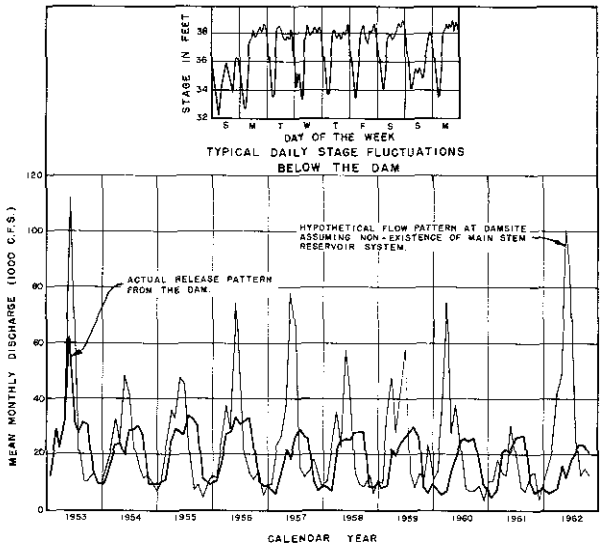


FIGURE 3.—Actual and hypothetical release trends from Fort Randall Dam.

regulatory effect of the reservoir system can be assessed by comparing the lower curve, which shows the actual releases, with the upper curve, which estimates the flows that would have occurred with no regulation. It can be seen that since 1953 the average monthly releases have followed a persistent pattern. During the navigation season the average release has been generally in the 20,000 to 30,000 c.f.s. range. During the winter months these flows were curtailed to 5,000 to 10,000 c.f.s. These flow patterns, however, do not portray the entire picture. Initial power generation from Fort Randall was started in 1954, and by 1956 all normal releases were being made through the eight turbines. Since this power project has been operated primarily as a "peaking plant," the resultant releases normally have a daily fluctuation from a low of nearly zero to a high of more than 40,000 c.f.s. during the navigation season and over 20,000 c.f.s. during the winter. Stage fluctuations resulting from the daily release pattern for a typical week during the navigation season are shown in the insert on figure 3. These power peaking operations have resulted in daily stage fluctuations up to 7 feet at the dam, decreasing to 5 feet at the U.S.G.S. measurement station 7 miles below the dam and 4 feet at Greenwood, located 15 miles below the dam.

Effects of Ten Years of Operation

For the first 2-year period after closure, the tailwater levels lowered at a rate of 1 foot per year, but since 1954 the annual rate has diminished to about one-tenth of that value (fig. 4). As also noted on figure 4, the initial degradation

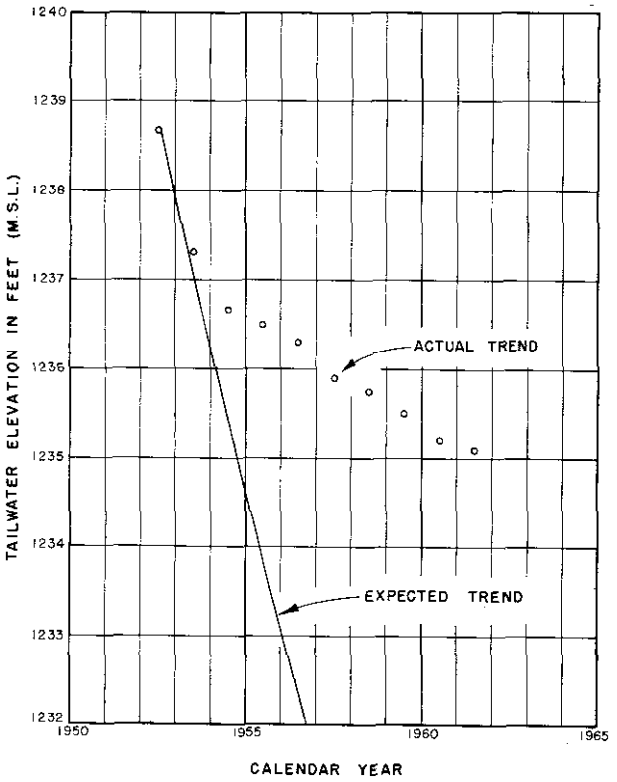


FIGURE 4. — Tailwater elevation trend since closure of Fort Randall Dam in 1952.

trend approximated the expected rate very closely until 1954. Comparative rating curve data at a U.S.G.S. measurement station 7 miles downstream from the dam indicated a major shift in 1952, due to the maximum flood of record, but only minor yearly changes since dam closure.

Frequent cross sectional resurveys were made at the degradation ranges below the dam to record volumetric changes. In order to evaluate properly the observed changes, however, the configuration of the river channel must also be considered. From the dam downstream to about river mile 919 the channel maintains a fairly uniform width of about 2,000 feet along a left bank bluff contact, while between river mile 919 and 915 it makes a diagonal valley crossing to another bluff contact on the opposite bank. Within this reach the channel widens to more than 3,000 feet, while the river meanders in two or more threads of flow between low sandbars. Below river mile 915 for the next 3 miles it again assumes a bluff contact while repeating another diagonal valley crossing between river mile 912 and 909.

Restricting our analysis to this upper 15 miles of the degradation reach, we record on figures 5 and 6 comparative cross sectional area, width, average bed, and thalweg changes as referenced to a common plane that paralleled the natural water surface profile and intersected most sections at about a bankfull stage. Thus, the data reflect changes occurring not only within the wetted perimeter but also to the bank lines of the channel and larger sandbar islands.

As might be expected under degrading conditions the channel widths tend to increase; again this widening occurred, particularly in the wide crossing reaches. Most of this apparent widen-

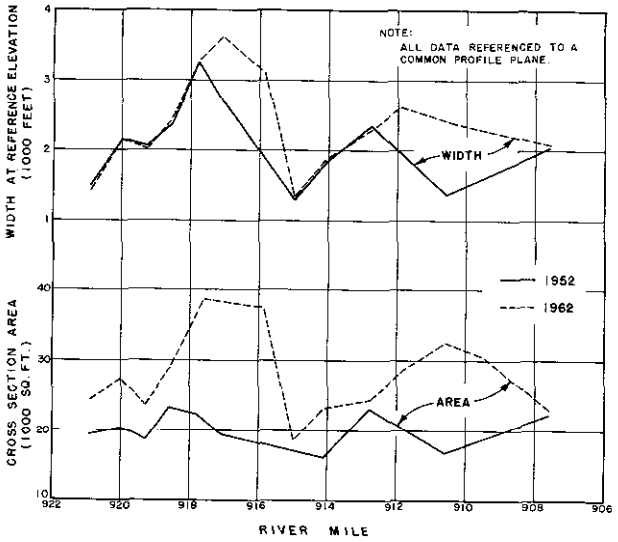


FIGURE 5. — Changes in cross sectional width and area as observed at degradation ranges between 1952 and 1962.

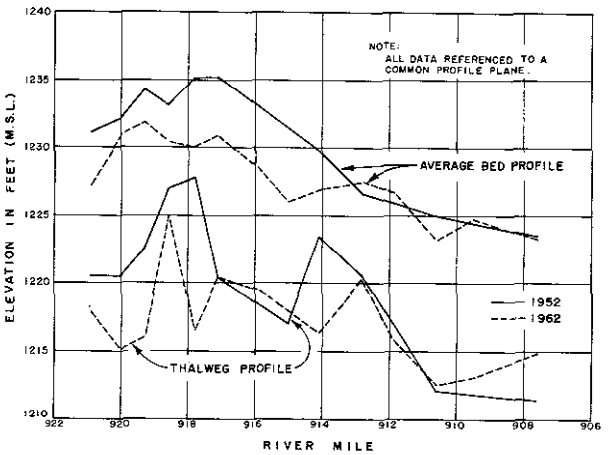


FIGURE 6. — Changes in average bed and channel thalweg elevations as observed at degradation ranges between 1952 and 1962.

ing, however, has been due to the disintegration of the channel sandbars; only a secondary amount of widening has been due to erosion of the high channel banks. For example, at river mile 917.1 the channel width increased 928 feet in the 10-year period, yet only one-third of this change was true bank erosion. The rest was loss of sandbars within the channel between the high banks. The flows attacked the bars in preference to the equally erodible channel banks. The plot of the cross section areas and the associated average bed profile also exhibit considerable changes, but again this represents predominately the influence of the reduction in sandbar area and not cutting of the high banks. In the lower crossing reach below river mile 912, the major change was due to extensive bank erosion; however, this was the result of a natural shift in the channel alinement that had been developing since the flood of 1952.

The volume of material that has been transported from this reach, as determined by end area computations, also reflects the trends noted in cross sectional area and width changes (fig. 7). Again, the major differences are noted in

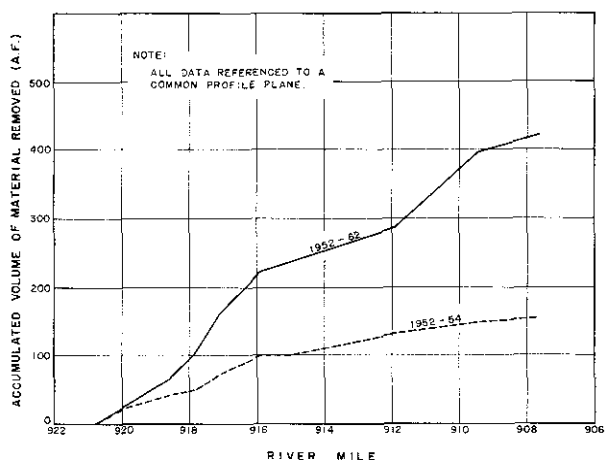


FIGURE 7. — Estimated volume of material removed by natural stream transport from the upper Fort Randall degradation reach between 1952 and 1962.

the channel crossing reaches where bed and bank materials are being introduced into active transport. Although the total volume removed approximates an average of 1,000 acre-feet per year, this value converted to an average daily sediment transport concentration represents less than 100 parts per million, as compared with a daily average of about 5,000 parts per million in the natural river prior to construction of the dam.

The changes in bed surface composition since 1951 are illustrated in figure 8, where the D_{90},

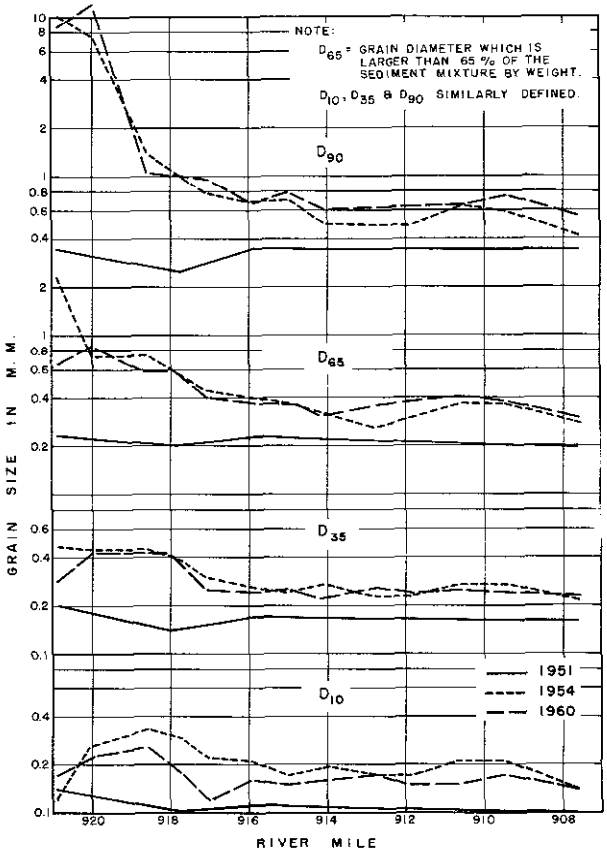


FIGURE 8. — Changes in the grain-size distribution of the streambed surface material between 1951 and 1960.

D_{65}, D_{35}, and D_{10} grain sizes are plotted against river mile for the years 1951, 1954, and 1960. There has been a significant coarsening of grain size throughout the reach from the dam downstream to river mile 907. For example, D_{65} was originally about 0.20 mm. In 1960 it was near 1.0 mm. at the dam, decreased progressively to about 0.35 mm. downstream to river mile 914, and remained fairly uniform at about this size below that point. D_{90} and D_{35} exhibit similar patterns. The uniform bed composition downstream of river mile 914, considered in conjunction with figure 6, which exhibits little change in bed elevations in this same reach, leads to the conclusion that this reach is temporarily in equilibrium, with the equilibrium load being supplied by degradation upstream of river mile 914. Degradation can be expected to progress downstream in years to come. Why there does not seem to be a significant change in bed surface composition throughout the reach between 1954 and 1960 is not readily explained. Perhaps such differences would be apparent if

the actual bed surface layer were sampled instead of the top 0.05 to 0.10 foot of the bed as discussed previously.

Armoring

The coarsening of the bed surface is associated with the phenomenon of armoring. As the sediment transporting capacity of the clear water released from Fort Randall Dam tended to become satisfied by scouring material from the riverbed, a continuous net lowering of the bed surface occurred, first near the dam, and progressing downstream. At the same time, due to the regulatory effect of Fort Randall Reservoir, both the incidence and the duration of high discharges were drastically reduced when releases were maintained in the 10,000 to 30,000 c.f.s. range after 1952. Curve 2 on figure 2, based on bed surface samples obtained under normal discharge conditions, indicates that few particles larger than about 1 millimeter are normally exposed at the bed surface. This would indicate there was normally little movement of grains larger than this size. Curve 1, based on the underlying bed material, however, indicates about 5 percent of the subsurface material is larger than 1 millimeter. In the natural river these larger grains were probably transported only during the annual flood periods when the discharge usually ranged above 100,000 c.f.s. The degradation trend, the regulation of flows to reduce the incidence of high discharges, and the presence in the bed of a fraction of material that is relatively nonmoving at normal discharges, were the elements necessary for armoring to occur.

The armoring process begins with a prolonged period of regulated flows during which the nonmoving particles segregate from the finer transport and are worked down into the bed where they accumulate in a sublayer at the lowest level to which the bed is turned over by the bar movement that accompanies the transport process. This lowest level is about in the plane of the troughs that occur just downstream of the crests of the moving bars. As bed material is transported from the reach below the dam, the entire bed system, the bars as well as the coarse sublayer, degrades. In this process fine bed material is leached up through the coarse sublayer to augment the material in transport. Bar movement continues, but as degradation progresses, an increasing number of nonmoving particles accumulate in the sublayer. Soon this accumulation interferes with the leaching process so that the rate of transport over the sublayer is not maintained at its former intensity. The moving bars begin to shorten with increasingly greater areas of the coarse sublayer exposed between them. Eventually enough coarse particles accumulate to shield, or "armor," the entire bed surface. Fines can no longer be leached from the underlying bed, transport in the reach is halted, and degradation is arrested. Figure 9 illustrates this armoring process in a flume study conducted at the University of California by A. S. Harrison.[1] A more dramatic illustration of the characteristics of armor layers has been presented in movies taken by L. G. Straub at the St. Anthony Falls Hydraulic Laboratory during experimental studies of gravel blankets for streambed stabilization for the Arkansas River project of the Corps of Engineers.[2]

This same armoring process — the gradual transition from continuous bar movements to the stable bed surface — developed below Fort Randall Dam; but observations were limited to sampling and soundings until the spring of 1962 when high inflow conditions on the lower river necessitated a lengthy period of zero releases from the dam. This circumstance allowed the channel to drain to a very low stage into a series of pools and permitted a visual inspection of a part of the streambed surface. The photographs shown as figures 10 through 14 represent typical channel bed conditions as observed through the armored reach, which extended several miles below the dam.

A general view of the bed surface and a typical examination hole is shown in figure 10. The characteristic shingled arrangement of the surface particles should be compared with the texture of the surface obtained by Mr. Harrison in his flume study (fig. 9). Figure 11 shows a closeup view of the cross section of an examination hole. Attention is directed here to the thickness of the armor layer and the composition of the underlying material, including an isolated particle of coarse gravel 6 inches below the surface. The large gravel particles, which appear so numerous in these photographs, were not detected from the original bed sampling and borings, probably for the reasons previously discussed.

The closeup views in figures 12 through 14 show in detail these important characteristics of the armor layer:

(1) That less than a single complete covering layer of larger gravel particles seems to suffice for a total armoring effect. Figure 12 shows a complete gradation of particle sizes throughout the gravel range.

(2) A natural "filter" seems to develop between the larger surface particles and the sub-

[1] HARRISON, A. S. REPORT ON SPECIAL INVESTIGATION OF BED SEDIMENT SEGREGATION IN A DEGRADING BED. Calif. Inst. Engin. Res., ser. 33, issue No. 1. September 1950.

[2] Presented at American Society of Civil Engineers Water Resources Engineering Conference at Omaha, Nebr., May 1962.

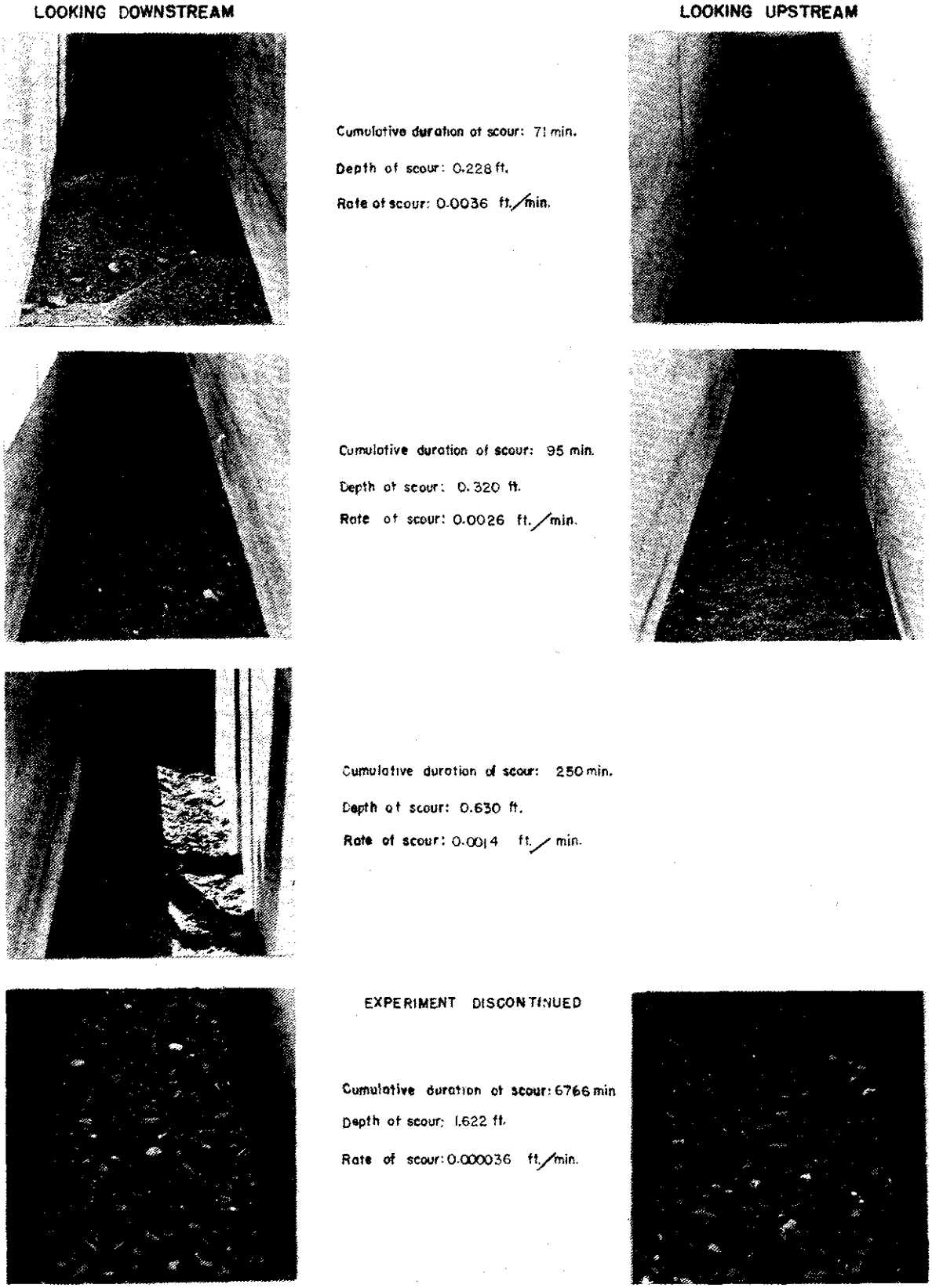


FIGURE 9. — Changes in bed characteristics during scour, as observed by A. S. Harrison in a flume study conducted at the University of California.

surface material to prevent leaching of the underlying fines. Figures 13 and 14 demonstrate that a surface layer of large particles



FIGURE 10. — General view of bed surface armor below Fort Randall Dam. Observations were made during extremely low stages in 1962. Reference scale in the examination hole has a 6-inch length.



FIGURE 11. — Cross sectional closeup of the examination hole shown in figure 10. Note armor thickness and composition of underlying materials, including an isolated particle of coarse gravel about 6 inches below the bed surface.



FIGURE 12. — Closeup view of typical bed surface armor. Note range of particle sizes.

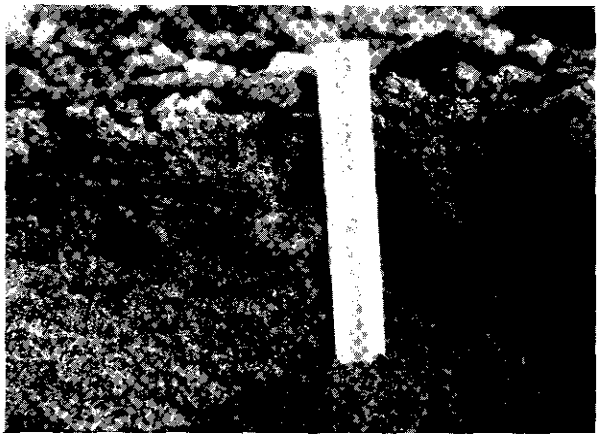


FIGURE 13. — Cross sectional closeup view of streambed armor, showing isolated example of a concentration of coarse gravel in the underlying strata.

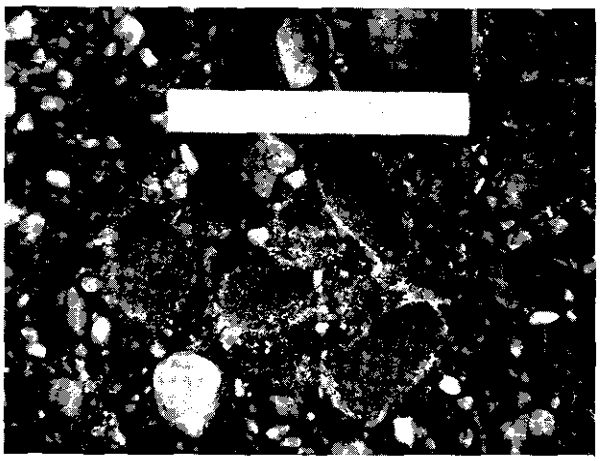


FIGURE 14. — Closeup view of bed surface armor, with several large gravel particles removed to show underlying sands.

does not have to be complete if medium particles are present to form a "filter" to fill in the interstices. The coarser sands collect beneath this surface.

(3) The shingled arrangement of surface particles is not restricted to the larger material but seems evident throughout the gravel gradation. This condition is very apparent in the arrangement of particles shown in figures 12 and 14.

If a single layer of material is sufficient to establish a bed armor, then armoring can develop quite readily. If a one-half inch particle is nonmoving and only 1 percent of the material underlying the bed surface is greater than one-half inch, then the depth necessary to accumulate a single surface layer of one-half inch particles would be 0.5 inch $\div$ 0.01 = 50 inches, or approximately 4 feet of degradation. Actually less degradation than this would suffice. First, the thickness of a single layer of alluvial gravel tends to be less than the nominal particle size, because the minor axes of the particles are smaller than the semimajor axes. The particles tend to lie with their minor axes vertical, although the semimajor axes determines the nominal sieve size of a particle. The second reason is that the bed need not be completely covered with armoring gravel if a filter layer of intermediate size particles is present between the armor layer and the underlying bed. For these reasons, it is probable that about a half layer of nonmoving particles is sufficient to armor the bed. This conclusion is supported by the results of the flume study cited above.

Conclusions

The rapid channel armoring which developed below Fort Randall Dam significantly arrested the desired degradation trend. It appears that three of the elements necessary for armoring to occur prevailed in this reach: (1) the degradation trend, (2) the regulation of flows to reduce the incidence of high discharges, and (3) the presence in the streambed of a fraction of material that is relatively nonmoving at normal discharges, plus a full gradation of material finer than this nonmoving particle size. Through visual inspection of the existing armor layer it becomes clear that less than a single layer of larger particles is sufficient for a total armoring effect, the armoring effect is particularly due to the natural filter that develops beneath the surface layer to prevent leaching of the finer underlying material. The shingling arrangement of the surface particles appear to reduce the maximum surface coverage necessary of the larger nonmoving particles. These characteristics of the bed armor below Fort Randall indicate that perhaps further degradation might be encouraged by disturbing this surface layer through artificial scarifying or jetting action.

STABILIZATION OF STREAMBEDS WITH SHEET PILING AND ROCK SILLS

[Paper No. 55]

By WALTER M. LINDER, *hydraulic engineer, U.S. Army Engineer District, Omaha*

Synopsis

The design of the Floyd River flood control project in Sioux City, Iowa, included a high velocity rock-lined channel that would carry the design discharge of 71,500 c.f.s. below the level of the adjacent natural ground. A high potential for erosion of the channel bed required the development of a series of channel stabilization structures that would maintain the desired water surface elevation and deter destructive bed degradation and upstream head cutting. A brief model study at the University of California at Berkeley indicated a row of sheet piling across the channel bed with some form of rock protection might adequately retard the development of head cutting and create sufficient losses to maintain the desired upstream water surface. Further model studies were conducted at the University of Iowa to develop criteria for adequate rock protection for the sheet piling and to obtain relationships for determining the head losses to be used for design purposes.

This paper summarizes the conditions that led to the development of the channel stabilization structure and describes the procedures followed in the model studies. The results of the model studies are discussed and presented in the form of curves suitable for general design purposes. Also suggested are some of the areas in which additional investigation would be profitable.

High Velocity Channel for the Floyd River Flood Control Project

The flood control project for the Floyd River consists of approximately 6 miles of realined and improved channel through the business and industrial area of Sioux City, Iowa. The design of the lower mile of the project was dictated to a great extent by severe limitations on the available right-of-way and by the numerous streets

and railroads traversing the area. Since the realined channel passes through an area occupied by numerous business and industrial establishments and extensive railroad yards, the right-of-way requirements had to be held at an absolute minimum. The numerous street and railroad crossings and adjacent railroad yards required that the water surface elevation be kept as low as possible in order to avoid the prohibitive cost of raising the railroad yards and adjacent street grades. Also the bed grade of the realined channel had to meet the bed grade of the existing natural channel at the head of the reach, and at the same time it could not be so steep as to produce excessive velocities for discharges up to 71,500 c.f.s. A general plan of the Floyd River flood control project is shown on figure 1.

Normally these stringent requirements would dictate the use of a high velocity concrete-lined channel; however, the cost of such a channel would have been prohibitive. The selected design calls for a trapezoidal channel with an erodible bed and rock protected 1 on 2½ side slopes. The bottom width of this channel narrows from approximately 280 feet at the Missouri River to about 100 feet at the upper end of the reach. Design water depth ranges from 17 to 25 feet from the lower to the upper ends of the reach, respectively. Average velocities through the reach will be slightly in excess of 14 ft./sec. for the design discharge of 71,500 c.f.s. Velocities will be greater than 10 ft./sec. for floods greater than 23,000 c.f.s., which is approximately the 25-year flood.

Throughout the rock-lined reach the channel lies in relatively erosion-resistant silts and clays of the Missouri River flood plain. However, a few feet below the design bed grade lie deep deposits of highly erodible sands and gravels. With the erodible material lying so close beneath the channel bottom there is a high potential for deep localized scour and extensive bed degradation with the occurrence of high velocity flows. In the event extensive degradation occurred, it could cause extensive damage by undermining the bridge piers and abutments and the toes of the revetted side slopes. Another consequence of degradation would be excessive drawdown of the water surface and destructive velocities in the leveed earth channel upstream of the rock-lined reach.

It was therefore necessary to develop a series of stabilization structures that would deter head cutting in the event degradation occurred and create sufficient head losses to maintain the desired water surface elevations. The preliminary design contemplated the driving of single rows of sheet piling across the channel at strategic locations with the tops of the sheet piling at or slightly below the design bed grade. At that time no specific information was available on the amount of head loss that could be expected once the sheet piling was exposed and projected into the flow. Also lacking was information as to its effectiveness in preventing head cutting and the amount of rock protection required to insure the stability of the sheet piling itself.

Model Studies at the University of California

At the time the high velocity channel for the Floyd River was being designed, model studies on rock sills for the Gering drain project were in progress at the hydraulic laboratories of the University of California at Berkeley. A brief two-dimensional study was set up to see what the behavior of a row of sheet piling across an erodible bed might be. The flume used for this study was a 60-foot long recirculating channel 2 feet wide and 1 foot deep. A piece of sheet metal across the channel and anchored to the flume bottom was used to represent the sheet piling. A 4-inch depth of small polystyrene pellets was used to represent an erodible bed. Since the 1.055 specific gravity of the pellets was only slightly greater than that of water, they moved at the slightest disturbance and served as an excellent indicator of the scour-inducing potential of the structure. No rock protection was used in the vicinity of the model structure.

The structure was tested at several different discharges with varying tailwater conditions and varying amounts of structure exposure above the average bed elevation. The results of the test indicated that sufficient losses could be generated over the sill, and that stability of the sheet piling did not appear to be a major problem. Based on the results of these studies, the initial design of a series of four sills for the Floyd River channel was completed. The sills consisted of a V-shaped row of sheet piling across the channel at a 10° angle with the normal and with the point of the V facing upstream. A 24-inch horizontal layer of rock was placed upstream of the sheet piling for a distance of 15 feet with the surface of the rock 3 feet below the top of the sheet piling. Downstream of the sheet piling a 36-inch layer of rock extended on a 1-on-2 slope from the top of the sheet piling to a depth of 9 feet below the top of the piling. The adjacent side slopes were protected with a 36-inch layer of rock, which tied into the horizontal blanket upstream and extended to a depth of 9 feet below the top of the piling on the downstream side of the structure. The original intention of the upstream facing V was to concentrate the flow over the sill into the center of the downstream channel and away from the

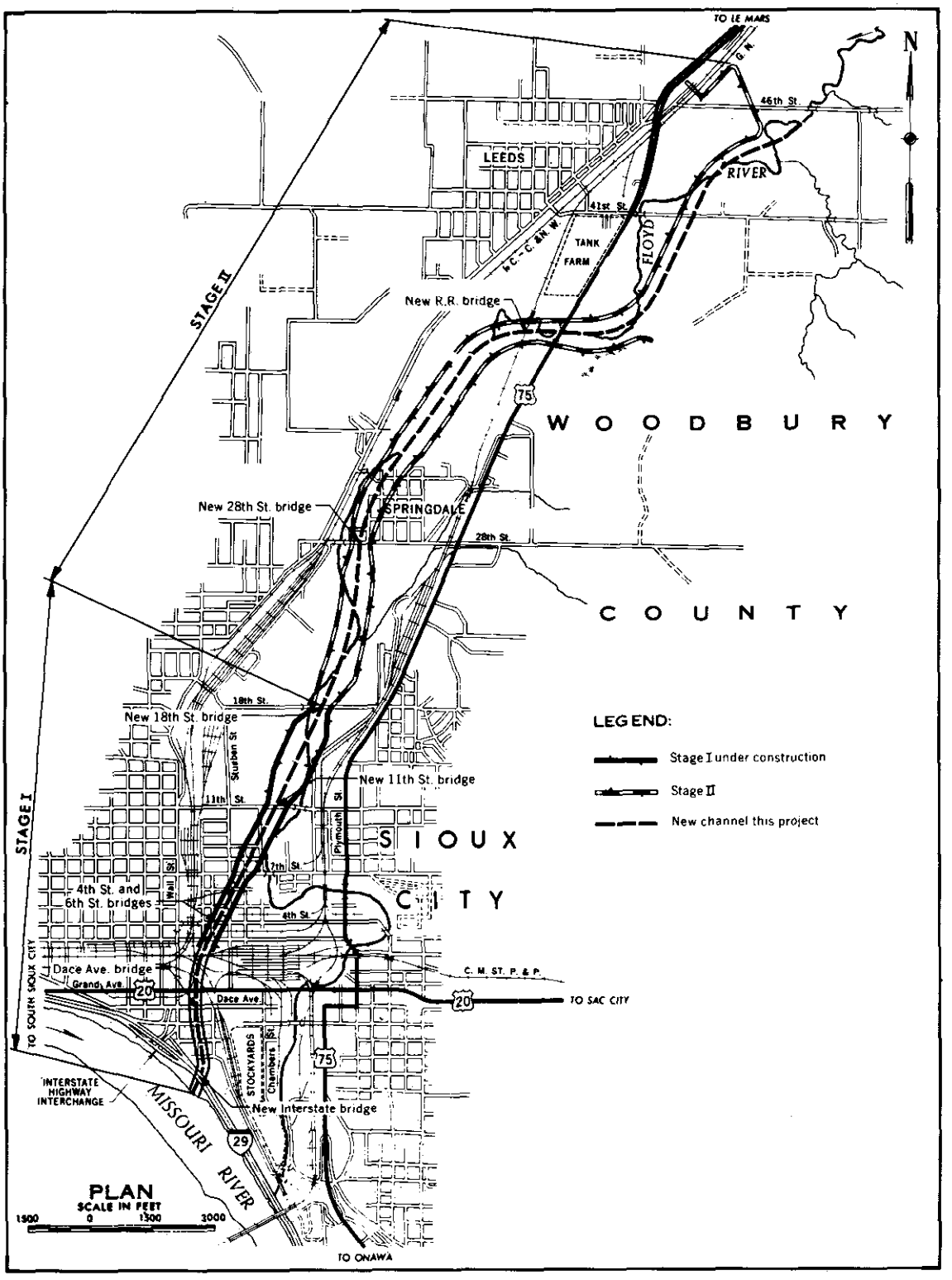


FIGURE 1. — General plan of the Floyd River Flood Control project.

side slopes, but model tests later showed that a straight transverse row of sheet piling gave better performance.

Model Studies at the University of Iowa

Objectives and Available Facilities

When the time came for the Iowa Natural Resources Council to give the required approval for the construction of the project, there was doubt expressed as to the ability of a series of sills to control channel erosion and prevent serious damage to the channel and related structures. Approval of the channel work was withheld until it could be demonstrated that the sills would perform as expected. In cooperation with the Iowa Natural Resources Council, an agreement was reached whereby personnel of the Omaha District, Corps of Engineers, would use the facilities of the hydraulics laboratory of the University of Iowa at Iowa City to construct a three-dimensional model of one of the proposed sills and demonstrate the performance of the structure. At the same time any deficiencies in the proposed design would be corrected; and, as time permitted, general design information would be obtained. Specific items of information desired were the minimum rock protection required, discharge coefficients, and the effect of submergence.

The flume used in these studies consisted of a concrete block channel 30 feet long, 5 feet wide, and 18 inches deep. The available discharge was approximately 5.25 c.f.s. Level rails on the walls of the flume permitted movement of a traveling carriage used for measurements of the water surface and the channel bed. The rails also served as guides for the screeds used in construction of the model channel. In addition, the flume was equipped with a mechanically driven sediment feed elevator at the upstream end and a combined sediment trap and weighing tank at the downstream end (figs. 2 and 3).

The 5-foot flume width permitted the use of a 1:50 scale ratio for the three-dimensional model. The relation between the model and prototype for this scale ratio, based on Froude's Law, are given in the following table:

	Prototype	*Model*
Channel bottom width...feet....	140	2.80
Channel length...feet.....	1,250	25
Design discharge...cubic feet per second	71,500	4.04
Average velocity...feet per second	14	1.98
Depth of flow...feet.....	25.3	0.505
Riprap, median size...inches in diameter	22	0.45
Derrick stone..inches in diameter	36 to 48	¾ to 1

Construction of the Model

The model bottom width of 2.80 ft. and depth of 0.505 ft. required a channel top width of 5.30 ft., or 0.30 foot wider than the flume. This resulted in a very slight reduction of the flow area. To avoid trouble with the intersection of the rough channel side slopes and the smooth flume walls, a strip of expanded metal screen was placed against the flume walls to provide

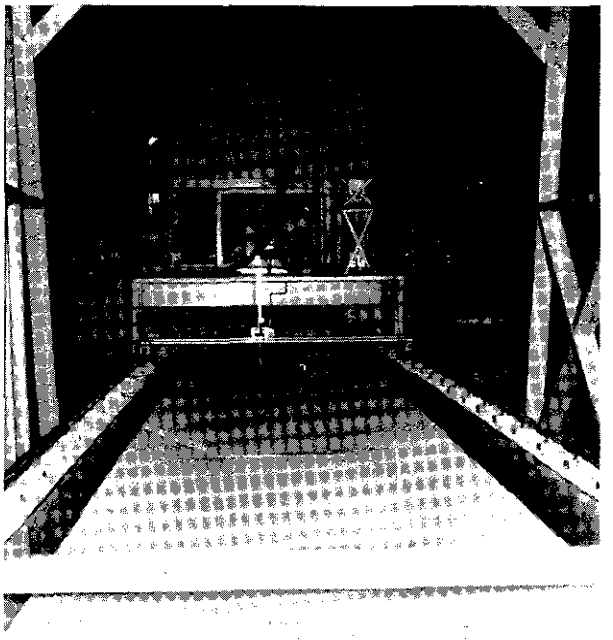


FIGURE 2. — Downstream view of the University of Iowa flume and the sand base for a model channel.

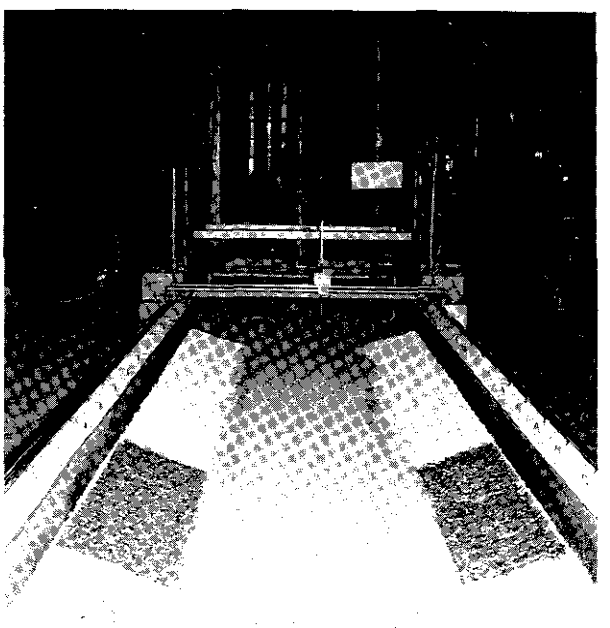


FIGURE 3. — Upstream view of the University of Iowa flume and the rock-lined model channel.

essentially the same degree of boundary roughness as the model riprap.

The material used to construct the channel base and represent the erodible bed was a uniform silica sand that had a median grain diameter of approximately 0.63 mm. A mechanical analysis of the sand is shown on figure 4.

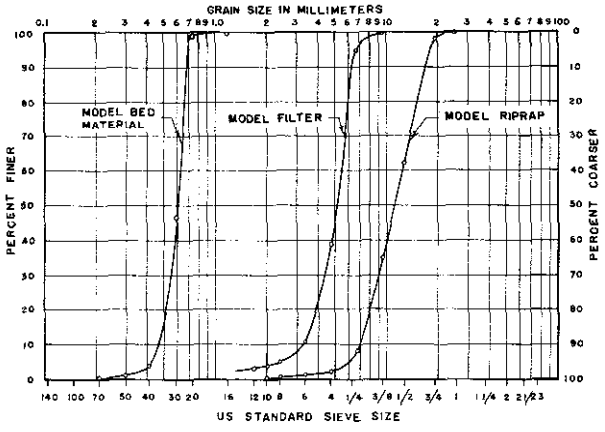


FIGURE 4. — Mechanical analysis of the sand and crushed rock used for model construction.

This sand may not have been the most suitable for the test, due to the excessively large dune formations and their possible effect on the test results. However, it was transported at a rate that gave a reasonable rate of degradation; and sufficient quantities of the sand were readily available in the laboratory.

The filter material and riprap were represented by crushed limestone obtained from a local quarry at Iowa City. Specific gravity of the limestone ranged from 2.713 for the filter to 2.708 for the riprap and derrick stone. Limestone chips with a median grain diameter of 5.25 mm., or 0.207 inches, formed the filter between the sand base and the model riprap. The model riprap was quarry-run crushed limestone that was hand-sieved to remove all material larger than ¾ inch in diameter. The resulting median size of the riprap was 0.446 inch, which is equivalent to a prototype diameter of approximately 22 inches, or a weight slightly under 600 pounds. Several other riprap gradations were used in test sections to determine the ability of a smaller rock size to withstand the expected velocities, but for the most part the riprap gradation described above was used throughout the test. Mechanical analyses of the filter and riprap are also shown on figure 4.

Two sizes of model derrick stone were obtained from the same crushed limestone. The larger would pass a 1-inch sieve and be retained on a ¾-inch sieve. This represented prototype sizes ranging from about 3 to 4 feet in diameter, or a range in weight from 2,600 to 6,400 pounds. The smaller derrick stone would pass a ¾-inch sieve and be retained on a ½-inch sieve. This corresponded to prototype sizes of 2 to 3 feet in diameter, or weights from 800 to 2,600 pounds.

The channel base was formed in the sand with a traveling plywood screed mounted on the flume side rails. A ⅜-inch layer of filter and slightly over an inch of riprap were placed with the aid of similar screeds. These rock layers were thicker than required by the prototype values, which would have scaled down to ⅛ inch for the filter and ½ inch for the riprap. The tedious and time consuming hand placement of such thin layers did not seem to be justified.

The sheet piling was represented by a heavy piece of sheet metal, which was either anchored to the flume bottom or in some instances pushed into the sand base and supported only by the surrounding sand. In all cases the top of the sheet metal was at the design bed grade. After placing the rock protection around the sheet metal and along the adjacent side slopes, the bed was filled with sand to the design grade and smoothed so that no part of the structure was exposed at the start of a test series. Before covering with sand, the surface of the rock in selected areas adjacent to the structure was lightly coated with spray enamel. This served as an indicator of damage to the rock protection, since the removal of even a single rock immediately showed as an unpainted area. During construction of the model, as-built cross sections of the channel and structure were measured for use in later analyses. Figures 5 and 6 show photographs of the sill in various stages of construction.

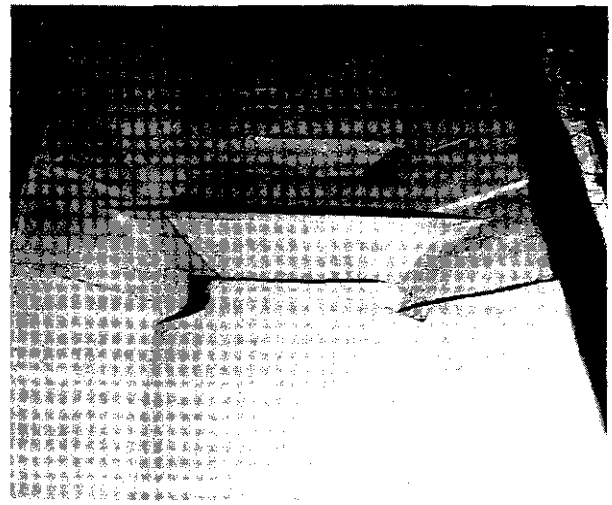


FIGURE 5. — Sand base for the model sill.

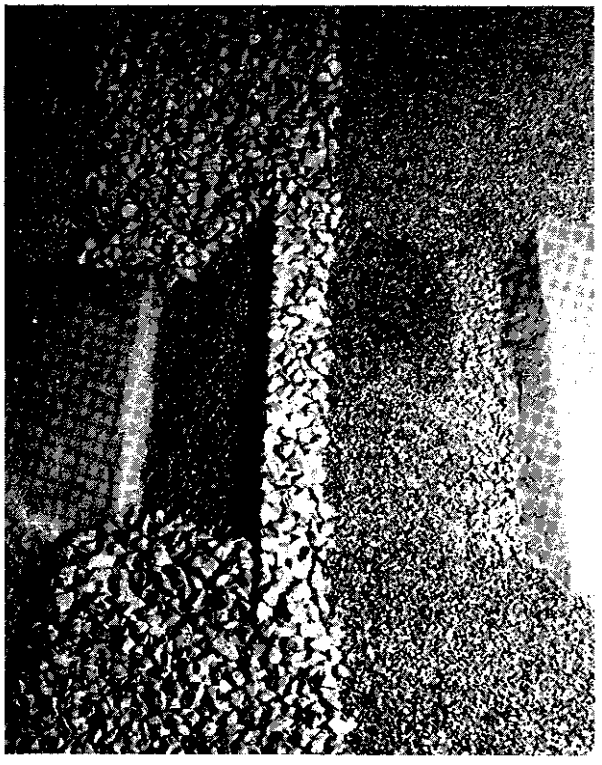


FIGURE 6. — Model sill after placement of filter and derrick stone. Flow will be from right to left.

Models Tested

Three separate series of tests were made in order to obtain general design information as well as develop a specific design for the Floyd River. They were as follows:

1. A 1:50 scale three-dimensional study.
2. A two-dimensional study that used the full flume width.
3. A one-half channel study that used one side slope from the three-dimensional study and extended the bed to the opposite wall of the flume.

The half-channel tests and the two-dimensional tests were run in order to extend the information obtained at Berkeley and aid in separating the end effects of the side slopes. Photographs of the two-dimensional, half-channel, and three-dimensional sills are shown on figures 7, 8, and 9.

Since the prototype sills would be spaced too far apart to interfere with each other, only one model still was installed in the flume for a series of tests. The installation of a single sill permitted the flow approaching the sill to become well stabilized and also left a sufficient distance for tailwater recovery downstream of the structure. Several sills in the flume would have interfered with each other to the extent that it would have been almost impossible to obtain the desired information.

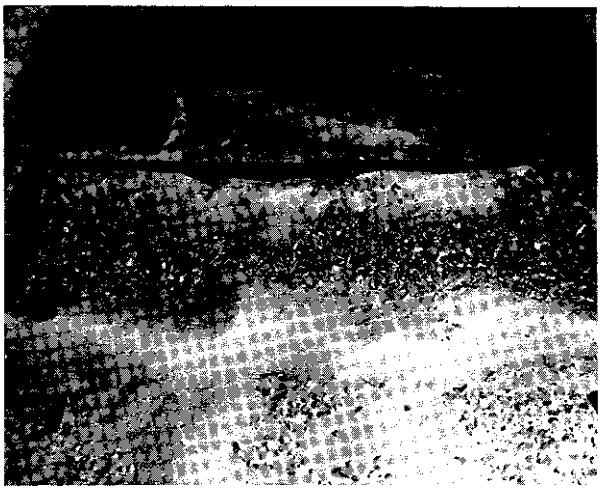


FIGURE 7. — Two-dimensional model sill; flow from top to bottom.



FIGURE 8. — Half-channel model sill; flow from top to bottom.

Preliminary three-dimensional tests were conducted with the V-shaped structure. However, it was soon apparent that this shape was not entirely satisfactory as the V tended to direct the flow against the banks just upstream and cause excessive scour at the point where the sheet piling entered the side slopes. As a result, the design was changed and all subsequent tests were made with a single row of sheet piling at right angles to the channel center line. This alinement minimized the tendency for attack on the side slopes upstream of the structure.



FIGURE 9. — Three-dimensional model sill; flow from top to bottom.

Test Procedure

The test procedure began with slowly back flooding the flume to avoid the damage from water flowing down a dry channel and pouring over the structure. The desired discharge was then established and the tailgate adjusted to give the desired tailwater conditions. If the test was started with the bed at the design grade, the bed was allowed to degrade until the sill was exposed. In any case, the discharge and bed configuration were allowed to become well stabilized before any profile measurements were attempted. After stable conditions were established two water surface profiles were measured with a traveling point gage. The water surface elevations were read at the third points of sections spaced a maximum of 2 feet along the flume and closer in the vicinity of the sill. During certain runs several velocity grids were measured with a pygmy current meter to see if the rough rock boundary of the channel was effective in keeping the high velocity filaments away from the channel side slopes. Visual observations were also made of the flow conditions downstream of the sill for various tailwater stages.

After measuring the water surface profiles, the flow was shut off and the flume allowed to drain. A sufficient number of bed elevations were then measured to determine the average bed profile. Maximum longitudinal spacing was 1 foot. The transverse spacing depended upon the width of the bed but was generally never greater than 1 foot. For use in determining the flow area, cross sections were periodically measured to supplement the bed profile measurements.

Throughout each run the scale recording the amount of sand caught in the trap was read to provide a record of the bed transport. When the sand trap was filled, it was emptied and the sand replaced in the bed and the sand feed elevator. At first an attempt was made to establish for each run an equilibrium bed transport in which the same rate of sand was being fed into the channel as was being trapped at the channel exit. When it became apparent that this would have no significant effect on the energy losses, the bed was allowed to degrade as the tests proceeded. Water surface profiles were thus obtained for varying amounts of sill exposure. Severe degradation downstream of the structure was permitted in order to determine the stability of the sill with large amounts of downstream erosion and also the ability of the sill to stabilize the upstream bed grade.

Development of the Rock Protection

Determination of the amount of rock protection required for the sheet piling was carried on concurrently with the measurement of head losses over the structure. At first the upstream riprap blanket was placed 5 feet below the top of the sheet piling and extended horizontally upstream for a distance of 50 feet. After several runs, profiles over the sill showed that rock from the surface of the blanket was being carried away. The remaining rock assumed a stable slope of 1 on 5 to 1 on 6 in the upstream direction. The upstream blanket was then revised to slope in the upstream direction at 1 on 6 with the horizontal. It was also found that derrick stone was required in this blanket for the first 12 feet upstream of the sheet piling. Derrick stone was initially placed on the entire side slope from 50 feet upstream to 50 feet downstream of the sheet piling.

As testing progressed, it was found that very little derrick stone was required on the side slopes above the sill crest or upstream of the piling, but additional derrick stone was required further downstream. From 12 feet upstream of the sheet piling to 50 feet downstream of the piling, derrick stone was required below an elevation 5 feet above the sill crest. From 50 feet downstream to the end of the derrick stone 100 feet downstream of the piling, the top of the derrick stone was sloped from 5 feet above the crest to 5 feet below the crest. The bottom of the derrick stone on the downstream side slopes was carried to 16-feet below the design bed. On the 1-on-2 apron downstream of the sill the original rock riprap was found to be satisfactory, except for a narrow strip along the intersection with the side slope. Here was found to be the area of greatest stress on the rock protection, and the side slope derrick stone was extended onto the downstream apron in this area. A plan view of

the final design of the sill is shown on figure 10, and sectional views are shown on figure 11.

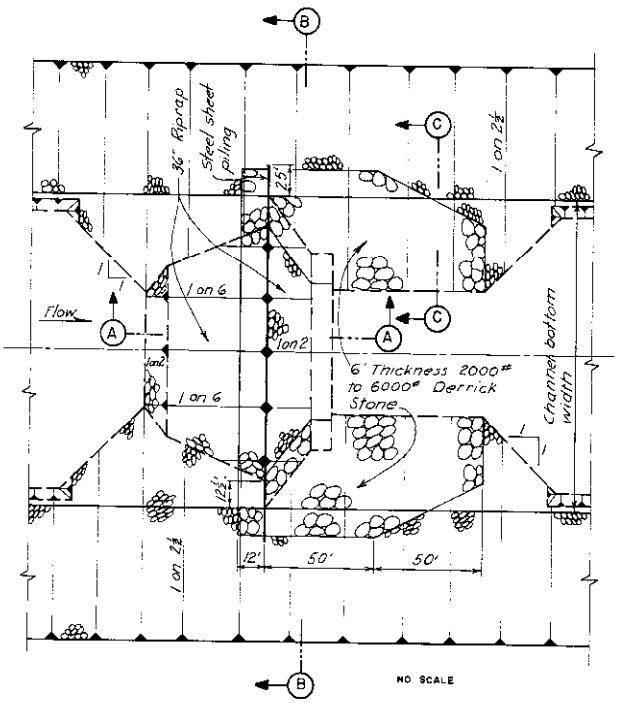


FIGURE 10. — Plan view of the final sill design.

Flow Condition Created by the Structure

The flow conditions in the vicinity of the sill are to a great extent dependent upon the tailwater stage. Upstream of the sheet piling the surface of the rock blanket was generally swept clean of sand and exposed for all tailwater stages. This was due to the turbulence caused by the discontinuity in the bed created by the projecting sheet piling. Downstream of the structure the flow pattern ranged from only minor surface disturbances to a well-developed hydraulic jump. Observations during the tests indicated that for ratios of the tailwater depth to the critical depth, T'/D_c, greater than approximately 1.65, the disturbance was minor with only a few ripples and undulations in the water surface. For lower tailwater stages where T'/D_c lay between 1.65 and 1.45, large standing waves and undulations developed downstream of the structure for a distance of several hundred feet. When the ratio of T'/D_c fell to about 1.45, a definite surface roller developed which became a well-established hydraulic jump for lower tailwater stages.

If the tailwater depth did not drop much below critical, the high velocity jet tended to float on the surface and did not attack the downstream bed. A reverse roller existed under the high velocity jet and actually deposited material back against the downstream face of the structure. Figure 9 shows this deposit of stand remaining while all the rest of the bed material

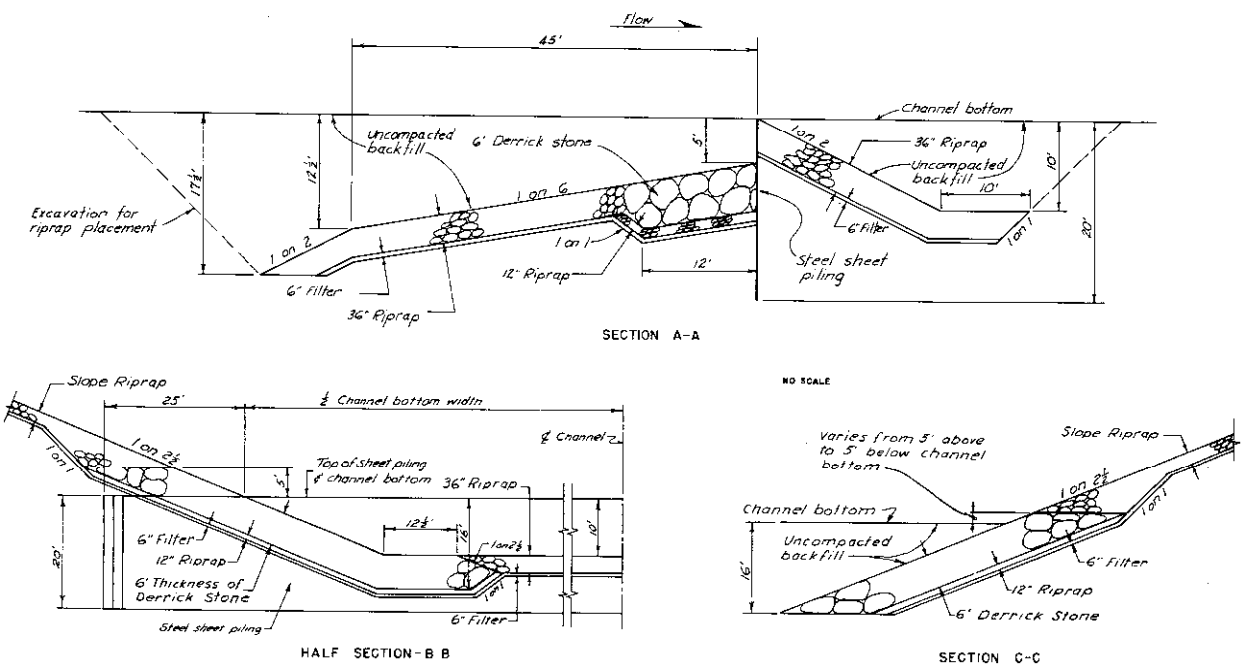


FIGURE 11. — Sectional views of the final sill design.

farther downstream has been carried away. If the tailwater dropped significantly below critical, the plunging nappe rapidly scoured a hole that undermined the rock on the downstream apron.

The turbulence near the water surface downstream of the structure tends to be confined to the central part of the channel by the lateral flow at each end of the crest. This lateral flow develops from the narrower water surface width downstream of the structure created by the change in water surface elevations in the trapezoidal channel. Figure 12 shows the downstream disturbance resulting from the design discharge and nearly 5 feet of drop in water surface over the structure; the turbulence is confined to the center of the channel and well away from the side slopes.

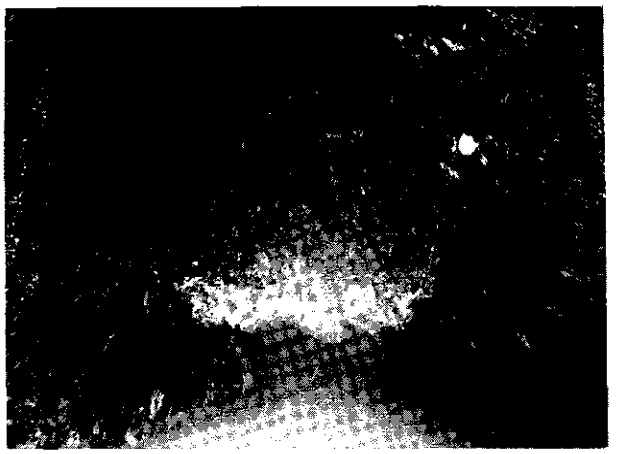


FIGURE 12. — Downstream surface turbulence resulting from the design discharge over the three-dimensional model.

Analysis of Data

The average water surface elevation and bed elevation measured at each cross section were plotted to obtain water surface and bed profiles. Upstream of the sill the best straight lines through the plotted bed and water surface profiles were determined by the method of least squares. The upstream energy grade line was then computed, based on the straight line bed and water surface. This energy grade line was then projected to the sill crest. Downstream of the structure it was generally impossible to use the least squares method to make a straight line determination of the bed or water surface due to the recovery zone of the water surface and the scour hole that developed in the bed. Instead the energy gradient was computed on the actual measured bed and water surface at each cross section. It was then usually possible to fit a straight line to the energy grade line computed in this manner. This downstream energy grade line was then projected to the sill crest. The flow area in a section was usually obtained by assuming a trapezoidal channel with the as-built side slopes. If it appeared the cross sections deviated significantly from this assumed geometry, the flow area was based on representative cross sections actually measured after each run. Figure 13 shows typical plotted profiles of the bed and water surface and the lines of best fit for the profiles.

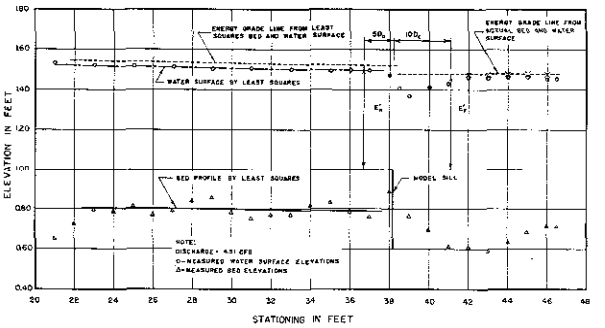


FIGURE 13. — Typical bed and water surface profiles from the University of Iowa tests.

The plotted profiles and the additional data collected during the testing gave the information required for analysis of the data. A brief definition of the symbols used in the following discussions and on the design curves is given below.

E_H = Headwater energy above the crest elevation based on a projection of the upstream energy grade line to the crest.

E_T = Tailwater energy above the crest elevation based on a projection of the downstream energy line to the crest.

E'_H = Headwater energy above the crest elevation at 5 critical depths upstream of the sheet piling.

E'_T = Tailwater energy above the crest elevation at 10 critical depths downstream of the sheet piling.

D_C = The critical depth in the cross section at the sill crest.

E_C = Critical energy in the cross section at the sill crest.

H' = Headwater depth of flow above the crest elevation at 5 critical depths upstream of the sheet piling.

T' = Tailwater depth of flow above the crest elevation of 10 critical depths downstream of the sheet piling.

P = Projection of the sill above the average upstream bed as determined by projecting the straight line representing the bed profile downstream to the sheet piling.

Q = Total discharge in the channel.

Q_o = Apparent discharge in the triangular-shaped end sections of the trapezoidal and half-channel.

$Q_\sqcup$ = Apparent discharge in the central portion of the trapezoidal and half-channel.
b = The crest width.
A = The total flow area between the crest elevation and the water surface elevation 5 critical depths upstream of the sheet piling.
A_v = The area in the triangular-shaped end sections between the crest elevation and the water surface elevation 5 critical depths upstream of the piling.
C = Coefficient of discharge for the total flow area.
C_v = Coefficient of discharge for the triangular end sections.
g = The gravitational constant
D_{15} = The grain diameter at which 15 percent, by weight, of the material is finer.
D_{50} = The median grain diameter.
D_{85} = The grain diameter at which 85 percent, by weight, of the material is finer.

The first attempt at analysis of the data was to make a dimensionless plot of E_H/D_C versus E_T/D_C for all the profile data. Examination of the plot immediately indicated separate curves for the two-dimensional channel data and for each discharge in the trapezoidal and half-channel. It was therefore necessary to devise some means of separating the effects of the triangular end sections from the essentially two-dimensional flow in the central part of the channel. This was accomplished by plotting the two-dimensional data in the dimensionless form of E_C/E'_H versus E'_T/E'_H. These points seemed to define a single curve, shown on figure 14, which is assumed to be applicable for wide rectangular crests. The apparent flow through the triangular end sections in the trapezoidal and half-channel tests was then computed by assuming the curve on figure 14 defines $Q_\sqcup$, the flow over the horizontal part of the crest. From the value of E'_T/E'_H measured during the test, the value of E_C/E'_H was determined from figure 14. E_C was then computed, based on the value of E'_H measured during the test. The corresponding D_C for a horizontal crest was then computed from $D_C = 2/3\ E_C$, and the apparent discharge over the horizontal crest was computed from $Q_\sqcup = bq = b\sqrt{gD_C^3}$. By subtraction the apparent discharge through the end sections became $Q_v = Q - Q_\sqcup$. An orifice-type coefficient for the flow in the end sections was computed from $C_v = Q_v/A_v\sqrt{2g}\ (E'_H - E'_T)$. The values of C_v thus obtained were plotted against E'_T/E'_H as shown on figure 15. Although there is considerable scatter, a relationship for estimating the flow through the end sections seems to be defined in figure 15, and it could be used in conjunction with figure 14 to compute head-discharge relation for the sills in this study.

The analysis just discussed, however, leaves something to be desired, considering the uncertainty suggested by the scatter in figure 15.

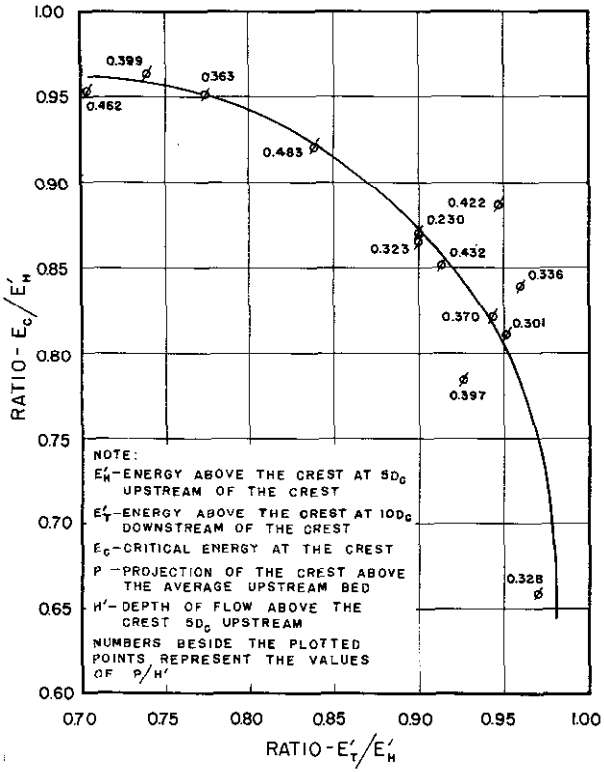


FIGURE 14. — Energy losses for a two-dimensional sill.

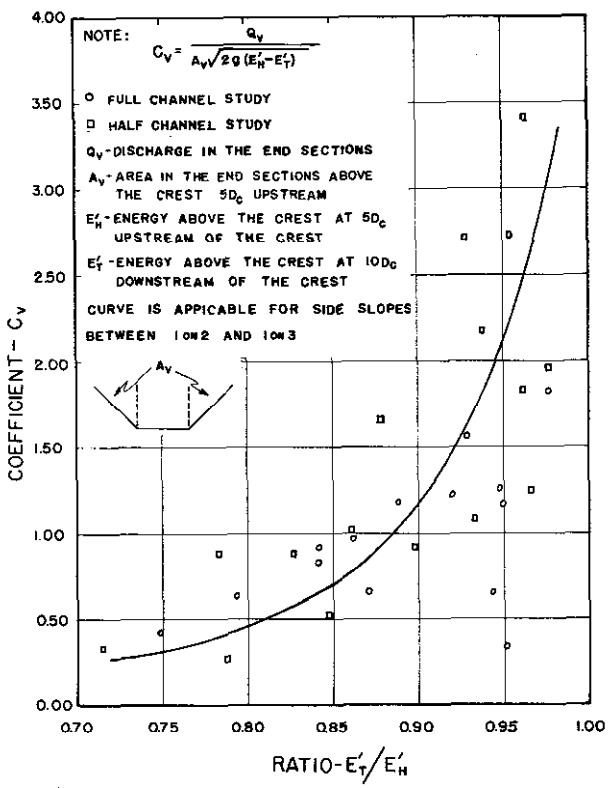


FIGURE 15. — Discharge coefficient for the end sections of a sill in a trapezoidal channel.

This method also involves a trial and error computation to determine the energy loss over a sill, if the normally known values of discharge and tailwater stage are given. Therefore, further analysis of the data was made to see if a more consistent relationship might be found and the computation for the upstream water surface simplified. Figure 16 shows a relationship between a total discharge coefficient defined from $C=Q/A\sqrt{2g\,E'_H}$ and the dimensionless submergence ratio T'/D_C, where D_C is the critical depth in the trapezoidal cross section at the sill crest. The number beside each plotted point represents the ratio of the area in the end sections to the total area. Two enveloping curves have been drawn through the plotted points. The upper curve is for the case of a rectangular channel where A_v/A equals zero. The lower is the estimate curve for A_v/A equals 0.30, which is the case where the area in the end sections is approximately one-third the total flow area. Of many possible combinations of data, this was the only one in which the two-dimensional data defined a boundary or limit and the data from the half-channel and trapezoidal channel fell in a position with respect to the relative amount of the area in the end sections. If the tailwater stage and discharge are known and an estimate can be made of the value of A_v/A, the upstream energy can be computed once the corresponding value of C is found from the plot. Figure 16 has been adopted for design purposes.

The scatter and lack of clear definition in figures 14 and 15 may be due to a number of reasons. One is the error in measuring the water surface. For high discharges and high degrees of submergence, the water surface was sufficiently rough to make the error in determining the water surface approach the magnitude of the losses being measured. The convention used in evaluating the discharge in the end sections leaves much to be desired when one assumes that the center part of the sill behaves as a two-dimensional sill with no influence from the end sections. The vertical walls at the edges of the channel may have had some influence in creating higher upstream stages over what would have been experienced with a true trapezoidal channel. The relatively large dune formations on the movable bed upstream of the sill undoubtedly influenced the tests results to some extent, as the dune height approached 20

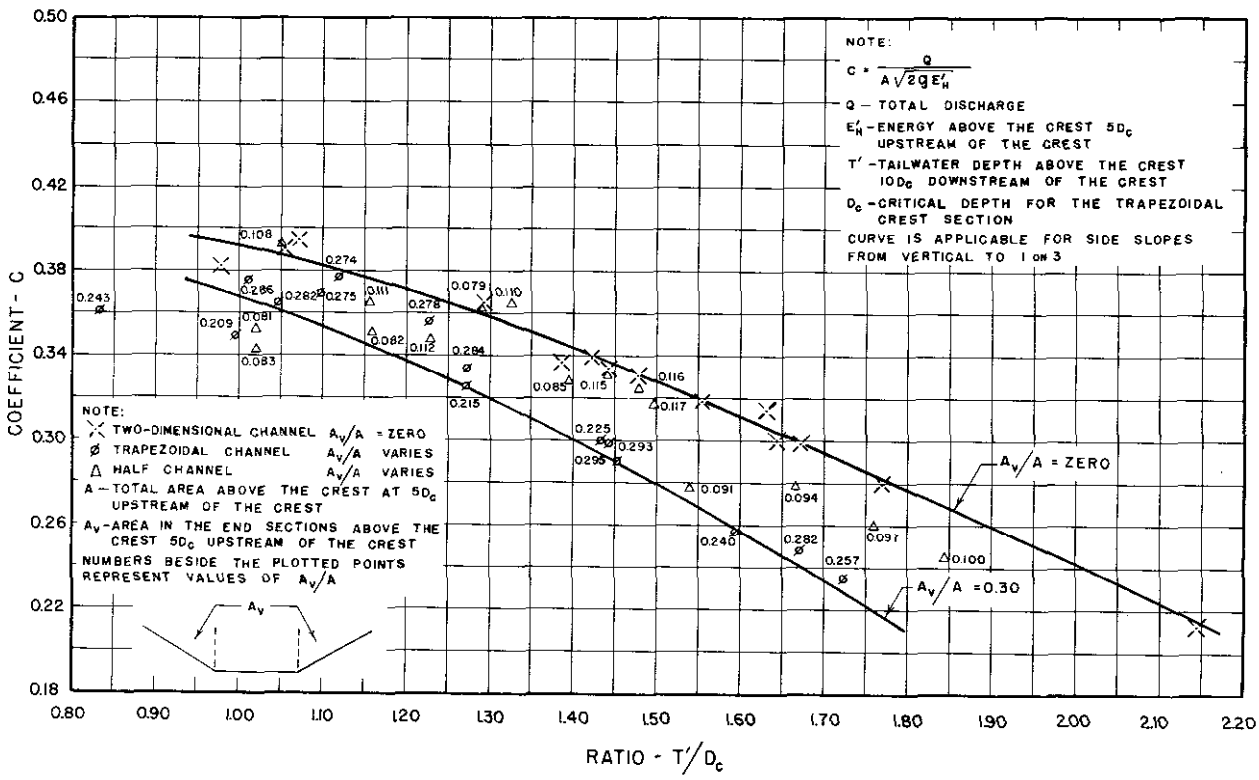


FIGURE 16. — Total discharge coefficient for a sheet piling and rock sill.

to 25 percent of the water depth in many cases. The effect of a dune temporarily burying the sill should not be reflected in the data, as water surface profiles were generally not measured while this condition was observed.

No definite relation between the degree of exposure of the structure above the bed and the amount of loss over the sill is indicated by the data obtained in the tests. On figure 14, where the value of P/H' is shown beside each plotted point, there is no apparent correlation between the values of P/H' and their location with respect to the curve. This may be due to the nearly constant degree of local exposure of the piling. Unless the sill is temporarily buried by a passing dune, the local turbulence created by the projecting piling will keep the upstream rock blanket swept clean of material. Even though the general bed elevation of the upstream channel may vary considerably from the elevation of the surface of the blanket, it will form a streamlined approach to the surface of the rock. The approach losses become only a small part of the total energy loss, since most of the loss is created by the local turbulence at the piling. For any given discharge, the loss due to this turbulence will remain nearly constant as the rock blanket places a limit on the amount of piling exposure that can take place.

Application of the Structure and Suggested Design Criteria

The application of the structure is limited in that it must essentially operate as a submerged sill where the high velocity jet will tend to float on the surface rather than plunge to the bottom. This requirement can be met if the tailwater does fall below approximately 0.8 of the critical depth at the crest section. If the tailwater is lower, the nappe will plunge and rapidly destroy the structure either by direct attack on the downstream slope or by creating a scour hole that will undermine the structure. The only time this tailwater criteria may be violated is for relatively low base flows, in which case adequate rock sizes on the downstream slope must be provided. If the above tailwater criteria can be met, the structure has many possible applications where it is desired to control water surface elevations and retard head cutting. The structure will not act as a barrier to bed transport except under low flow conditions, and it will not directly protect the upstream channel from scour. Erosion upstream of the structure will be controlled primarily through reduced velocities created by the increase in water surface elevations induced by the sill as it becomes exposed. Controlling the erodible bed of a high velocity channel and controlling bed and bank erosion that threatens damage to bridges or structures adjacent to the channel are examples of situations where the sill would be suitable. A decided advantage of the structure is the flexibility of the rock to adjust to conditions that would destroy a conventional concrete structure. If tailwater conditions prevent the use of a single sill, it may be possible to meet the tailwater criteria and raise the water surface to the desired elevation by the use of a series of sills.

Design of the structure starts with a determination of the tailwater rating curve and then proceeds with the selection of a crest elevation that satisfies the condition that T'/D_C must be greater than 0.80. The crest width should be essentially the same as the width of the upstream and downstream channel in order to avoid the inherent difficulties with a constriction where damaging lateral eddies will develop downstream. The headwater rating curve may be determined either with the curves on figures 14 and 15 or with the curves on figure 16. The curves on figures 14 and 15 should not be used for side slopes steeper than 1 on 2 or flatter than 1 on 3. If a reasonable estimate of the ratio A_o/A can be made, the curves on figure 16 will be simpler to use and can probably be applied to any side slope from vertical to about 1 on 3. If the headwater rating curve indicates that a single structure does not create the required head losses, it will be necessary to add additional sills until the desired water surface elevation is attained.

The type of sheet piling and the depth to which it must be driven will depend more upon construction conditions than hydraulic requirements. Since one or both sides of the piling will be exposed to considerable depth during construction, the piling should be designed to withstand the load created by a saturated fill on one side and exposure to the depth required by construction on the other side. It should also be capable of withstanding the impact of large rocks being placed against it. The sheet piling should extend into the side slopes far enough so that it will not be outflanked if a considerable amount of rock loss or adjustment takes place at the ends of the sill.

If the sill is intended for use in a channel where no bank revetment is contemplated, the minimum amount of channel side slope protection should extend from the upper end of the upstream blanket to the end of the derrick stone downstream of the sheet piling. Upstream of the piling the side slope protection should tie into the top upstream blanket. Downstream of the piling it should tie into the top of the derrick stone. It is suggested that the determination of the required rock size for the side slopes be

based upon the following relationship developed by Isbash.[1]

$$V = Y\sqrt{\frac{2g\ (S-W)}{w}}\ \sqrt{D}\ \sqrt{\text{Cos }\alpha}$$

Where V = velocity against the stone,
Y = numerical constant with a range in value from 0.86 to 1.20,
S = specific weight of the rock,
W = specific weight of water,
D = diameter in feet of a sphere of equivalent weight,
α = the angle between the horizontal and the bed upon which the rock lies.

For the rock on the channel side slopes, a value of 1.20 for Y would be appropriate, since the velocity distribution in the channel is well developed. For more severe conditions when the flow impinges directly against the rock, the more conservative value of 0.86 for Y should be used. The value for Cos α should be taken from the angle between the side slopes and the horizontal.

The surface of the upstream blanket should be from 3 to 5 feet below the top of the sheet piling and should be placed on a 1-on-5 to 1-on-6 slope in the upstream direction. It should extend upstream from 1½ to 2 times the expected head on the structure and should terminate in some form of toe to prevent raveling and undermining of the upstream edge. Except for the area immediately upstream of the sheet piling, the same rock size gradation as used on the channel side slopes will be adequate.

The downstream apron of the structure requires rock protection from the top of the sheet piling to ¾ to 1 critical depth below the top of the piling. This rock should be placed on a slope no flatter than 1 on 2, since flatter slopes may become subject to attack from the high velocity nappe. A short horizontal apron at the bottom of the 1-on-2 slope may be desirable to aid in turning high velocity filaments and directing them parallel to the bed. Except for the corners where the downstream apron and side slopes intersect, extra heavy rock protection is not required on the downstream apron. However, the rock should be of sufficient size to withstand the low flows that may plunge onto it. The previously discussed relationship developed by Ishbash should be used for determining the required rock size in this area. Since the low flows may plunge directly onto the rock, a value of 0.86 for Y would be appropriate. This value of Y will give essentially the same rock size as suggested for similar conditions by the Bureau of Reclamation in their Hydraulic Laboratory Report Hyd–409.[2]

Three areas in the vicinity of the structure are subject to above normal stress from the flow conditions created by the sill, and heavy rock protection is required in the form of derrick stone. These areas are (1) just upstream of the sheet piling, (2) in the corners where the downstream apron and channel side slopes intersect, (3) on the lower side slopes downstream of the crest. The plot on figure 17, which was developed from observations made during the study, shows a relationship between the ratio of tailwater to headwater, T'/H', and the required median diameter of derrick stone. The plotted points for the three rock sizes represent conditions varying from loss of a small amount of rock to essentially complete failure where significant loss of rock and severe slumping of the side slopes at the end of the crest took place. The lower solid curve represents the minimum derrick stone size that should be considered, and the upper dashed curve is the one suggested for design purposes.

The derrick stone on the upstream side of the structure should extend upstream of the piling for a distance of two to three times the vertical exposure of the sheet piling. It should also extend up the adjacent side slopes to about one-third critical depth above the sill crest. Since the stress in this area is not the most severe, the median derrick stone size may be determined from the lower of the two curves on figure 17.

The corners downstream of the sheet piling are subject to severe stress, and it is suggested that the size of derrick stone for this area be determined by the upper curve on figure 17. This heavy derrick stone should extend onto the downstream apron for a distance of 10 to 15 feet. On the lower side slopes it should be carried downstream three to four critical depths from the sheet piling. The lower side slope derrick stone should start about one-third critical depth above the sill crest and should be extended downward to an intersection with the downstream apron. If it is expected that the structure will be subjected to flows of relatively long duration or frequent high discharges with relatively low tailwater stages, it may be desirable to extend the derrick stone to six to eight critical depths downstream of the sheet piling. The rock size for this additional protection could be determined from the lower curve on figure 17.

Figure 10 shows the area in which derrick stone was required for the Floyd River sills. In order to reduce the number of rock sizes re-

[1] ISBASH, S. V. CONSTRUCTION OF DAMS BY DUMPING STONES INTO FLOWING WATER. Sci. Res. Inst. Hydrotech. Leningrad. 1932. Trans. by A. Dovjikov, U.S. Army Corps of Engineers, Eastport District, September 1935.

[2] U.S. DEPARTMENT OF THE INTERIOR. STILLING BASIN PERFORMANCE STUDIES, AN AID IN DETERMINING RIPRAP SIZES. Hydraulic Lab. Rpt. Hyd-409, U.S. Bur. of Reclam. February 1956.

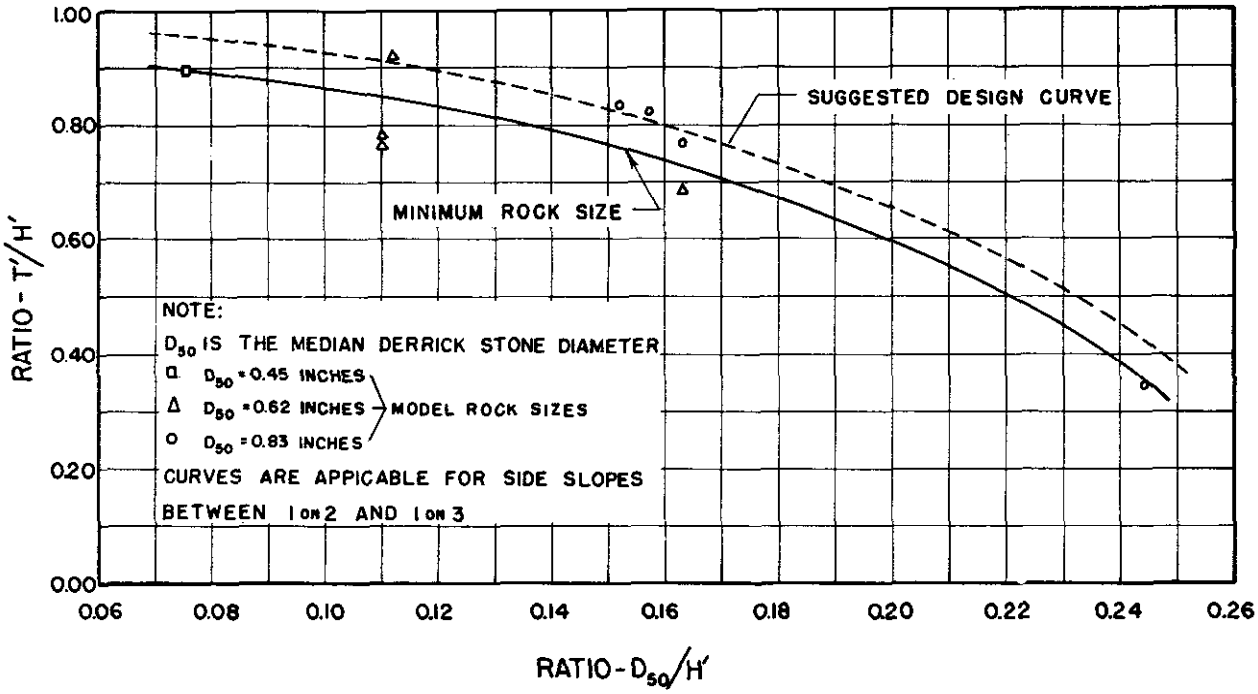


FIGURE 17. Derrick stone sizes required for sill protection.

quired, the same gradation of derrick stone was used throughout the structure. If it should prove economical to specify several gradations of derrick stone, the area may be zoned in the manner illustrated in the preceding paragraphs.

The recommended layer thickness for all of the rock in the vicinity of the structure is two median rock diameters, and in no case should it be less than 1½ times the median rock diameter. It is also very important that an adequate filter be placed between the rock and the bed material. The corners downstream of the piling are subject to severe piping of bed material through the voids in the surface rock. In runs with low tailwater stages, the limestone chips used for the filter material could often be observed to literally stream through the voids in the model derrick stone. This is indication of the very strong lift forces that exist in this area. It is recommended that the relation developed by Terzaghi in conjunction with the Corps of Engineers[3] be used for the design of the filter. This relationship is as follows:

$$\frac{D_{15}\ \text{Filter}}{D_{85}\ \text{Base}} < 5$$

$$4 < \frac{D_{15}\ \text{Filter}}{D_{15}\ \text{Base}} < 20$$

$$\frac{D_{50}\ \text{Filter}}{D_{50}\ \text{Base}} < 25$$

If the surface rock is derrick stone, several layers of filter material may be required and the above conditions must hold for successive layers if an adequate filter is to be provided.

Areas for Additional Investigation

Although the study developed an adequate design for the Floyd River, additional investigation in several areas would be desirable. Additional information is needed on the minimum rock size required in the various areas of the rock protection for the sill. Stability of the rock is a function of factors other than the velocity of the flow. For example, in the corners downstream of the crest damages to the rock blanket often resulted from the individual rocks being bodily picked off the surface and carried away by the flow rather than being merely rolled down the slope.

Another interesting aspect is the effect of

[3] POSEY, C. J. SOME BASIC REQUIREMENTS FOR PROTECTION AGAINST EROSION. Minn. Internatl. Hydraul. Convention Proc. 1953, pp. 85-87.

TERZAGHI, KARL, and PECK, RALPH B. SOIL MECHANICS IN ENGINEERING PRACTICE. John Wiley and Sons, Inc., New York. 1948.

U.S. WATERWAYS EXPERIMENT STATION. INVESTIGATIONS OF FILTER REQUIREMENTS FOR UNDERDRAINS, FIELD AND LABORATORY INVESTIGATION OF DESIGN CRITERIA FOR DRAINAGE WELLS, and LABORATORY INVESTIGATION OF FILTERS FOR ENID AND GRANADA DAMS. U.S. Waterways Expt. Sta. Tech. Memo. 183-1, 195-1, and 3-245, respectively.

sediment transport on the flow regime over the structure. Whether this is due to streamlining of the structure or a change in fluid properties cannot be determined by the results of these tests. In the Berkeley tests the extremely heavy bedload seemed to reduce the energy losses compared to a sill with no bed transport. The tests at Iowa City generally did not have a heavy rate of bed transport. However, it was observed that when a dune was passing over the structure, the turbulence downstream of the sill was reduced and the headwater seemed to lower slightly.

Conclusions

The study successfully demonstrated the ability of the structure to perform as desired, and approval for construction of the high velocity channel was granted by the Iowa Natural Resources Council. Computations based on the test results showed that five sills spaced approximately 2,000 feet apart in the lower 1½ miles of the Floyd River would effectively maintain the water levels in the channel despite considerable bed scour. Computations showed that even if the bed was to degrade 10 feet, the water surface would be maintained within 1 or 2 feet of the water surface for no degradation. The structure can provide an economical solution to problems of a similar nature where it is desired to control bed degradation and the water surface elevations, provided the tailwater conditions will allow the sill to essentially operate as a submerged structure.

Acknowledgments

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COMPUTING SUSPENDED SAND LOADS FROM FIELD MEASUREMENTS

[Paper No. 56]

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Synopsis

On the Missouri River the bulk of the bedload material is transported in suspension with low concentrations near the water surface and high concentrations near the bed. Because of this nonuniform distribution of particles in suspension, a large part of the sands coarser than 0.10 mm. is carried within a half foot of the bed below the lowest level that can be sampled. It is necessary, therefore, to determine through field measurements the vertical distributions of velocity and concentration in the part of the flow that can be measured, and then to extrapolate these distributions toward the bed in order to compute total suspended sand load. The most reliable procedure is direct measurement of these distributions by obtaining a series of point velocities and point samples in the vertical. Velocities are relatively economical to obtain, as they involve only some extra field work. The samples, however, are relatively expensive, as they involve costly laboratory analysis, not only for concentration but also for mechanical analysis, since it is desired to compute the loads for individual grain-size fractions.

In the interest of economy a procedure has been devised that requires only two sediment samples in each vertical, one depth-integrated sample and one point sample at a reference depth, obtained with a P–46 sampler. A vertical velocity distribution is also measured with a current meter. From these field data the concentration distributions in the measurable part of the flow are deduced by a method developed from relationships presented by Vanoni and Einstein. This method gives a more reliable determination of suspended load than the modified Einstein method of the U.S. Geological Survey, because it uses a direct measurement of the velocity distribution and requires fewer assumptions in deducing the sediment concentration distributions.

Criteria for how close to the bed to extrapolate the measured velocity distribution and the deduced sediment distributions have also been developed. The integration of the velocity and sediment distributions to obtain total suspended load follows the method of Einstein. In the Missouri River, no attempt is made to estimate bedload, as this is small compared with the suspended load.

The Problem

The determination of bed material loads in the Missouri River has become particularly important since Gavins Point Dam was closed in 1955 and now that the navigation and bank stabilization project below Sioux City is nearing completion. The aim of the Corps of Engineers sedimentation program on the Missouri River is continuous evaluation of the changes in river regime brought about by these projects in order to foresee any adverse effects in time for remedial action, to provide data needed in the design of future work, and to contribute to the general knowledge in the field of river engineering. The changes in the riverbed, its elevation, its grain-size composition, and its roughness configuration are of particular significance in this evaluation — and these changes are directly related to the bed material load, defined as the transport rate of those grain sizes that are also found in significant quantities in the bed.

The riverbed material for over a hundred miles below Gavins Point Dam is a fine sand with median grain diameter 0.2 mm. Very little is finer than about 0.06 mm. or coarser than about 0.5 mm. Occasional gravel pockets are found, usually at the downstream toes of sandbars. The river slope is about 0.9 foot per mile. Water depths normally range up to 25 feet in a cross section, with a mean depth around 10 feet, and velocities average 3 to 4 feet per second. Under these conditions the bed material load is transported mainly in suspension. Transport as bedload is a very small part of the total, although for the coarsest grain-size fractions bedload may be more significant.

Two factors complicate the determination of suspended sand loads from field measurements. Since neither the velocities nor the concentrations are uniformly distributed in the vertical, a number of points must be measured in order to define the vertical distributions. This is not only time-consuming in the field but costly in the laboratory. Each sample must be analyzed, not only for concentration but for mechanical analysis, because sand loads are desired for each individual grain-size fraction. Figure 1 shows typical vertical distributions of velocity, obtained with a Price current meter, and concentration, obtained from P–46 samples, for the Missouri River at Omaha.

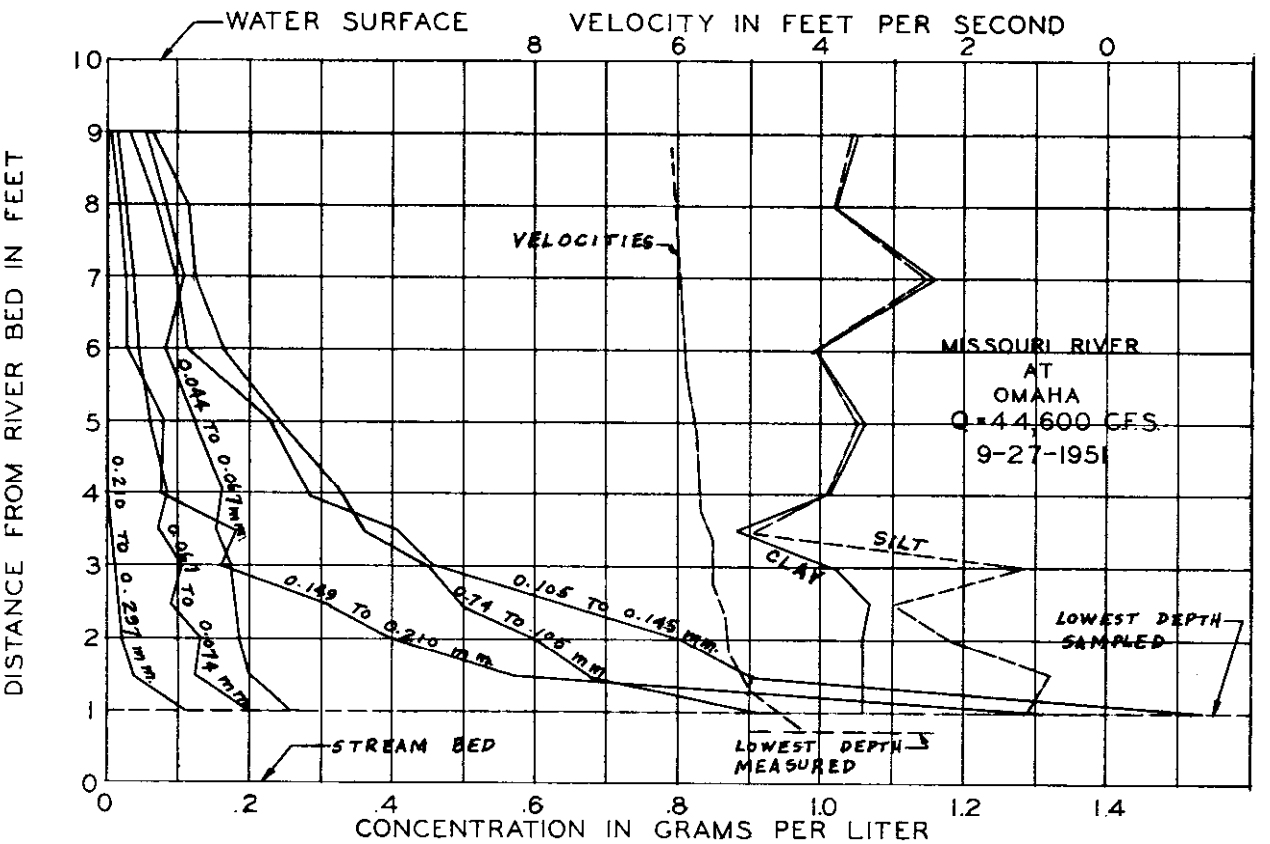


FIGURE 1. — Vertical velocity and concentration distributions in Missouri River.

The second complicating factor is that a significant part of the sand load is carried below the lowest point that can be sampled with the suspended load sampler. This is due to the relatively high concentrations near the bed as compared with the water surface. The nearest approach to the bed that is practical with the Price current meter or with the P–46, the D–43, or the D–49 samplers is about 0.5 foot. It is, therefore, necessary to extrapolate the measured distributions toward the bed in order to determine the total sand load. Figure 2 shows

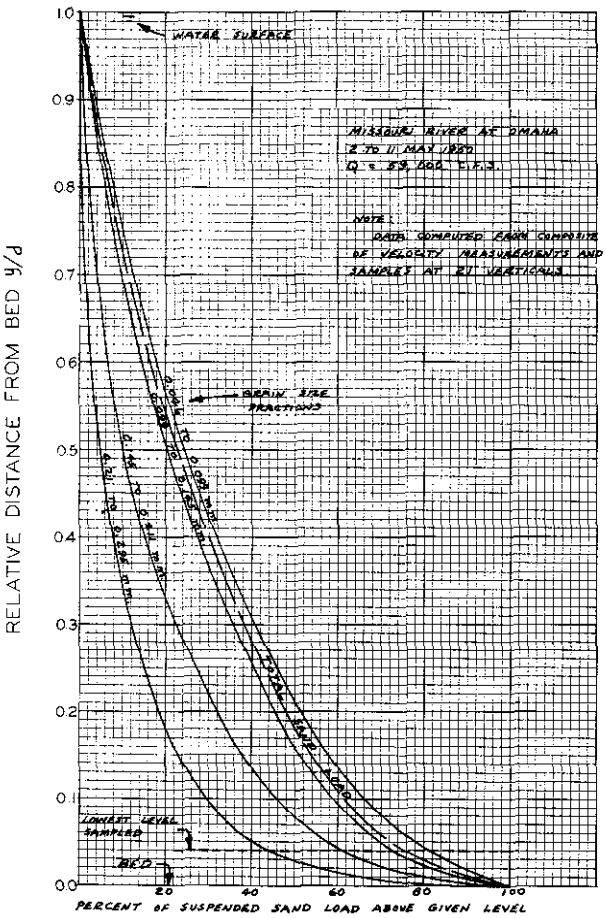


FIGURE 2. — Vertical distribution of suspended sand load in Missouri River.

typical distributions of sand loads in the vertical by grain-size fractions for the Missouri River at Omaha. These curves were computed below the lowest measured points by the extrapolation method to be described later in this paper. A distance from the bed equal to about 0.04 of the depth is indicated in figure 2 as the lowest level that could be measured. The curves in figure 2 show that 75 percent of the total sand load is carried above this level. For the finer grain-size fractions, this includes the bulk of the load. For the coarser sand fractions, however, a smaller part of the load is measured directly. For example, only 45 percent of the load in the 0.211- to 0.295-mm. range is carried within the measured zone; and the rest of the load in this size range must be based on extrapolation of measured points. Since the median grain size of the material in the riverbed is about 0.2 mm., the transport rates for the coarser suspended sand fractions are important in evaluating changes in the riverbed regime.

Objectives

The analysis that follows will lead to four objectives: (1) to obtain relationships based on the measured data that permit extrapolations of velocity and concentration distributions downward below the lowest level of measurement; (2) to establish tentative criteria for the lower limit of integration — or in other words, how close to the bed to extend the velocity and concentration distributions; (3) to develop analytical procedures to facilitate the integration of concentration times velocity over the water depth to obtain sediment load; and (4) to develop a procedure requiring the minimum amount of field measurement and laboratory analysis to define the velocity and concentration distributions in the measurable part of the flow.

Nomenclature

A = a/d, a dimensionless expression for a.
a = The height from the bed of the lower limit to which the suspended load integration is carried (in feet).
α = The height from the bed of the lowest level sampled by a depth-integrating sampler (in feet).
C_α = The average concentration for a given grain-size fraction from the water surface down to a distance α from the bed, as measured with a depth–integrating sampler (grams per liter).
C_p = Known concentration of a given grain-size fraction at a distance p from the bed (grams per liter).
C_{pt} = Known concentration of all sand grain-size fractions at a distance p from the bed (grams per liter).
C_y = The concentration of a given grain-size fraction at a distance y from the bed.
C_{yt} = Total concentration of all sand grain-size fractions at a distance y from the bed (grams per liter).
D = Geometric mean grain size of a given grain-size fraction (in feet).
d = Water depth (in feet).
I_1, I_2 = Definite integrals, functions of A and Z.
J_1, J_2 = Definite integrals, functions of A and Z.
N = The slope $\triangle V_y / \triangle (\log_{10} y)$ of the semilog plot of the velocity distribution.
p = Value of y at point of known concentration (in feet).
q_α = The water discharge per unit width in that part of the flow from the water surface

down to a distance α from the bed (in cubic feet per second per foot).

q_s = The suspended load for a certain grain size fraction per unit width of cross section (in pounds per second per foot).

$q_{s\alpha}$ = The measured load for a certain grain-size fraction per unit width of cross section in that part of the flow from the water surface down to a distance α from the bed (in pounds per second per foot).

q_{st} = The total suspended load for all sand grain-size fractions per unit width of cross section (in pounds per second per foot).

V = Average velocity in a vertical (in feet per second).

V_y = The velocity at a distance y from the bed (in feet per second).

$Y = y/d$ = a dimensionless distance from the bed.

y = Vertical distance from the bed (in feet).

y_1 = Value of y where the extrapolated velocity distribution gives $V_y = 0$ (in feet).

y_2 = Value of y where the extrapolated concentration distribution gives $C_{yt} = 480$ grams per liter (in feet).

Z = The slope $\triangle \log_{10} (C_y) / \triangle \log_{10} \frac{d-y}{y}$ of the logarithmic plot of the concentration distribution.

Semilogarithmic Velocity Distribution

When the velocity distribution in figure 2 is plotted on semilog paper, as in figure 3, the points fit a straight line reasonably well. The equation for this straight line is

$$V_y = N \log_{10} y + \text{const} \qquad (1)$$

Measurements in many rivers verify this semilogarithmic form of the vertical velocity distribution, and additional theoretical justification has been provided by Keulegan (*5*). With the velocity distributed as in equation 1, the *average* velocity in the vertical is given by

$$V = \frac{1}{d}\int_0^d V\,dy = \frac{N}{d}\int_0^d (\log_{10} y)\,dy + \int_0^d \frac{\text{Const}}{d}\,dy$$

$$V = N\,(\log_{10} d) - N/2.3 + \text{const} \qquad (2)$$

Combining equations 2 and 1,

$$V_y = N \log_{10} (y/d) + (V + N/2.3) \qquad (3)$$

Equation 3 describes any semilogarithmic velocity distribution. The parameters V and N can be graphically determined from a plot such as figure 3. N is the slope of the best-fitting straight line through the points. V can also be obtained from the line. To find the value of y where $V_y = V$, equate (2) and (3):

$$N \log_{10} y + \text{const} = N \log_{10} d - N/2.3 + \text{const}$$

$$\text{Log}_{10}\, d/y = 0.434$$

$$y/d = 0.368$$

Therefore, V is obtained from the line of best fit by reading V_y at a height 0.37 of the water depth from the bed ($y = 0.37d$).

The velocity distribution can be extrapolated below the lowest measured point by extending the straight line as in figure 3 or by using equation 3.

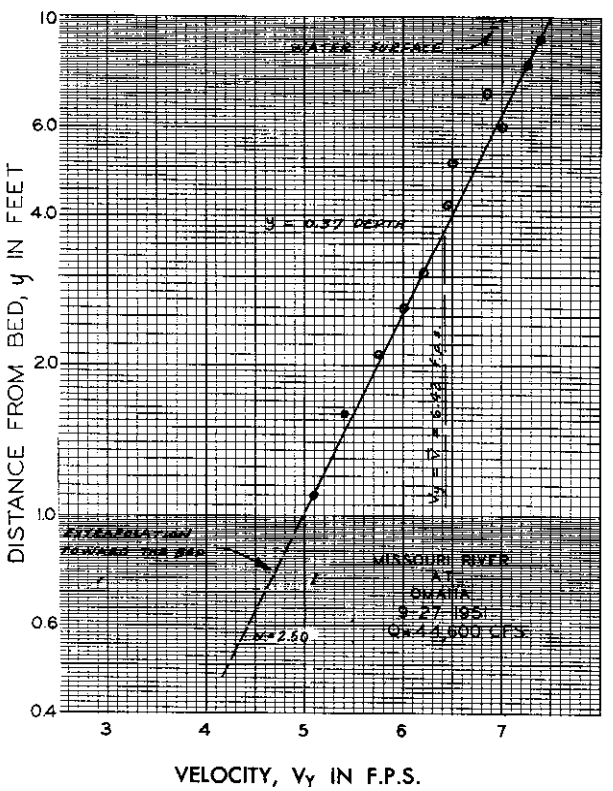


FIGURE 3.—Semilogarithmic plot of velocity distribution.

Concentration Distribution

We also find from field experience, with supporting evidence from the laboratory and some theoretical justification (*6*), that the vertical concentration distributions plotted on log paper in the form $\frac{d-y}{y}$ vs. C_y tends to be a straight line. This is the case in figure 4 which contains logarithmic plots for three of the grain-size fractions shown in figure 2. This form of the concentration distribution seems to hold not only for C_y, the concentration of individual grain-size fractions, but also for C_{yt}, the total concentration for all grain-size fractions in the sand range. The equation for these straight lines is in the form

$$C_y = C_p \left(\frac{p}{d-p}\right)^Z \left(\frac{d-y}{y}\right)^Z \qquad (4)$$

in which Z, C_p, and p are parameters obtained from the straight line that best fits the points plotted as in figure 4. Z is the slope of the line, and C_p is the concentration read from the line at a value of $\frac{d-p}{p}$ corresponding to a reference distance p from the bed.

The concentration distribution can be extra-

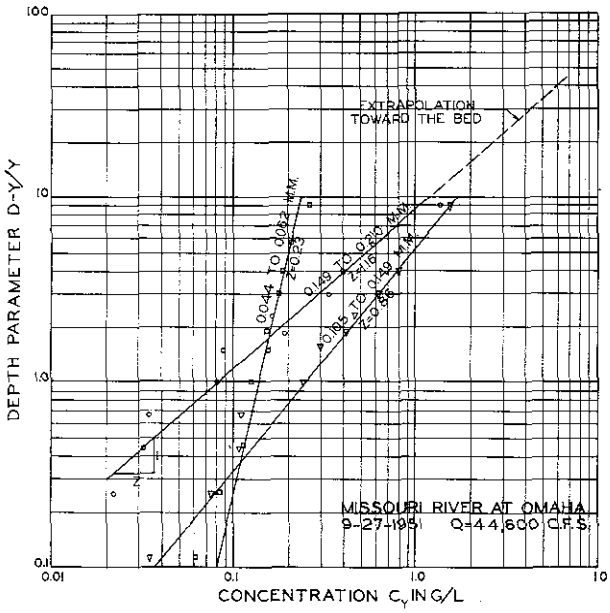


FIGURE 4.—Logarithmic plot of sand concentration distributions.

polated below the lowest measured point by extending the straight line as in figure 4 or by using equation 4.

Integrating to Get Suspended Load

The suspended load per foot of channel width for a given grain-size fraction is obtained by integrating the product of the velocity times the concentration over the depth in the vertical:

$$q_s=0.0624\int_a^d V_y \cdot C_y \cdot d_y \qquad (5)$$

The choice of a, the lower limit of integration, will be explained later.

The above integration can be performed graphically, numerically, or analytically. In the graphical or numerical methods the velocity distribution and the concentration distributions for each grain-size fraction are plotted as in figures 3 and 4. The straight lines of best fit are then extrapolated below the measured points to the lower limit of integration, to $y=a$ for the velocity and to $\frac{d-y}{y}=\frac{d-a}{a}$ for the concentrations. By multiplying values of V_y and C_y at corresponding values of y, a third plot of V_yC_y vs. y is prepared. Finally the area under this curve between $y=a$ and $y=d$ is determined either planimetrically or numerically to obtain the unit suspended load as indicated in equation 5. It may be found that, due to the rapid changes in the values of V_yC_y near the bed, a logarithmic scale for y will be necessary. These methods are laborious, since separate integrations are required for each grain-size fraction for each sampled vertical in the river cross section. They are so time consuming, in fact, that they are impractical for routine use.

Einstein (*2*) has developed an analytical method of integration that reduces the time and labor required to the point where routine use is practical, especially when the method is programed for a digital computer. In preparation for this method, the vertical velocity distribution and the vertical concentration distributions for each grain-size fraction are plotted as in figures 3 and 4 and the straight lines of the best fit are drawn through the plotted points. The values of V, N, Z, p, and C_p are then read from these lines. These parameters are used to obtain q_s from a relationship that will now be developed. If we substitute equations 3 and 4 into equation 5,

$$q_s=0.0624\int_a^d\left[N\log_{10}(y/d)+(V+N/2.3)\right] C_p\left(\frac{p}{d-p}\right)^z\left(\frac{d-y}{y}\right)^z d_y \qquad (6)$$

Let $Y=y/d$, then $dy=d\,dY$. At $y=d$, $Y=1$, and at $y=a$, $Y=\frac{a}{d}=A$.

$$q_s=0.0624\,N\,d\,C_p\left(\frac{p}{d-p}\right)^z\left\{\int_A^1\left(\frac{1-Y}{Y}\right)^z(\log_{10}Y)\,dY+(V/N+0.434)\int_A^1\left(\frac{1-Y}{Y}\right)^z dy\right\} \qquad (7)$$

$$\text{Let } J_1=\int_A^1\left(\frac{1-Y}{Y}\right)^z dy$$

$$\text{and } J_2=\int_A^1\left(\frac{1-Y}{Y}\right)^z(\ln Y)\,dY$$

$$q_s=0.0624\,C_p\,N\,d\left(\frac{p}{d-p}\right)^z\left[0.434\,J_2+(V/N+0.434)\,J_1\right] \qquad (8)$$

The integrals J_2 and J_1, plotted as functions of A and Z, are available in reference (*3*) for the ranges $0.00001<A<0.6$ and $0<Z<2.0$. For values of $2.0<Z<5.0$, we may use Einstein's integrals I_1 and I_2, plotted as functions of A and Z in reference (*2*). J_1 and J_2 are obtained from I_1 and I_2 are as follows:

$$J_1=4.63\frac{(1-A)^z}{A^{z-1}}I_1 \qquad (9)$$

$$J_2=4.63\frac{(1-A)^z}{A^{z-1}}I_1 \qquad (10)$$

If equations 9 and 10 are substituted in equation 8:

$$q_s = 4.63 \times 0.0624\, C_p\, N d\, A \left[\frac{p}{A}\left(\frac{1-A}{d-p}\right)\right]^Z \left[0.434\, I_2 + (V/N + 0.434)\, I_1\right] \quad (8A)$$

Knowing the water depth, d, choosing a lower limit of integration, $A = a/d$, reading the parameters V, N, Z, p, and C_p from the measured velocity and concentration distributions, and obtaining values of J_1 and J_2 from references (*2*) or (*3*), we can compute the unit suspended load q_s from equation 8 for each grain-size fraction.

Defining Concentration Distributions With Less Samples

Figures 3 and 4 indicate that suspended samples and velocity measurements must be obtained at a number of points in the vertical in order to define adequately the parameters V, N, C_p, and Z. The velocities are relatively economical to obtain as they involve only fieldwork. The sediment samples, however, are relatively expensive, as they involve not only the fieldwork in obtaining them but also costly laboratory analysis that includes determination of the mechanical analysis of each sample as well as its concentration. In order to reduce the cost of processing the large volume of load measurements at the Missouri River stations, we sought a method of field sampling and office analysis that would minimize the number of suspended sediment samples required to define the concentration distributions.

In the method that was evolved we continued to measure enough point velocities to define V and N in a semilogarithmic plot such as figure 3. We obtained only two suspended samples, however:

(1) A point sample at a reference level p from the bed. From this sample, values of C_p are obtained for each grain-size fraction.

(2) A depth-integrated sample based on a traverse of the flow from the water surface down to a distance a from the bed. From this sample, values of C_a, the average concentration in the measured, depth-integrated part of the flow, are obtained for each grain-size fraction.

The actual field practice is to obtain two point samples and two depth-integrated samples and to combine each pair in order to obtain larger samples for laboratory analysis. The field measurements define values of d, V, N, p, C_p, a, and C.

The load q_{sa} for each grain-size fraction in the part of the flow that was depth-integrated is given by

$$q_{sa} = 0.0624\, C_a\, q_a \quad (13)$$

q_a is obtained from

$$q_a = \int_a^d V_y dy \quad (14)$$

Substitute the expression for V_y in equation 3 and integrate:

$$q = (d-a)\, V - N\, a \log_{10}\frac{a}{d} \quad (15)$$

q_{sa} can, therefore, be determined from equations 15 and 13, using known measured quantities.

Lower Limit of Integration

The question now arises how close to the bed to extend the velocity and concentration distributions in computing the suspended load. How do we choose a, the lower limit of integration? Inspection of equations 4 and 8 or 8A indicates that a must not equal zero or an infinite concentration at the bed, or infinite values of the integrals I_1, I_2, J_1, or J_2 would result. Three practical limitations on the choice of a come to mind:

(1) Extrapolation should be down to the upper edge of the layer near the bed where bedload supplants suspended load as the mode of transportation. For want of a better criterion, this is taken from Einstein (*2*) as $a = 2D$.

(2) The total concentration cannot be unrealistically high. Recent work of Einstein (*4*) and Bagnold (*1*) indicates that suspensions tend toward some limiting concentration, say 30 p.c.f. (480 g./l.).

(3) The velocity distribution should not be extrapolated below the value of y where $V = 0$, because equation 3 will yield negative velocities for smaller y values.

These limitations suggested the following procedure for choosing the lower limit of integration.

1. Compute the value of $2D$ for the particular grain-size fraction.

2. From a logarithmic plot of $\frac{d-y}{y}$ vs. C_{yt} for the total sand concentrations, obtain the values of Z, C_p, and p. Extrapolate this concentration distribution to find the distance from the bed, y_2, at which $C_{yt} = 480$ g./l. This may be done with a graphical extrapolation of the straight line of best fit or by using equation 4 to obtain

$$y_2 = \frac{d}{1 + \left(\frac{480}{C_{pt}}\right)^{1/Z}\left(\frac{d-p}{p}\right)} \quad (11)$$

3. Extrapolate the semilogarithmic plot of the velocity distribution toward the bed to find the distance from the bed, y_1, at which $V_y = 0$. This may be done with a graphic extrapolation of the straight line of best fit or by using equa-

tion 3 to obtain

$$y_1=\frac{0.37d}{10^{V/N}} \quad (12)$$

4. Choose $a = 2D$ or y_1 or y_2, whichever is the largest.

An expression for q_{sa}/C_p can be obtained from equations 8 or 8A:

$$q_{sa}/C_p=0.0624\,N\,d\left(\frac{p}{d-p}\right)^z \left[0.434J_2+(V/N+0.434)\,J_1\right] \quad (16)$$

$$\text{or } q_{sa}/\,C_p=4.63\times 0.0624\,N\,dA\left[\frac{p}{A}\left(\frac{1-A}{d-p}\right)\right]^z \left[0.434I_2+(V/N+0.434)\,I_1\right] \quad (16A)$$

Knowing p and $A=a/d$ and assuming various values of Z, a relationship between Z and q_{sa}/C_p can be computed and plotted. The resulting graph can be entered with the values of q_{sa}/C_p determined from the field measurements, and the value of Z for each grain-size fraction can be read from the curve. When p is not equal to a, equation 16 or 16A may define two values of Z for a given value of q_{sa}/C_p. In this event Z values are read from the limb of the curve that gives ascending values of Z for q_{sa}/C_p values corresponding to ascending values of D.

After the Z values are determined by this method, the suspended load throughout the vertical is computed from equation 8 or 8A for each grain-size fraction.

Engineers with experience in measuring and computing suspended sand loads will recognize that this method, based on only two suspended load samples, cannot be as good as the more direct method of sampling a number of points to define the vertical concentration distribution. While theoretically, the results will be the same, practically, the precision is not comparable. Due to the variability of point concentrations, the more points that are sampled in a vertical in a short period of time, the better defined is the concentration distribution. On the other hand, regardless of the precision with which the distributions are defined in the measured part of the flow, we still face the uncertainty in extrapolating these distributions to the proximity of the bed. Since the estimated load must be indeterminate, regardless of the precision of the measurements, some economies in making these measurements would seem to be realistic. Although the above method of determining indirectly the Z values requires additional office analysis, the use of a digital computer in making the computations relieves this disadvantage.

This method gives a more reliable determination of suspended load than the modified Einstein method (*3*) developed by the U.S. Geological Survey, because it uses a direct measurement of the velocity distribution and uses enough suspended concentration data to define the vertical distributions without relying as heavily on theoretical relationships. The modified Einstein method does provide an estimate of the bedload as well as the suspended load. The method presented in this paper estimates the suspended loads only.

Examples

Example 1

The velocity and concentration distributions plotted in figure 1 were obtained at one vertical in the Missouri River at Omaha on 27 September 1951. From the plot of V_y vs. $\log_{10}y$ in figure 3, $N=2.50$ and $V=6.42$ f.p.s. For each grain-size fraction, $\log_{10}C_y$ was plotted against $\log_{10}\frac{d-y}{y}$ as in figure 4. The values of Z and C_p obtained from the best fitting straight lines are shown in table 1. The known concentration, C_p, was taken at mid-depth where $p =$ 5.0 ft. and $\frac{d-p}{p}=1$. The results of the suspended load computations for this vertical, with the use of equation 8A, are summarized in table 1. The ratios in the right-hand column show how much the suspended load exceeds the measured load for each grain-size interval and for the total sand. The values in the first line were based on a $\log_{10}C_{yt}$ vs. $\log_{10}\frac{d-y}{y}$ plot of the total sands. These are less than the values in the last line in table 1, obtained by adding the loads for the individual grain-size fractions.

Example 2

At station 3 + 65 on the Aksarben Bridge across the Missouri River at Omaha on 27 June 1957, the following data were obtained (table 2):

(1) A depth measurement of 16.0 feet.

(2) A vertical velocity distribution.

(3) A depth-integrated sample down to a point 0.5 foot from the bed.

(4) A point sample 8.0 feet from the bed.

A plot of V_y vs. $\log_{10}y$ gave $N = 3.00$ and $V =$ 5.77 f.p.s. Laboratory analyses of the two sediment samples yielded the values of C_a and C_p shown in table 3. A Z vs. q_{sa}/C_p relation was computed from equation 16A. The results are summarized in table 2 and plotted in figure 5.

The determination of the Z values in the sampled vertical is shown in table 3. From the measured values of C_a and C_p, values of q_{sa}/C_p are computed for each grain-size interval. The corresponding Z values are then read from figure 5.

TABLE 1.—*Suspended sand load computation from point samples at station 3+65 of Missouri River at Omaha, 27 September 1951*

$N=2.50$; $V=6.42$ f.p.s.; $d=10.0$ ft.; $p=5.0$ ft.; $\alpha=1.0$ ft.;

$$y_1=\frac{0.37\times10.0}{10^{6.43/2.50}}=0.010 \text{ ft.; } y_2=\frac{10.0}{1+\left(\frac{480}{0.82}\right)^{1/6.72}}=\frac{10.0-5.0}{5.0}=0.00143 \text{ ft.}$$

Size interval	Mean size D	Z	C_p	Computed suspended load: $2D$	Lower limit a	$A=\frac{a}{d}$	I_1	I_2	q_s	Computed measured load: $A=\frac{a}{d}$	I_1	I_2	$q_s\alpha$	Ratio $\frac{q_s}{q_s\alpha}$
Mm.	*Mm.*		*G./l.*	*Ft.*				(—)	*Lb./sec./ft.*			(—)	*Lb./sec./ft.*	
Total sand	0.185	0.72	0.820	0.00121	y_1	0.0010	3.616	10.29	5.46	0.10	0.4878	0.6165	3.448	1.59
0.044–0.062	.053	.23	.139	.00035	y_1	.0010	47.72	68.02	.559	.10	1.1444	1.0603	.495	1.13
0.062–0.074	.068	.55	.058	.00045	y_1	.0010	7.980	18.09	.301	.10	.6325	.7309	.0222	1.36
0.074–0.105	.099	.58	.270	.00065	y_1	.0010	6.880	16.26	1.456	.10	.6027	.7084	1.047	1.39
0.105–0.149	.127	.86	.240	.00083	y_1	.0010	2.065	6.935	2.099	.10	.4031	.5421	1.117	1.88
0.149–0.210	.179	1.16	.082	.00117	y_1	.0010	0.812	3.579	1.579	.10	.2837	.4295	.505	3.12
0.210–0.297	.254	1.39	.0031	.00167	y_1	.0010	0.488	2.460	.132	.10	.2263	.3602	.025	5.32
Total or average									6.126				3.412	1.80

TABLE 2.—*$q_{s\alpha}C_p$ computed for various values of Z at station 3+65 of Missouri River at Omaha, 27 June 1957*

$N=3.00$; $V=5.77$ f.p.s.; $d=16.0$ ft.; $p=8.0$ ft.; $\alpha=0.5$ ft.

Assumed Z	$A=\alpha/d$	I_1	I_2 (—)	$q_{s\alpha}/C_p$
0	0.03125	6.7081	5.9593	5.73
0.375	.03125	2.0820	2.7992	5.76
0.575	.03125	1.2710	2.0166	6.59
0.790	.03125	.80812	1.4852	8.25
1.10	.03125	.45209	.9318	12.90
1.26	.03125	.38223	.8771	17.1

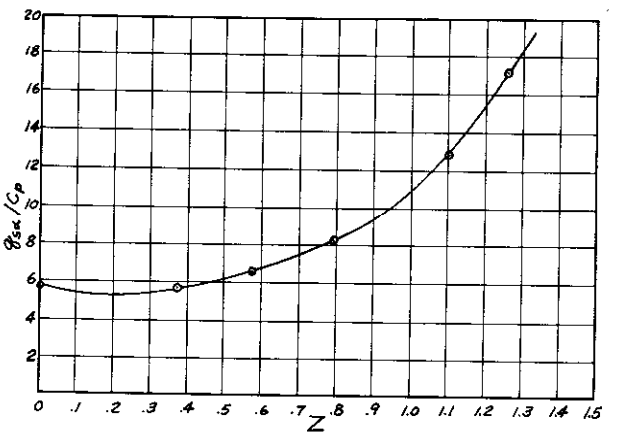


FIGURE 5. Z vs. $q_{s\alpha}/C_p$ plot for example 2.

With the use of the Z values determined in table 3, the load for each grain-size interval can be computed as in example 1. It can be seen from table 3 that considerable judgment must be applied in selecting the Z values and that the method will fail if measured concentrations are too low for adequate computation of the needed parameters.

TABLE 3.—*Determining Z from measured q_s/C_p values at station 3+65 of Missouri River at Omaha, 27 June 1957*

$N=3.00$; $V=5.77$ f.p.s. $d=16.0$ ft.; $\alpha=0.5$ ft.; $q\alpha=(16.0-0.5)$ $5.77-3.00\times0.5\log_{10}0.5/16.0=91.6=\frac{\text{c.f.s.}}{\text{ft.}}$

Size interval	Mean size D	$C\alpha$	C_p	q_s	$\frac{q_s\alpha}{C_p}$	Z (from fig. 5)
Mm.	*Mm.*	*G./l.*	*G./l.*	*Lb./sec./ft*		
0.044–0.053	0.049	0.115	0.098	0.657	6.70	[1]0
0.053–0.074	.063	.142	.132	.811	6.15	0.50
0.074–0.105	.099	.154	.055	.881	16.0	1.22
0.105–0.149	.127	.0426	.0422	.243	5.75	([2])
0.149–0.210	.179	.0033	.0063	.0189	3.00	([2])
Total Sand		.458	.365	2.62	7.19	.66

[1] Point falls to left of $Z=0$ on left limb of curve. Z taken as zero as would be expected of such a small-grain size in this high a velocity.

[2] Measured concentration too low to provide a good estimate.

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DESIGN PRACTICE FOR LEVEE REVETMENT ON WEST COAST INTERMITTENT STREAMS

[Paper No. 57]

By A. P. GILDEA, *chief, Hydraulics Section, U.S. Army Engineer District, Los Angeles*

Synopsis

One of the early attempts to evaluate the effectiveness of riprap revetment to protect the banks of intermittent streams flowing at velocities approaching the critical value was made by the U.S. Army Engineer District at Los Angeles after a major flood occurred in the area in March 1938. Unfortunately, record surveys of conditions before the flood were not sufficient to compute accurately the maximum scour during the flood. Because a substantial amount of the revetted levees and banks were destroyed by the flood, drastic design modifications were made for rehabilitation after the flood.

Since 1950 there has been a progressive development of design criteria for revetment to resist the erosive forces of velocities in the range between 10 and 18 feet per second. In 1952, the Office, Chief of Engineers, issued Civil Works Engineer Bulletin 52-15 in order to standardize levee slope protection works. In recent years the Los Angeles District of the Army Corps of Engineers has built levees on three California streams that have only intermittent discharge. The design of revetment is different for each. These designs are delineated so that current practice may be appraised. To protect from underscour, the toe of the revetment should be carried to a point slightly below maximum depth of anticipated scour by the design flood. This point may be 6 to 12 feet below the thalweg of the stream.

Introduction

During the mid-1930's emphasis was placed on public works in order to bring employment to depressed areas. The big problem, however, was that the entire United States was a depressed area. Thus, flood control practices in the form of local protection projects were established by the Congress. On the West Coast one of the first such projects authorized was that for the Los Angeles River, which flows through the metropolitan area of the city of Los Angeles. As part of the project work, the banks of the Los Angeles River and its tributaries were revetted with dumped and hand-placed stone obtained from a local quarry. These were the days when the term "one man stone" had meaning.

After completion of many miles of paving of the Los Angeles River, a major flood occurred in the area in March 1938. Much of the channel revetment resisted the flood flows, but unfortunately levees and banks were destroyed in many places. The revetment destroyed by the flood appeared to have failed as a result of underscour of the levee toe. A total of 15 miles of hand-laid revetment stone had been placed on a side slope of 1 on 3 and controlled by two specifications:

(1) Type A for concave side of bends with curvature having radii of 3,500 feet or less. Stone consisted of 12- to 16-inch pieces placed on a 6-inch blanket of spoils. Thickness of stone layer was generally 12 inches, except at streambed, where a heavier section was made up of 16- to 24-inch quarry stone extended to a depth below grade.

(2) Type B for straight levees, for the convex side of all bends and for the concave side of bends with a curve radius greater than 3,500 feet (fig. 1).

Types of bank protection used by local agencies in the area consisted of wire and rock mattress, gunite slope paving, asphalt slope paving, and loose stone paving. A total of 23 miles of channel revetment consisted of heavy stone dumped in place on a slope of about 1 on 2 (fig. 2). About 30 miles of channel had banks protected by pipe and wire fence as shown in figures 3 and 4.

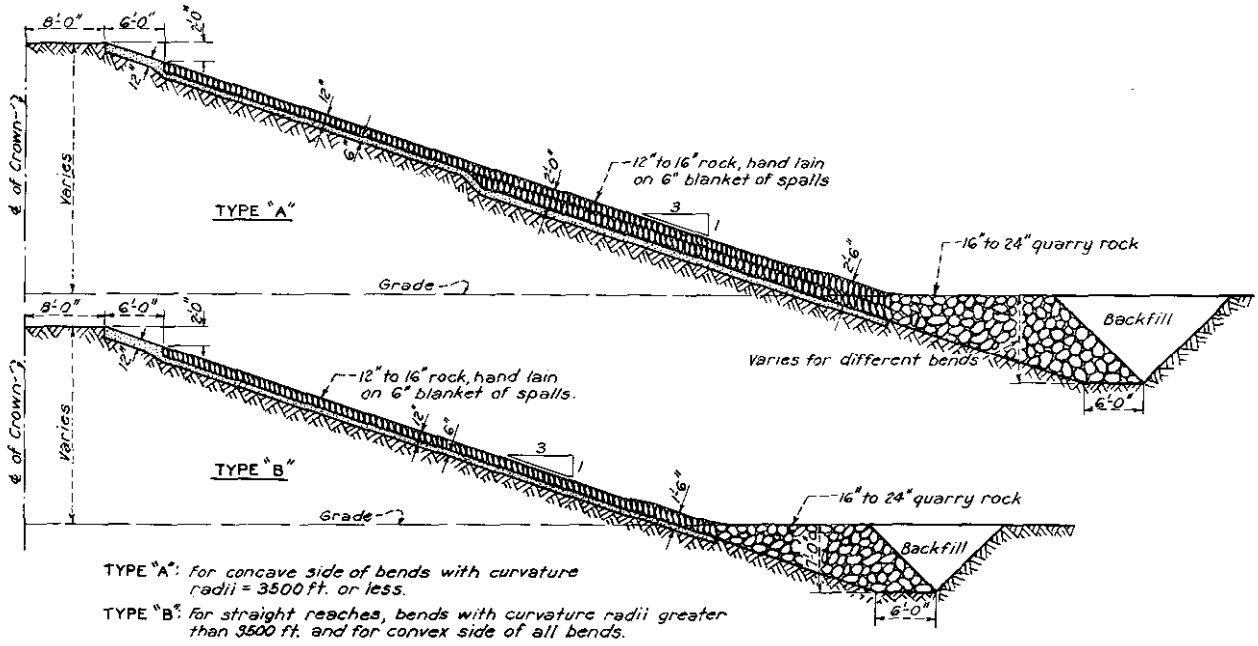


FIGURE 1. — Rock paving bank protection; typical section.

Damage Resulting from 1938 Flood

The upper reach of Los Angeles River northwest of the city was not completed as a whole at the time of flood. The channel in the vicinity of Griffith Park was designed to carry a discharge of 40,000 c.f.s. on a bottom slope varying between 0.0037 and 0.0040. An estimated peak discharge of 60,000 c.f.s. occurred in this reach on March 2, 1938, with velocities in excess of 20 feet per second. The failures on the upper Los Angeles River may be attributed directly to poor channel alinement, disturbances created by side inflows, and ponding of local drainage behind levees. Exploration by test pits and probing at the sites of some failures disclosed that 18 inches of the top layer of toe rock had been removed, but that otherwise the deeper rock had not been disturbed. There were some indications that failure may have commenced immediately above the toe rock. Other failures, especially on curves, seem to have commenced by underscour of the levee toe, the stone at this point having been moved out, and water entered the fine sand of levee fill, undercutting the upper

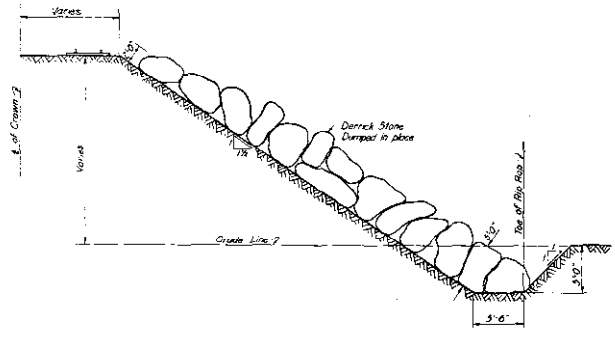


FIGURE 2. — Riprap bank protection; typical section.

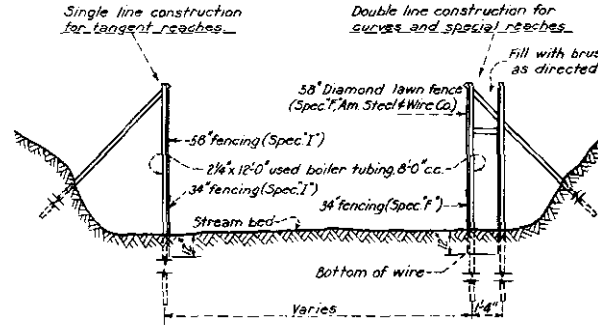


FIGURE 3. — Pipe-and-wire bank protection; typical section.

section. Failure progressed downstream until a break of sufficient capacity to bypass the levee was made, or until the current was deflected away from the levee. The velocity of flow in this reach of the river probably equaled critical velocity with attendant undulations until the channel banks were breached. Figure 5 shows an aerial view of damaged levees as a result of



FIGURE 4. — Typical pipe-and-wire fence used for bank protection. Note how channel bottom is eroding and undercutting fence along left bank.

sinuous channel alinement. Figure 6 shows two photographs; one indicating destruction of one bank of the river as a result of concentrated side inflow from the opposite bank; the other showing failure along the outside of a bend. Figure 7 shows photographs of destruction of a railroad bridge as a result of one pier foundation being undercut by scour.

The downstream reach of the channel southeast of the city of Los Angeles carried a peak discharge estimated at 63,000 c.f.s., with the maximum velocity of approximately 17 feet per second. The slope of the bottom was 0.0025. Although the flood was of sufficient magnitude to destroy partially or totally seven railroad or highway bridges and to do considerable damage to the bank in several locations, no appreciable damage occurred to the 2.4 miles of straight rock-paved channel, again indicating that channel alinement was a major factor in the performance of the revetment. Also the velocity of flow was definitely less than critical velocity. Photographs of riverbank dumped stone revetment show failure probably resulting from underscour along the levee toe (fig. 8). Figure 9 shows a straight reach where bridges were destroyed but the bank revetment remained intact throughout the flood.

The pipe and wire fence type of protection used extensively along Rio Hondo and San Gabriel River played an important part in preventing bank erosion and confining the flood within channels in straight or nearly straight reaches. Large sections failed, however, on the outside of bends or where some obstruction deflected the main current against the fence. The weakness of this form of protection was the inability to deflect cross flows sufficiently to prevent bank erosion back of the fence and the ease with which pipe posts were broken off or pulled out when encountered by the full force of the current. Figure 10 shows photographs of bank erosion and destruction of pipe and wire fence type of protection.

Transportation and Deposition of Sediment

After the March 1938 flood, approximately 36 miles of the Los Angeles River from a point in the San Fernando Valley to the Pacific Ocean were examined in an effort to determine the magnitude of scour. However, between the date of the previous survey and the date of the flood, considerable construction work had been carried out in several reaches. For the most part this consisted of removal of material from the



FIGURE 5. — Upstream view of damage to Los Angeles River channel in 1938 flood: Maximum discharge 60,000 c.f.s.; maximum velocity about 20 f.p.s.

riverbed for levee construction. This work undoubtedly had some effect on the sediment movement, but its importance cannot be determined. The change in volume in several typical reaches not affected by construction was computed. That is, computations of "scour" or "deposit" were attempted. In many cases scour of both banks and bottom occurred. In other cases where the entire levee was eroded away, the scour in the riverbed could only be estimated.

It is recognized that the above measurements indicate only the net change in section due to the storm. It is probable that many reaches of channel were scoured to a considerable depth on the rising stage and refilled to practically the original elevation on the falling stage. In such cases measurements after the flood would erroneously indicate little or no movement of sediment during the flood. To obtain an indication of the maximum scour, probings to a solid bottom were made in several locations. On the Los Angeles River immediately upstream from Los Feliz bridge, 14 cross sections were surveyed and probed. Thus the quantity of deposit after the maximum scour could be estimated. No satisfactory cross section studies before the storm were available. Therefore, the maximum scour and net change of section could not be determined. As for the ground surface formation in the streambed and banks, it would be classed as recent alluvium consisting chiefly of silt, sand, and gravel. Examination of the plot of centerline profiles measured before and after the storm leads to the conclusion that the average scour was about 6 feet (fig. 11). Width of channel at this point was 190 feet. The flood discharge at this location was estimated at 60,000 c.f.s., with mean velocity of about 13 feet per second.

Near the mouth of the Los Angeles River probings were made at two cross sections 200 feet apart. The probings and the cross sections

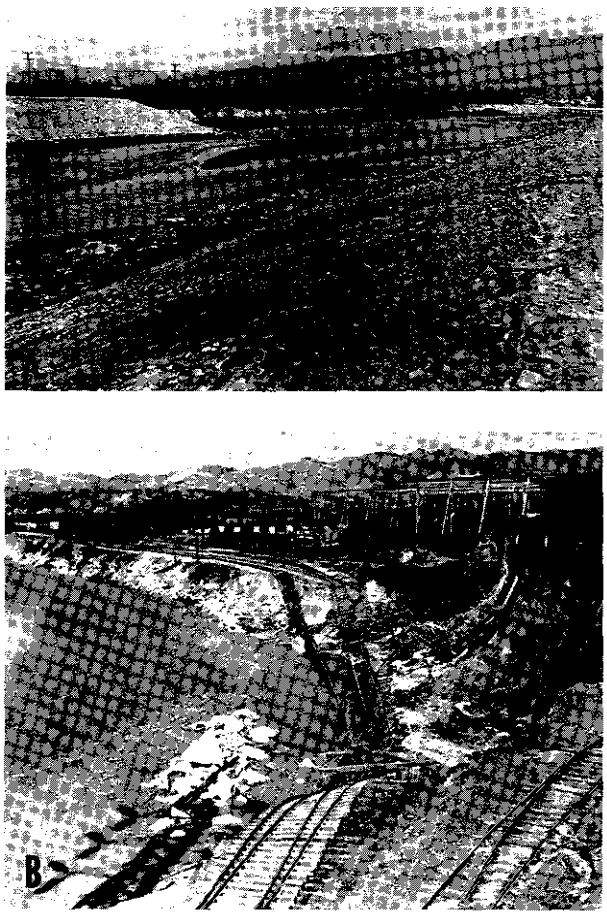


FIGURE 6.—Upstream views of Los Angeles River channel after 1938 flood: *A*, Right bank destroyed downstream from Verdugo Wash confluence; *B*, left bank damaged along outside of bend.

indicate a scour of 15,400 cubic yards per station before the peak scour, and a deposit of 13,530 cubic yards between the time of the peak scour and 14 March, about 12 days later. An additional deposit of 1,270 cubic yards per station occurred in the subsequent month. The width of channel at this point is 540 feet, and the peak discharge was about 75,000 c.f.s. with a mean velocity of about 12 feet per second. One final effort was made to obtain a reasonable estimate of total volume of sediment transported from the Los Angeles River by means of a hydrographic survey of the Pacific Ocean at the mouth of the river. From a study of aerial photographs and soundings, it was estimated that 6,600,000 cubic yards of sediment was deposited by the flood.

Design Modifications Resulting From 1938 Flood

The flood of March 1938 demonstrated that the type of rock toe constructed along the river prior to the flood was vulnerable to high velocity currents. That is, the revetment was weak at the junction of the slope and the invert grade and the size of the rock in the toe protection section was too small. Therefore, a complete new design was developed for rehabilitation of the river after the flood. All remaining undamaged stone revetment was grouted, and where levee sections had to be rebuilt the slope paving consisted of a 12-inch thick concrete slab.

Levee Toe Protection

To protect the levee slope paving from being undercut by the high velocity current, a heavy toe section made up of derrick stone varying in size from 1 to 3 tons was developed. The depth of the toe protection extended 7 feet below

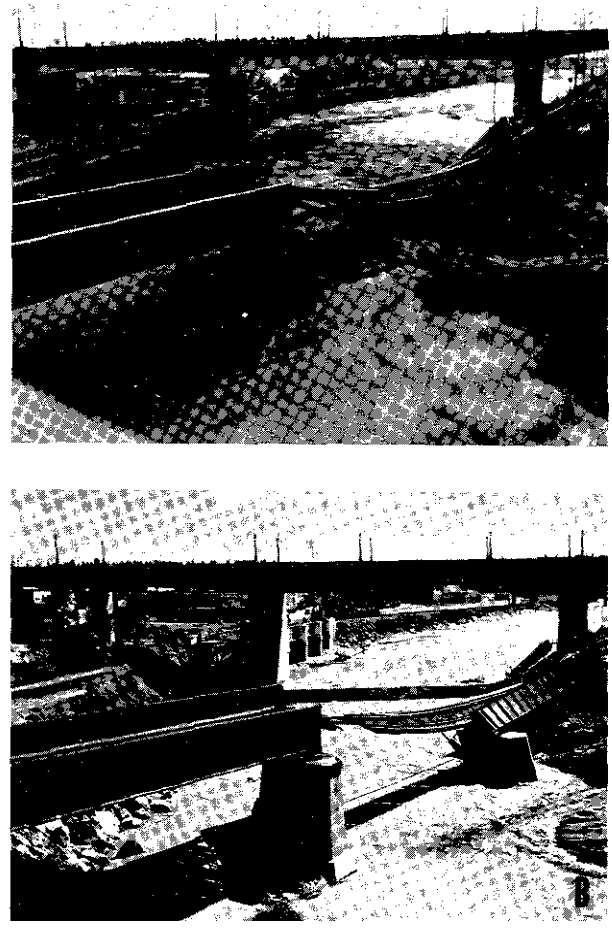


FIGURE 7.— Railroad bridge crossing Los Angeles River destroyed during 1938 flood as result of underscour of pier: *A*, High discharge during flood; *B*, after flood receded.

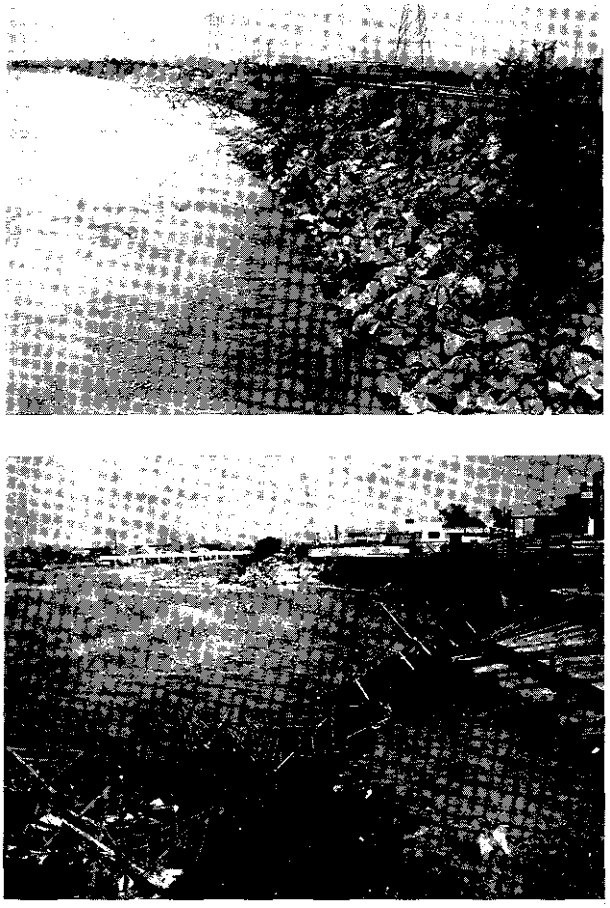


FIGURE 8.— Downstream views of Los Angeles River showing bank failure after dump stone was undercut and washed away.

streambed, and the voids between the stone were filled with concrete. Along the outside of sharp curves, the toe was extended 9 feet below streambed and 20-foot long steel sheet piling was driven under the toe for added protection. Details of this design are shown in figure 12. Placement of the heavy stone along the toe of levee is shown in figure 13. When the voids in the toe stone were filled with concrete, the entire section was made massive but also made rigid. The 9-foot depth of toe would allow 9 feet of streambed degradation, but if the degradation exceeded 9 feet, the heavy toe would be undermined and probably would fall into the scour hole. Also filling the voids with concrete produced a relatively smooth and rigid boundary that in turn induced scour along the levee toe. Figure 13 shows photographs of the toe stone being placed along the banks of the Los Angeles River channel. The photographs also show the character of the banks and bottom material, which was made up of fine sand and silt.

Use of Stabilizers

In the Los Angeles River, the use of transverse sills, called stabilizers, were placed at intervals of about 3,000 feet to act as controls for preventing progressive streambed degradation. During the period 1939–44, flood discharges in the river were moderate, the maximum discharge being about one-third of the design value. A survey of streambed conditions in 1944 showed that local scour at the stabilizers had progressed to the point where extraordinary maintenance work was required. To resist further degradation, heavy stone had to be placed downstream from the stabilizers and along the exposed toe of the levees for a considerable distance downstream. Figure 14 illustrates the eroded condition of the streambed in the vicinity of a typical stabilizer. Note also the exposed toe of levee for a considerable distance downstream. The smooth surface produced by the grouted stone induces scour along the rigid boundary. The experience with stabilizers on Los Angeles River indicated that the introduc-

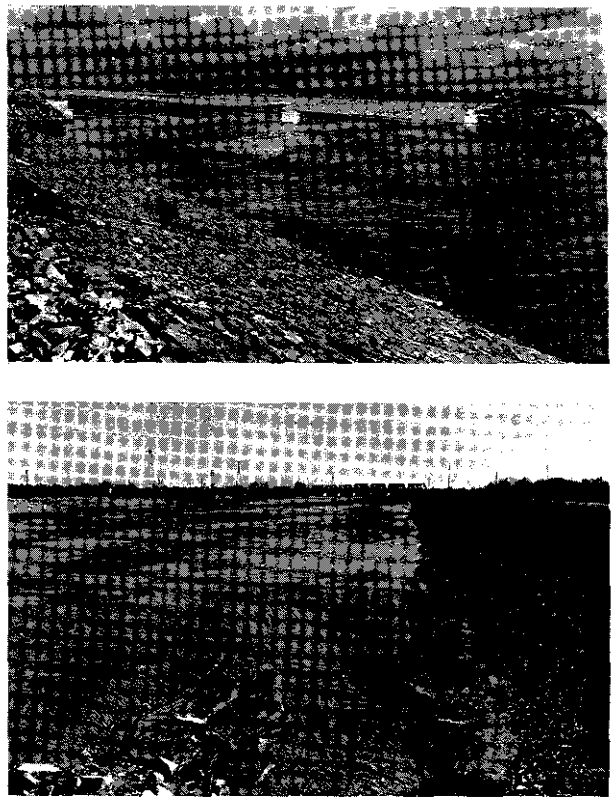


FIGURE 9.— Straight reach of lower Los Angeles River where handplaced stone withstood the flood. Note bridge truss destroyed in upper photograph.

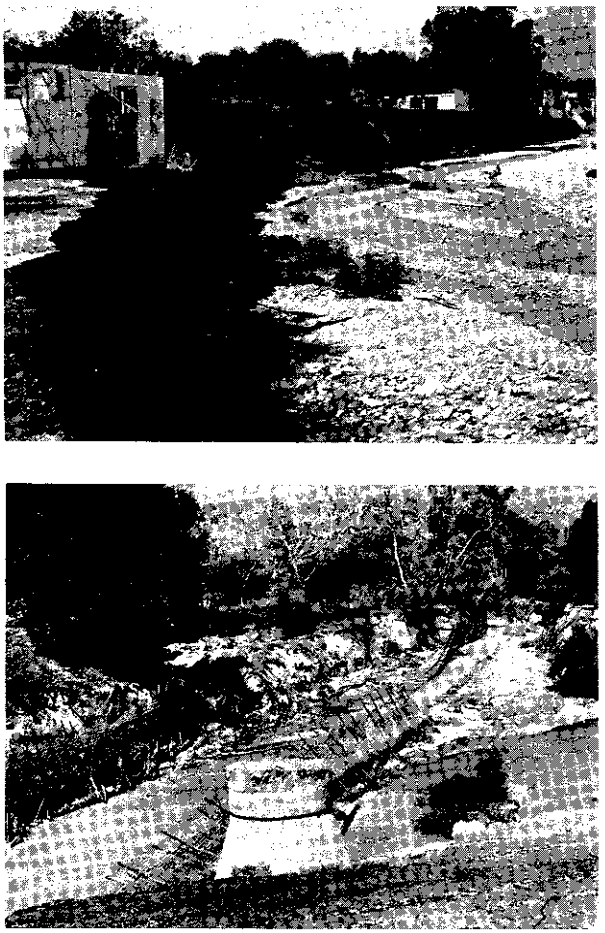


FIGURE 10. — Views showing inadequacy of pipe-and-wire fence to protect against bank erosion.

tion of a rigid boundary in the form of a transverse sill produced an aggravated local scour condition that might produce levee failure more rapidly in any one flood than would general streambed degradation.

Use of Grouted Cobblestone

On streams such as the upper reaches of San Gabriel River near the mountains and the Ventura River in which the streambed material is made up of a large percentage of cobblestones, advantage was taken of this material and a levee-revetment design based on the use of cobblestones was developed. This design specified a layer of dumped stone 18 inches thick at the top of the levee and 24 inches thick at the toe, and the voids filled with sand-cement grout. The toe of the stone layer was placed at a depth 8 feet below the thalweg of the stream. The excavated toe trench was backfilled with random streambed material. Cost of this type of levee was less than importing quarry stone large enough to resist the velocity that would be 16 to 18 feet per second. Figure 15 shows the design details for this type of revetment. Figure 16 shows characteristics of streambed material and dumped stone before and after placement of grout. Only moderate floods have occurred since completion of construction of these two levees; therefore, no experience is available to demonstrate the effectiveness of this type of construction.

Field Research to Support Design Assumptions

West Coast districts of the Corps of Engineers have made studies to determine adequacy of riprap revetment for protection of levee banks against erosion. The usual practice was to use large particles with corresponding large voids. For example, in the Portland District field observations in 1949 showed that an 18-inch layer of dumped stone consisting of individual pieces weighing from 50 to 300 pounds is generally adequate for streams having maximum observed bank velocities of approximately 10 feet per second, provided the toe section is properly designed to prevent undercutting. The general practice in this regard was to carry the bank revetment to a depth 5 feet below the thalweg of the stream.

Civil Works Investigations

In 1951 the Portland district conducted high velocity revetment tests in the outlet channel immediately downstream from Dorena Dam in the Willamette Valley. Four sizes of rock revetment were tested against velocities ranging from 5 to 23 feet per second. The test sections had the advantage of being conducted under prototype conditions, with flows in the test channel controlled by regulating the outlet gates of Dorena Dam. The bottom of the 200-foot long test channel was grouted to prevent undercutting at the toes of the revetment slopes. The bank for the test sections had a slope of 1 on 2. During the initial stages of the tests the downstream end of the test channel eroded severely and a grouted rock rib was constructed at that point. The results of the tests in general showed that rocks ranging from 3 to 130 pounds in size, with an average weight of 20 pounds, resisted attack velocities adjacent to the bank from 6 to 11.5 feet per second. Rock ranging in size from 3 to 200 pounds, with an average size of 40 pounds, resisted attack velocities of 5.5 to 9 feet per second and probably would have resisted considerably higher velocities. Rock ranging in size from 7 to 700 pounds, with an average size of 80 pounds, failed with an average attack velocity of 13.3 feet per second. Also, rock in the range from 8 to 700 pounds with an average

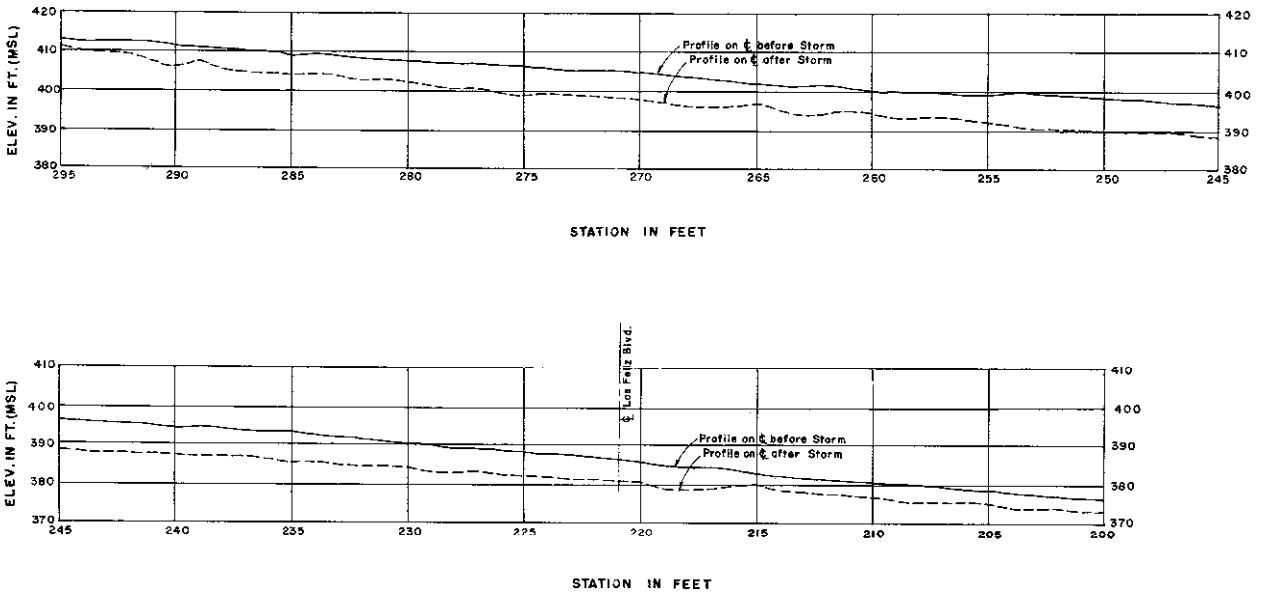


FIGURE 11. — Los Angeles River streambed profile before and after flood of March 1938.

size of approximately 100 pounds failed with an average attack velocity of 16.5 feet per second. The thickness of the slope protection tested ranged from 12 to 24 inches.

Between 1949 and 1951 the Seattle District conducted a civil works investigation to study the minimum riprap requirements for protection against high velocity flow. The investigation involved construction of test sections with various sizes of riprap along the Green River,

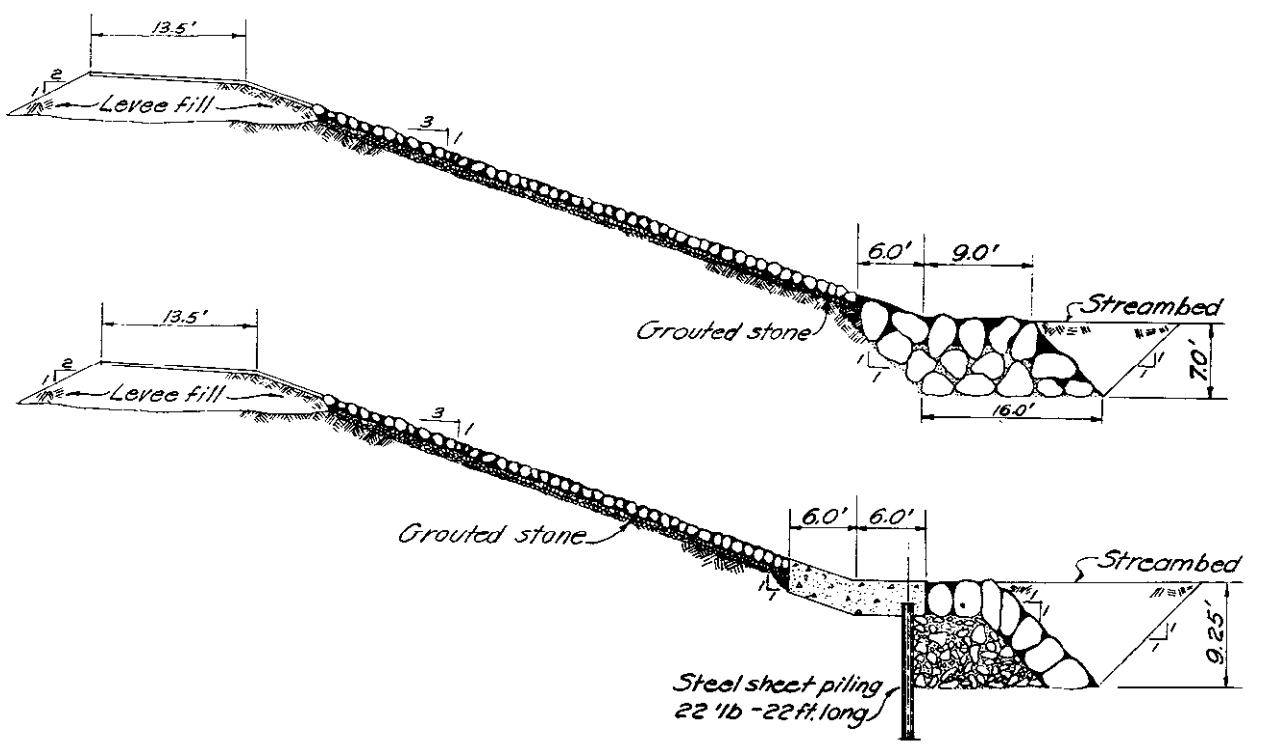
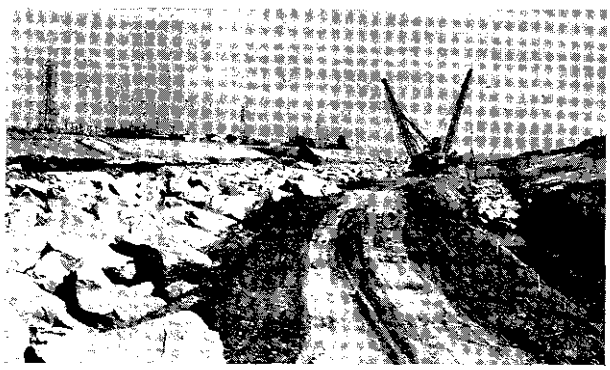


FIGURE 12. — Los Angeles River typical levee revetment; toe design used after 1938 flood.

which is a high velocity stream. A total of 6 rock sections were constructed with average rock size ranging from 20 to 400 pounds. The thickness of the revetment on the various sections ranged from 12 to 24 inches and the various sections were constructed on 1-on-2 slopes. In addition, one section was constructed with cobblestone ranging in size from 4 to 12 inches and the thickness of the revetment ranging from 16 to 19 inches. During a high water period in February 1951, observations and channel and bank velocity measurements were made at the test sections. The results of the rock revetment tests indicate that a 12-inch blanket of riprap in which 50 percent of the rock weighs between 20 and 90 pounds will provide protection against velocities of up to 10 feet per second and that an 18-inch thick riprap blanket consisting of rock averaging 100 pounds or over is necessary to provide protection against current velocities of up to 12 feet per second.



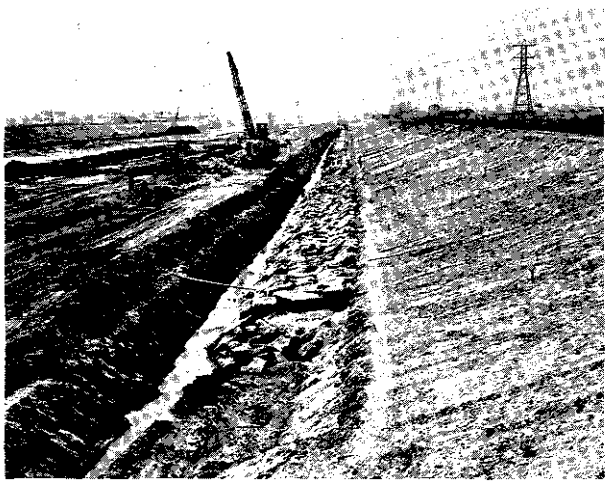


FIGURE 13. — Heavy toe stone being placed by derrick along banks of Los Angeles River channel.





FIGURE 14. — Eroded streambed in vicinity of stabilizer: *A*, Scour at stabilizer for skew railroad bridge is about 6 feet and diminishes to 3 feet in a downstream distance of 80 feet; *B*, along right levee toe, scour is maximum of 5 feet and diminishes to 3 feet in a downstream distance of 500 feet.

Engineer Bulletin 52-15

As a result of these civil works investigation projects involving an appraisal of existing slope protection on river banks and levees and the special test sections, the Office, Chief of Engineers issued Civil Works Engineer Bulletin 52–15, entitled "Slope Protection," dated 2 June 1952. This bulletin gives riprap thickness, rock size, and approximate grading for various ranges of channel velocities. In the bulletin the required thickness and size of rock are based on the velocity within 10 feet of the bank. The bulletin provides that where hydraulic dredge tailings are readily available, a 12-inch layer of cobbles with a maximum size of 10 to 12 inches is satisfactory for bank velocities up to 8 feet per second. The bulletin also provides that where channels are formed by banks or levees of fine sands or silts that are readily erodible, protection by a layer of gravel or crushed stone

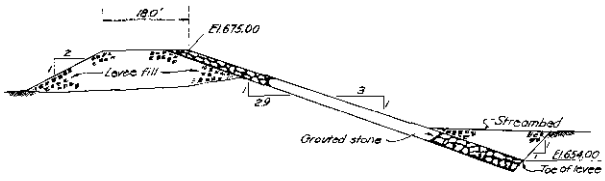


FIGURE 15.—Ventura River typical levee revetment of grouted cobblestone.

8 to 12 inches in thickness will be required for velocities between 3 and 6 feet per second. The bulletin further states that in most cases, riprap should be terminated in a rock toe at the level of the streambed to prevent undermining the channel protection. The toe should have a minimum base width of 6 feet and a minimum thickness of 3 feet with 1-on-1½ side slopes. Where the channel is composed of sand or silt, it is considered better practice to carry the riprap down below the channel level to a minimum depth of 5 feet vertically and omit the rock toe. On large rivers, or tidal estuaries, having a considerable depth of flow at low water stages, the riprap need be carried down only 5 feet vertically below mean low water. In such cases a rock toe is unnecessary. The recommended stone gradation and thickness is given in table 1:

TABLE 1.—*Recommended sizes of stone riprap for indicated velocity*

Current velocity (ft./sec.)	Riprap thickness	Maximum size	Riprap grading		Filter thickness
			Average size	20 percent size	
	Inches	*Pounds*	*Pounds*	*Pounds*	*Inches*
6 to 12	12	150	30 to 50	20	6
12 to 15	15	250	50 to 80	20	6
15 to 18	18	400	80 to 150	20	6

Many offices of the Corps of Engineers have had difficulty in using the criteria in Civil Works Engineer Bulletin 52–15 because the specified velocity 10 feet from the bank during design flood stages is very difficult to measure in the field and cannot be reliably computed. However, the criteria given in the bulletin are considered as a guide rather than as hard and fast specifications. Deviation from Civil Works Engineer Bulletin 52–15 appears desirable, particularly when the levee follows sinuous alinement.

Recent Design Practice

The various field observations and research investigations appear to have established adequate design criteria for size of revetment stone and thickness of layer to withstand various velocities up to a maximum of about 15 feet per second. However, to date little knowledge has been gained regarding the depth to which the levee revetment must be carried to be safe against scour. Unquestionably, the maximum scour depends upon a number of variables, such as the fineness of the streambed material, alinement of the boundary of the stream, gradient and velocity of stream, and source of supply of sediment.

In recent years the Los Angeles District of the Corps of Engineers has constructed levees of three West Coast streams; namely, Santa Ana River in Riverside County, Santa Clara River in Ventura County, and Santa Maria River in Santa Barbara County, Calif. On each of these rivers, the most controversial feature of the design was the required depth of the levee toe.

On the Santa Ana River in the vicinity of the city of Riverside, levees were designed and constructed to protect the local area from flood flows having near critical velocities of about 18 feet per second. The design discharge and depth of flow would be 195,000 c.f.s. and 15 feet, respectively. Figure 17 shows an aerial view of partial improvement in which a single levee has

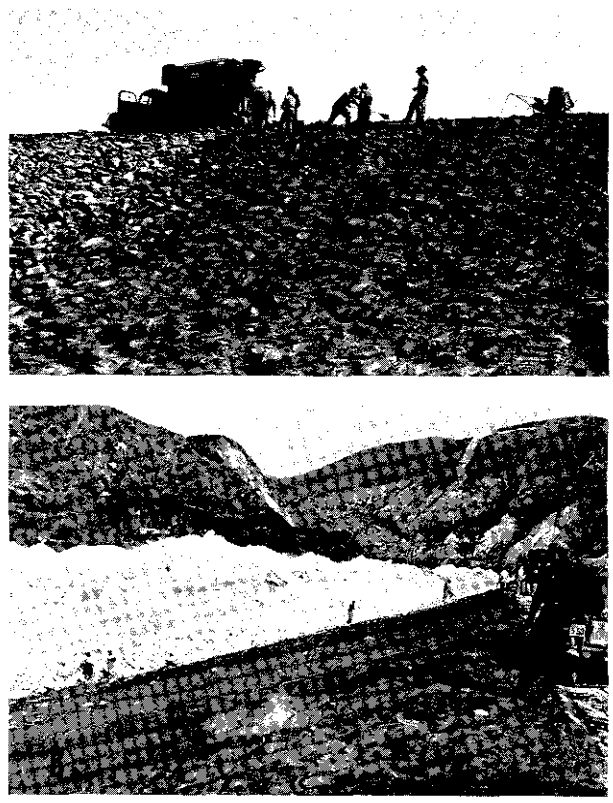


FIGURE 16.— Cobblestones obtained from streambed are graded and placed on levee, then grouted: *A*, Ventura River levee; *B*, San Gabriel River levee. Note character of streambed material in toe trench excavation.

been completed upstream from Crestmore Road. Double levees were subsequently built downstream from Crestmore Road. The details of the levée design are shown on figure 18. Maximum size stone used on the levee was 400 pounds with 50 to 70 percent being 150 pounds or less. The maximum toe stone size was 2,000 pounds and 60 percent ranged between 400 and 1,000 pounds. The toe rock was placed 11.5 feet below streambed because of the anticipated

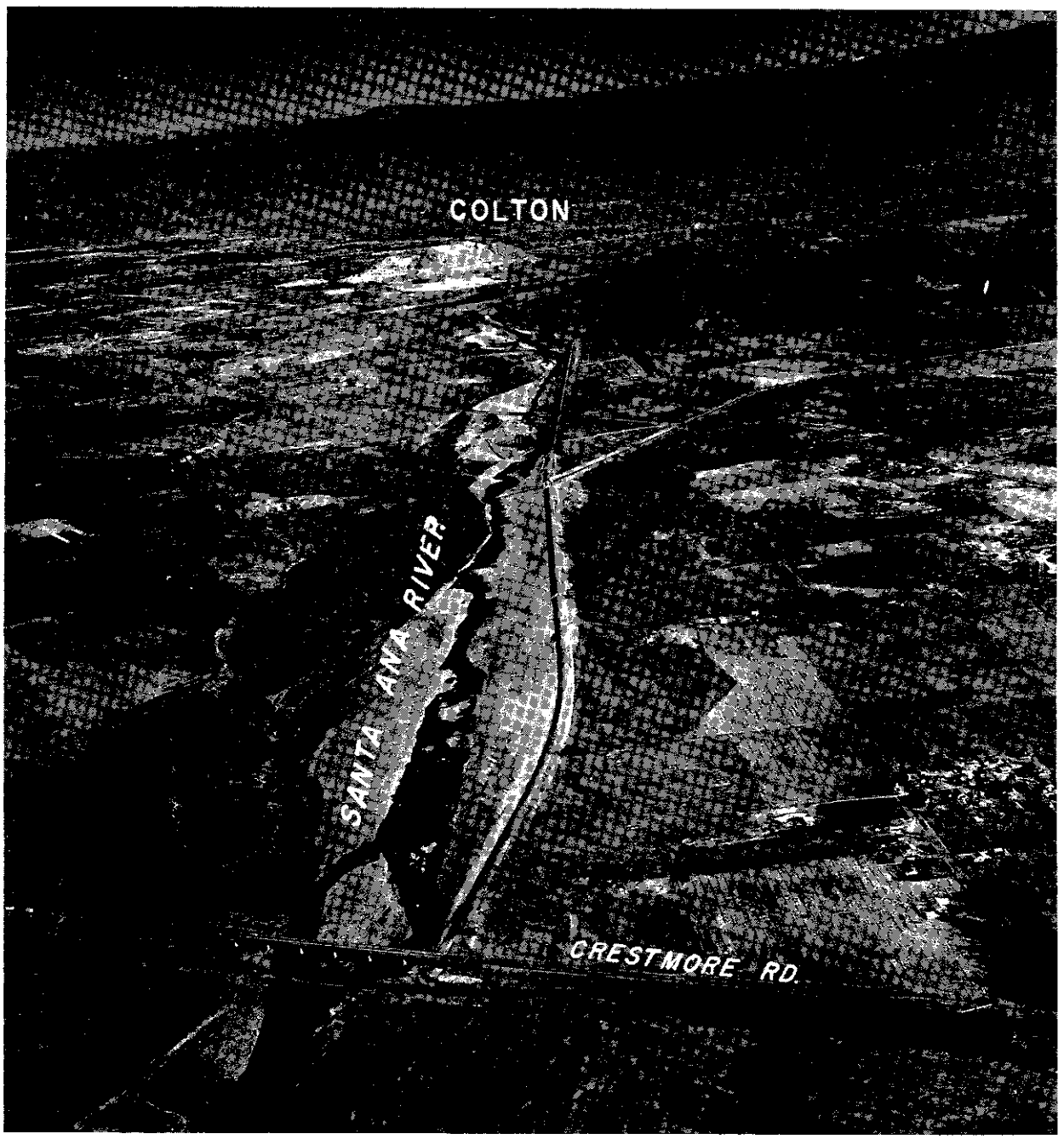


FIGURE 17. — Upstream view of Santa Ana River in Riverside County, Calif., showing partial completion of the project. Double levees were subsequently built downstream from Crestmore Road to confine a discharge of 195,000 c.f.s.

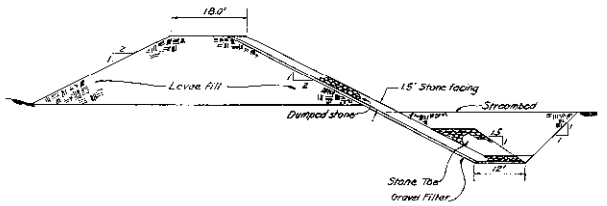


FIGURE 18.—Santa Ana River typical levee section.

scour during the design flood. Streambed material consisted of medium and fine sand and silt. The slope of the bottom of the stream was 0.0042.

On the Santa Clara River a single levee was designed and constructed to protect the coastal plain, in the vicinity of Oxnard, from floodflows having subcritical velocities of about 14 feet per

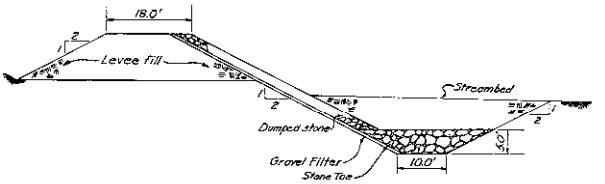


FIGURE 19.— Santa Clara River typical levee section.

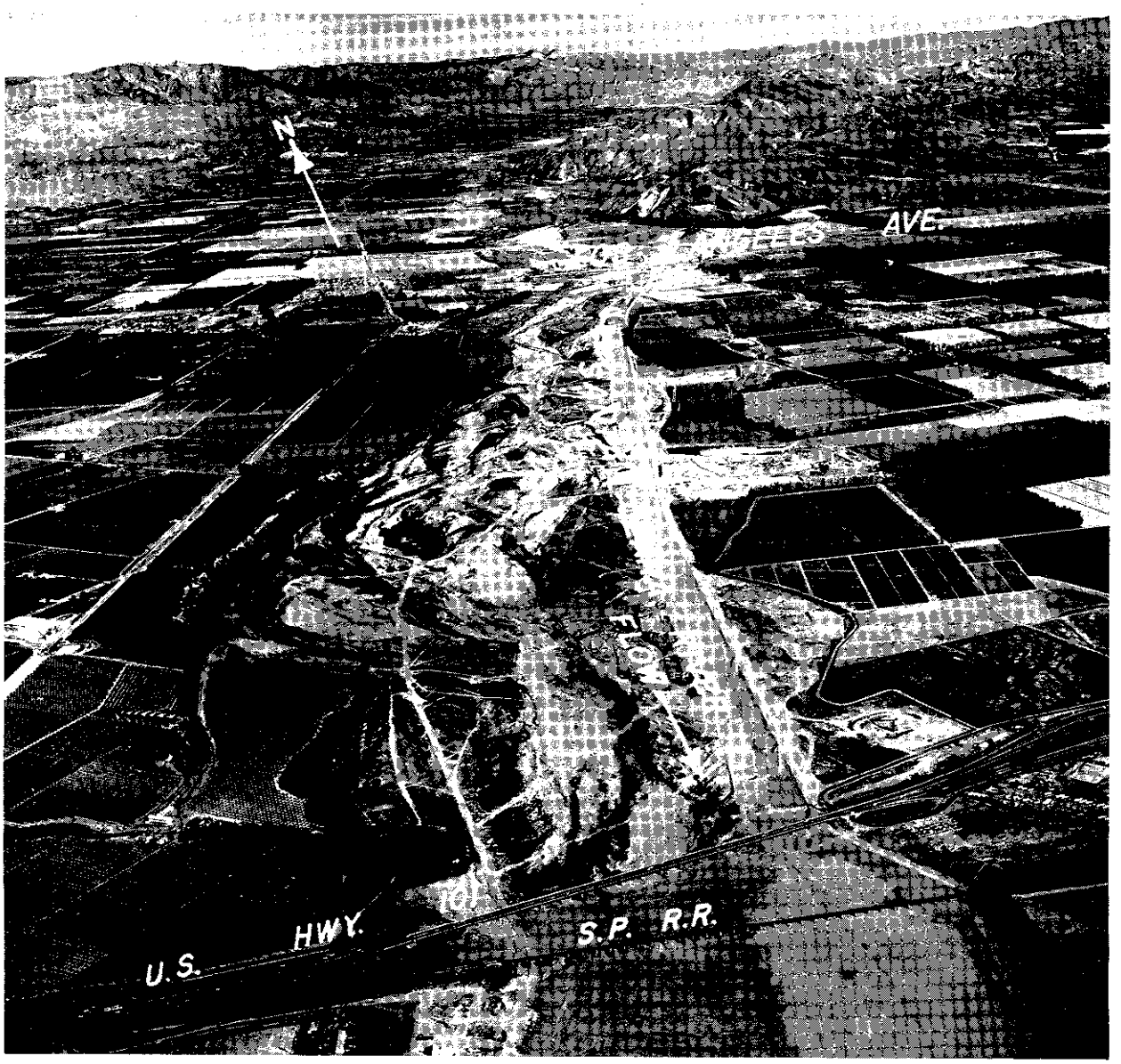


FIGURE 20.— Upstream view of Santa Clara River in Ventura County, Calif., showing a single levee designed to confine a discharge of 225,000 c.f.s.

second. The river was confined on the right side by a high natural bank. For the design flood of 225,000 c.f.s., the depth of flow would be about 15 feet. The stream gradient was 0.0030. Sand and gravel mining was being carried on in the streambed by private land owners, and it appeared that the open pits remaining would mitigate against the possibility of maintaining current directions parallel to the levee. Thus, heavy stone at the toe of the levee appeared necessary to withstand impinging attacks by the current. Maximum size stone used on the levee was 400 pounds and 50 to 70 percent were 150 pounds or less. Maximum toe stone size was 1,000 pounds and the median size was about 400 pounds. The toe stone was placed 8 feet below the thalweg of the stream. The design details of a typical levee section are shown in figure 19. An aerial view of the Santa Clara River after construction of the levee is shown in figure 20. The placement of levee stone and toe stone is shown in figure 21.

On the Santa Maria River single and double levees were built to protect the area in the vicinity of the city of Santa Maria, from flood



FIGURE 21. — Riprap revetment on Santa Clara River levee: *A*, Downstream view showing filter course and facing stone; *B*, upstream view showing placing of heavy stone in toe trench.

flows having subcritical velocities of 10 feet per second. The streambed material was medium and fine sand and silt. The design discharge was 150,000 c.f.s. and the maximum depth of flow would be about 8 feet. In this case no heavy toe stone was provided, but the depth of revetment stone was carried 10 to 12 feet below streambed because of the curvature in alinement. An aerial view of the Santa Maria River after construction of the single levee along the upper part of the project is shown in figure 22. Photographs showing the placing of the dumped stone revetment is given in figure 23. The stone had a maximum size of 300 pounds and was graded as follows:

Stone size:	*Percent smaller by weight*
300-pound	100
150-pound	60 to 90
75-pound	40 to 65
20-pound	10 to 30
5-pound	0 to 5

The design details are shown on figure 24.

Discussion and Conclusion

In order to place the revetment stone well below the streambed to protect against underscour, it is necessary to excavate a large toe trench. In practice, the excavation is usually done by dragline and the material cast onto a stockpile in the riverbed parallel to the levee. This material is later used for backfill in the toe trench and is usually not compacted. Thus, the streambed material adjacent to and over the levee toe where the greatest resistance is needed becomes the least resistant to erosion because it has been disturbed. It appears that where the streambed material is already fairly resistant to erosion, the point of diminishing returns may be reached in a deep toe excavation. Thus, judgment and experience appear to govern the design criteria rather than a theoretical approach to the solution of the question of how deep should be the levee toe.

In order to obtain field data on relative scour along a confining levee after construction, scour gages were devised to show maximum depth of scour during the peak of a flood, even though some filling might result on the recession side of the flood hydrograph. These scour gages consist of 15-foot deep holes, 12 inches in diameter, filled with colored stone granules. The horizontal and vertical location of the holes in a typical installation is shown in figure 25. Monuments were provided for location of the holes. The holes were drilled with a rotary-type auger drill. Wash water was not permitted. After drilling, the casing was removed as the stone granules were placed in the hole. The stone granules were specified as "Flamingo-Red natural stone (Roofing Granule) * * * reason-

ably well graded within the following limits: Percent by weight, passing square mesh sieve — 1½ inch size, 100 percent, ⅜ inch size, 0 percent." The magnitude of maximum scour during a flood will be determined by measuring after the flood the difference in elevation of the top of the stone granules remaining in the holes before and after the flood.

A board of consultants comprised of H. A. Einstein, L. G. Straub, and V. A. Vanoni was retained by the Los Angeles District of the Corps of Engineers to advise on criteria to be used in design of riprap levee revetment. Although the consultants were concerned primarily with details of design in each specific instance, some of the conclusions might be generalized: To obtain maximum effectiveness, riprap should consist of a dense mass of well graded, well interlocked angular stone rather than large individual particles. A thick layer of well graded small stone is better than a thin layer of large stone having correspondingly

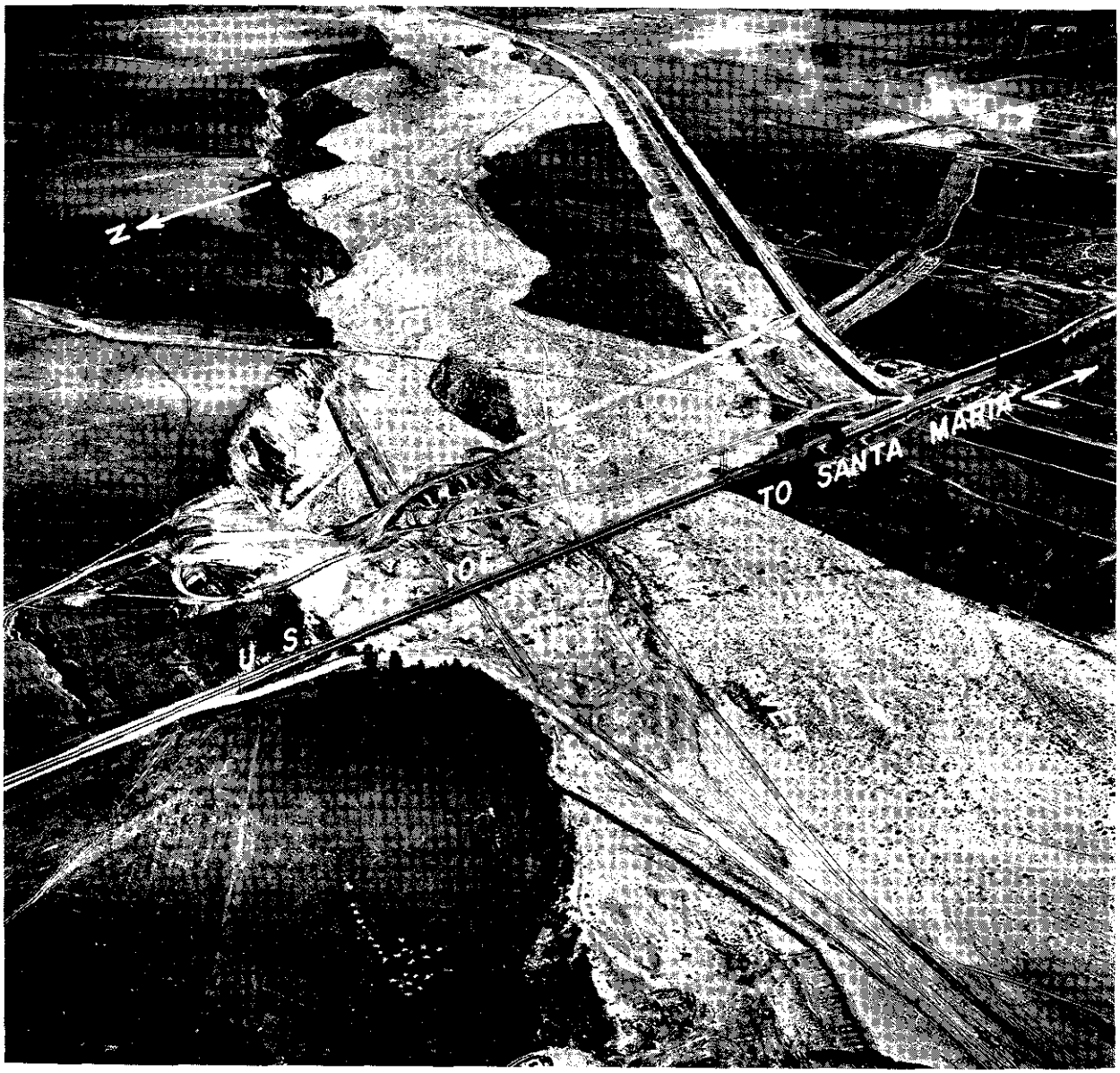


FIGURE 22. — Upstream of Santa Maria River in Santa Barbara County, Calif., showing partial completion of the project. Right and left bank levees were subsequently built downstream from U.S. Highway 101 to contain 150,000 c.f.s.

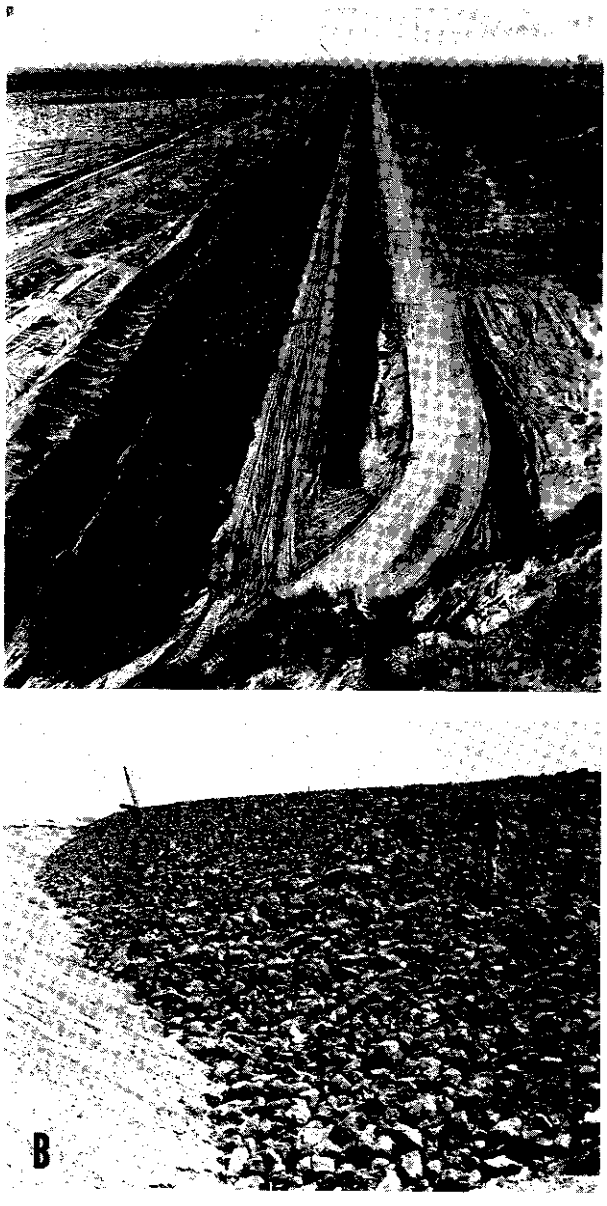


FIGURE 23. — Downstream views of levee construction on Santa Maria River: *A*, General view showing toe trench excavation; *B*, closeup view of riprap revetment.

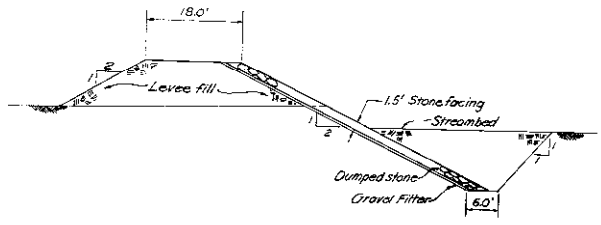


FIGURE 24. — Santa Maria River typical levee section.

large voids. Pressure fluctuations acting in the voids can lift large particles out of place and allow the single piece to be carried away. Curvature in the levee alinement induces attack along the toe on the outside of the curve. The radius of the curve should be made as long as practicable even at the sacrifice of expensive right-of-way. The toe of the levee revetment should be 6 to 12 feet below the thalweg of the stream, depending on character of material and hydraulic characteristics of the streamflow. If a meander provides a berm of 200-ft. width of undisturbed bank material between stream and levee, the depth of the toe may be reduced.

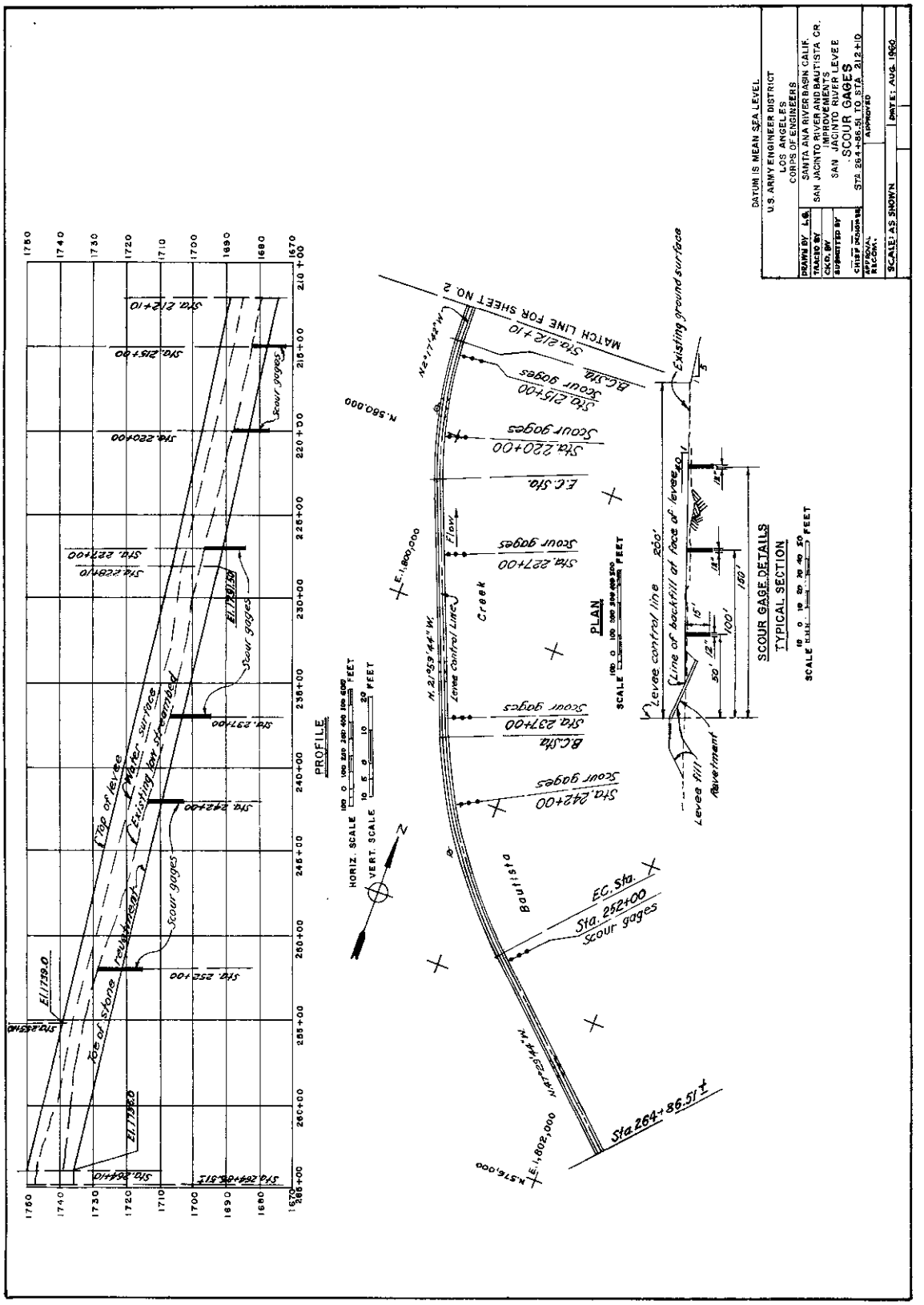


FIGURE 25. — San Jacinto River levee scour gages.

HYDRAULIC MODELS FOR STUDY OF RIVER SEDIMENTATION PROBLEMS

[Paper No. 58]

By JOHN J. FRANCO, *chief, Waterways Branch, Hydraulics Division, U.S. Army Engineers Waterways Experiment Station, Vicksburg*

Abstract

The paper reviews the state of knowledge concerning the hydraulics of natural streams and related sedimentation problems. It points out the difficulty encountered by the river engineer in the solution of sedimentation problems by analytical means and the reasons why the development of basic law and principles from experience in the field has not been practical.

The opinion is expressed that the advancement of the principles involved in river engineering and the solution of many sedimentation problems will have to depend for the most part on laboratory studies correlated with results of field investigations. The types of studies required are mentioned and the types of models used for these studies are described. One section of the paper is devoted to the design, verification and operation of the movable-bed model.

Movable-bed model studies in connection with the canalization of the Arkansas River and some of the results which could be applicable to alluvial rivers in general are discussed. The paper also describes an investigation in progress to determine the effects of water temperature on the movement of bed material and bed formations and includes preliminary results indicated by the study so far.

Developments within natural streams involving sedimentation are affected by many factors, some of which are interrelated and extremely complex. The struggle toward the regulation, stabilization, and improvement of natural streams for water supply, soil conservation, navigation, and flood control has been going on for centuries in all parts of the world. In spite of the vast effort and experience, few basic principles have been developed which can be used by the engineer in the design of river structures, and little is known as to why some structures are effective and some are not. The evaluation of conditions in the field is extremely difficult, because of the variation in geometry from one reach to the next, the variation in flow conditions, and the impracticability of comparing the effectiveness of various types and layouts of structures under the same conditions. Efforts have been made to obtain data on the performance of certain structures in the field, but very little of the available data have been evaluated and documented. In most cases, the data obtained have not been in sufficient detail to permit a reasonable evaluation of conditions before and after the installation of the improvement works.

Investigations of Sediment Problems

The study of the mechanics of sedimentation has been the object of many research projects. Much information is available covering the works of various experimenters, and the evaluations of the results of such studies have led to the development of several sedimentation formulas. Since these formulas were derived mostly from flume studies involving two-dimensional flow, they can be applied only in a general way to the solution of problems in natural streams. There is need for more research involving the factors affecting the movement of sediment, but the ultimate solutions of many practical problems will require research in the field and research with models of natural streams reproducing three-dimensional flow.

Hydraulic models have led to the development of some of the basic factors affecting the movement of sediment and the meandering of alluvial streams. Model studies have also pointed out some of the characteristics of structures affecting their performance in alluvial streams and have led to a better understanding of some of the processes involved in the development of channel configurations. However, the knowledge of the hydraulics of natural streams and of the sedimentation processes is not sufficient to permit the solution of many problems by analytical means with any degree of confidence. It is believed that the advancement of the science of river engineering, the optimum solution to many sedimentation problems, and the answers to many questions regarding the performance and effectiveness of various types and arrangements of structures will have to be based for the most part on the results of model studies correlated with results of field investigations.

Field investigations to be of value in the solution of sedimentation problems should be planned for that purpose and should be one of the major objectives of the project. When construction is involved, the investigation should be started in sufficient time to permit an evaluation of developments in the reach with low and high flows and should be continued during and after construction until the effects of the works involved have been definitely established.

Hydraulic Models

Hydraulic models used for the study of sedimentation problems are either of the fixed-bed type or of the movable-bed type. The study of problems in the fixed-bed type of model is based on the direction of movement of lightweight, plastic material or dye and current direction and velocity measurements. This type of study has certain advantages in that such models can be designed to obtain hydraulic similitude, are easier to adjust and less expensive to construct and operate, and results can be obtained within a comparatively short time. However, fixed-bed models are generally suitable only for the study of local problems in relatively stable streams involving the movement of small quantities of sediment.

The model generally used for the study of problems in alluvial streams is of the movable-bed type having fixed banks and overbank with the bed molded of a granular material such as sand, crushed coal, granulated plastic, etc. Erodible banks have been used to reproduce sections of banks where active caving is present in the prototype stream or where bank caving can be expected from the installation of the improvement plans to be tested; however, the use of erodible banks is usually not considered practicable because of the variations in the erodibility and other characteristics of the materials forming the banks of the natural stream and the difficulty of reproducing the conditions in the model with any degree of accuracy even if enough prototype data were available. When bank caving is a factor in the development of the stream, its effect can usually be simulated artificially in the model without compromising the ultimate results.

Movable-Bed Models

Design

The design of a movable-bed model must consider the characteristics of the stream involved, the problem or problems to be investigated, and the requirement that the material forming the channel bed be moved in simulation of the sediment movement in the prototype. Other factors that also affect the design of the model are space and facilities available and cost and time limitations. The scales selected for movable-bed models are generally too small to provide hydraulic forces sufficient to move material of a practical size and specific weight. The forces required to obtain general movement of the bed material are obtained by distortion of the linear scales or by use of supplementary slope. Distortion that involves the use of a vertical scale larger than the horizontal scale provides greater model depths and slope. Supplementary slope provides a tilt to the model in addition to that resulting from the linear scales, thus increasing the total model slope by a fixed amount. Distortion of the linear scales and supplementary slope tend to affect the relationship of velocity, width-depth ratio of the channel and curvature, and, consequently, the distribution of energy within the channel. Also, distortion produces changes in the size and shape, which in turn affect the performance of certain types of structures. Because of these effects, distortion and supplementary slope should be maintained as small as feasible. The effect of distortion on the performance of some structures such as sills or stilling basins can usually be eliminated by using the vertical scale to reproduce certain horizontal dimensions, particularly the dimensions in the direction of flow.

Verification

Complete similarity between a movable-bed model and its prototype can seldom be obtained, because of inherent distortions that are incorporated during model design and the impracticability of reducing the grain size of the bed material to the scale of the model. Because of the dissimilarities mentioned, the degree of similitude attainable in these models cannot be established by mathematical analysis. The reliability of a movable-bed model is usually based on the model verification. The normal verification is an intricate process of adjusting the various hydraulic forces and model operating technique until the model demonstrates its ability to reproduce with acceptable accuracy the changes in bed formations that are known to have occurred in the prototype during a given period.

The successful verification of the model is used to establish the time and discharge scales, the rate of introducing bed material for each flow reproduced, model operating technique, and the degree of similitude that can be expected. The verification involves considerable time and effort, careful observation and analysis of developments, and the exercise of engineering judgment in the evaluation of the factors affecting development in the prototype and model. The degree of similitude attainable and the time required for model verification depend upon the amount and type of prototype data available, the scales of the model, experience with models of this type, and the refinements to which the adjustment of the model is carried.

Test Procedure

After the model has demonstrated its ability to reproduce conditions similar to those that occurred in the prototype and before the tests of various plans are undertaken, a "base test" is conducted. For this test the model is usually operated by reproducing typical, or average,

annual stage and discharge hydrographs until developments within the model channel resulting from the flow conditions reproduced have become stable. Improvement plans are tested with the same hydrograph used in the base test, and the results of the latter form the basis for determining the relative effectiveness of the various plans. However, under certain conditions, tests might also be conducted by reproducing a hydrograph including an unusually low-water or high-water period.

Models Used in Study of Arkansas River Problems

Hydraulic models of the movable-bed type have been concerned mostly with the investigation of specific problems, and the scopes of these studies generally have not been sufficiently broad to permit the development of basic information applicable to streams in general. Some of the more versatile types of movable-bed model studies have been concerned with problems in connection with the canalization of the Arkansas River. Studies completed or in progress are concerned with such problems as design and location of lock and dam structures, effects of the structures on the movement of sediment, development of channel depths and alinement, radii of curvature of bends, effectiveness of various types and arrangements of regulating structures, and navigation conditions in the approaches to the locks.

The models used for the study of problems in the Arkansas River have been constructed to fairly large scales to permit studies to be made in greater detail and to facilitate the study of the varied problems involved. The horizontal scales used in these models have ranged from 1:120 to 1:250, with vertical scales ranging from 1:36 to 1:80; distortion of the linear scales (vertical scale divided by the horizontal scale), has ranged from 1.5 to 4.17. The time scales developed for these models were such that one day of flow in the prototype was reproduced in from 5 to 10 minutes; the reproduction of an annual hydrograph would therefore require from 30.4 to 60.8 hours of operation. The test of a modification usually requires several reproductions of the annual hydrograph, depending upon the extent of change produced by the modification, before definite indications are established. Since the time required to test a given plan is appreciable and the time available before the information is usually needed by the design engineer is limited, these studies of necessity have to be confined generally to the development of a practical solution rather than to the development of a refined optimum solution.

The results of studies in connection with the canalization of the Arkansas River have brought out some information that should be applicable to the regulation and improvement of natural streams in general. These studies have shown how the use of spur dikes, particularly on the concave side of a bend, can produce considerable disturbance to flow, reduce channel efficiency (increase roughness), and could produce shoaling upstream and downstream. It has been demonstrated that dredging of the channel bed without other changes would produce only a temporary effect, regardless of the size of the cut. The studies have emphasized the importance of the lower reach of a bendway on developments downstream. The depth and alinement of the channel over a crossing can be improved, and the depth of scour and current attack along the revetted bank in a bend can be reduced by proper design of the lower reach of the bend upstream.

Effect of Water Temperature

It has long been noted that the movement of bed material in movable-bed models can be affected by the temperature of the water. With materials of a certain grain size, riffles would form with cold water and not form with water at a higher temperature. It was also noted that the temperature at which a given material would riffle could be lowered by increasing the median-grain size of the material. It has been suspected from the above that the differences in the stage-discharge relation observed in alluvial streams might be caused by the difference in the temperature of the water. Research investigations are in progress to determine the effect of water temperature on the movement of bed material, bed formations, and channel efficiency.

This research is being conducted in a tilting flume equipped with a water circulating system, water temperature control, automatic bed-material feeder and a device that automatically records the weight of the material extruded. Tests are being conducted with a constant discharge, rate of bed feed, and tailwater elevation for each of three water temperatures (40°, 60°, and 80° F.). The flume is operated until the rate of material extruded and the water-surface slope have become stabilized for each test condition. The first series of tests has been conducted with a fairly uniform sand having a median grain diameter of about 0.2 mm. A considerable amount of data was obtained during these tests, but analysis of the results has not progressed sufficiently to provide definite conclusions. Preliminary plots of some of the data obtained show an increase in the energy gradient with a decrease in the water temperature for the same rate of bed movement (fig. 1). This could be an indication that the carrying

capacity of the water decreases with temperature or that the formations (riffles) along the bed are changed so as to increase the roughness of the bed as the temperature of the water is lowered (fig. 2).

The material used in the first series of tests riffled at all three of the water temperatures tested. It is planned to conduct the next series of tests with a material that does not riffle at those temperatures. A comparison of the results of the two series of tests might provide some of the answers.

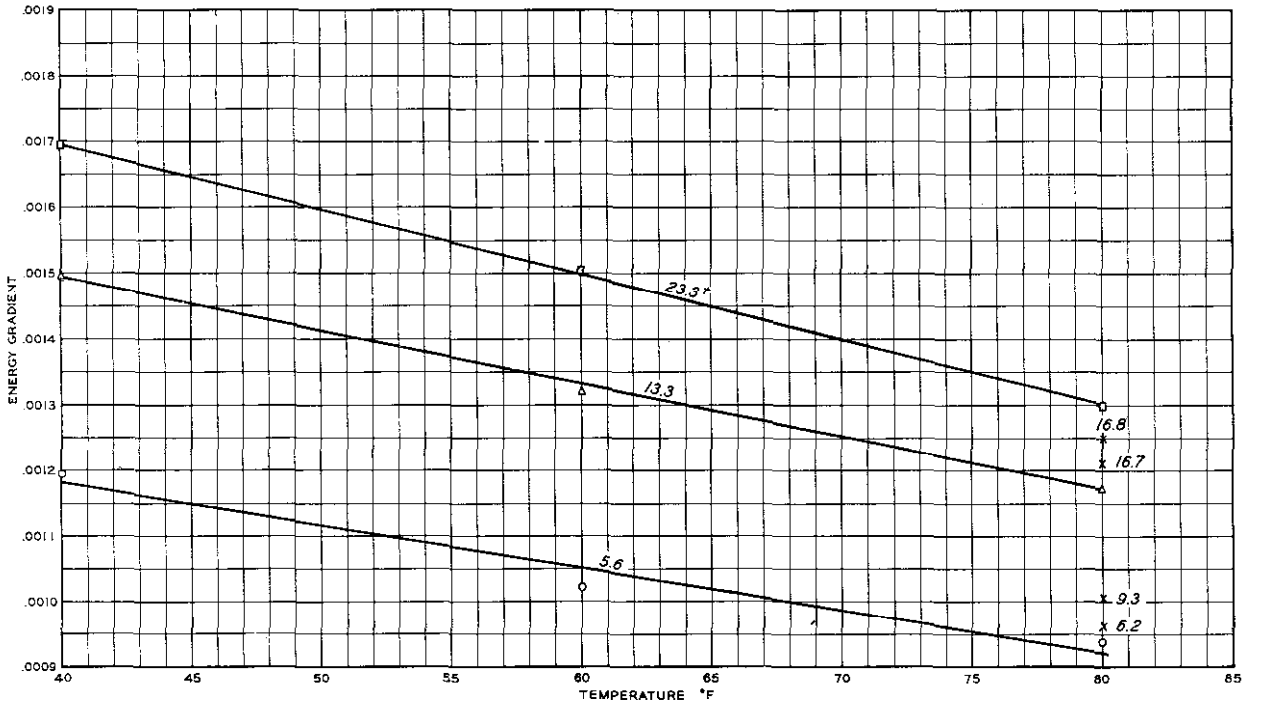


FIGURE 1. — Energy gradient-temperature curves; fine sand bed material.

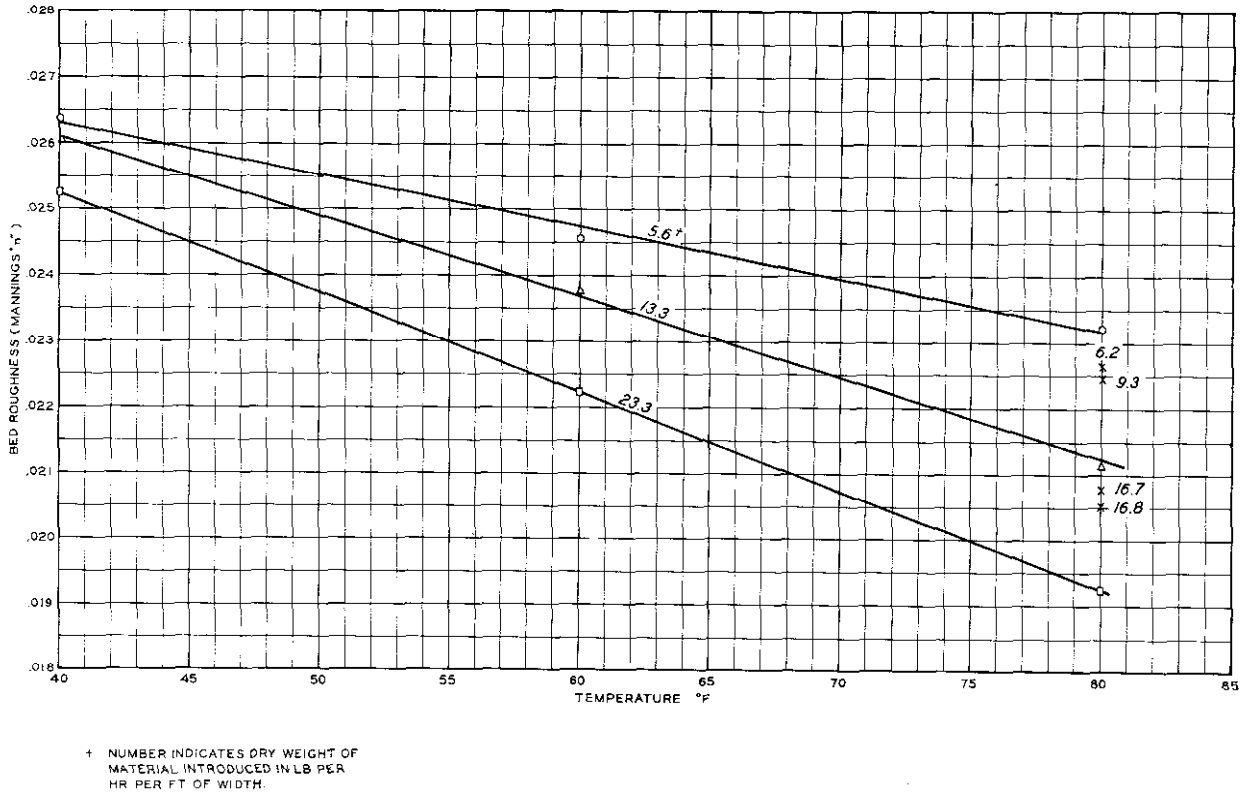


FIGURE 2. — Bed roughness-temperature curves; fine sand bed material.

LOWER HUDSON RIVER SILTATION

[Paper No. 59]

By FRANK L. PANUZIO, *assistant chief, Engineering Division for Civil Works, U.S. Army Engineer District, New York*

Synopsis

Siltation of the navigable channel and adjacent slips of the Lower Hudson River, a tidal estuary of the Port of New York, is a major item of local and Federal maintenance and a deterrent to the full use of the river frontage for maritime purposes.

To obtain a solution, analysis of field prototype data is being augmented by studies on two complementary models that integrate the nonanalytical complex factors and influences involved in the hydraulic circulation of fresh and salt water and in the siltation processes of this tidal estuary. A comprehensive model to study hydraulic circulation and channel shoaling reproduces 100 miles of the New York Harbor and the Hudson River from the Atlantic Ocean to Poughkeepsie, N. Y. An enlarged sectional model to study slip shoaling reproduces 9 miles of the Lower Hudson River from 1 mile upstream of the Battery to 1 mile downstream of George Washington Bridge.

Fresh and salt water circulation and siltation processes within this estuary; variation of shoaling quantities and pattern before and after a change in geometry at a critical point of the estuary; cross sectional distribution of velocity, salinity, and suspended sediment flow; and comparative model results of possible solutions involving sediment traps, dikes and barriers, and changes affecting channel geometry or hydraulic circulation and salinity are discussed.

The final solution to the problem must await the results of further model tests and final evaluation of hydraulic and economic factors.

Introduction

The lower Hudson River is a principal

navigable estuary of the Port of New York and the entrance to the Hudson River channel to Albany, N. Y. Heavy shoaling is occurring in the 11 miles of Federal channel and in 250 adjacent slips. In addition to the Federal cost of annually dredging about 1,600,000 cubic yards of sediment from the navigable channel, local interests incur the annual cost of removing about 3,000,000 cubic yards of sediment from the slips. Because of this heavy local cost, full use is not being made of the river frontage for maritime purposes. So, in July 1959, at the request of local interests, the Public Works Committee of the House of Representatives authorized a study, currently underway, to determine what improvements are necessary and feasible to lessen shoaling in the pier slips and in the Federal channel.

Even though this shoaling problem is unusually severe, it is typical in that all partially mixed estuaries have similar shoaling problems where fresh water runoff contacts salt water, causing flocculation and settlement of fine materials in suspension. The problem is complex because the interplay of fluvial and tidal currents in the lower Hudson River defy mathematical analysis, resolution, and expression. To solve the problem, field measurements and studies are being supplemented by measurements and studies on two hydraulic models that integrate effectively the many forces, factors, and influences involved in the problem.

The Hudson River

The roughly cross-shaped watershed of the Hudson River (fig. 1) extends 120 miles east to west and 236 miles north to south. This basin (fig. 2) drains 13,400 square miles of the northern and eastern parts of New York with small areas in Vermont, Massachusetts, Connecticut, and New Jersey. From its source in the Adirondack Mountains, the river flows generally south to its junction with its principal tributary, the Mohawk River. The main river continues to flow in a southerly direction for 155 miles to its mouth at the Battery, where it empties into the Upper New York Bay.

In its lower reach (fig. 2), the Hudson River has been improved for 11 miles for navigation, with depths varying from 30 to 48 feet below mean low water from the Battery to a point downstream of George Washington Bridge. Upstream of this point, for 143 miles to Albany, the navigable depth is 27 feet and work is underway to deepen it to 32 feet below mean low water. Upstream of Albany the rivers have been canalized as a part of the New York State Barge Canal System. North of Albany 49 miles of the Hudson River have been canalized with depths of 12 to 14 feet to connect with the nar-

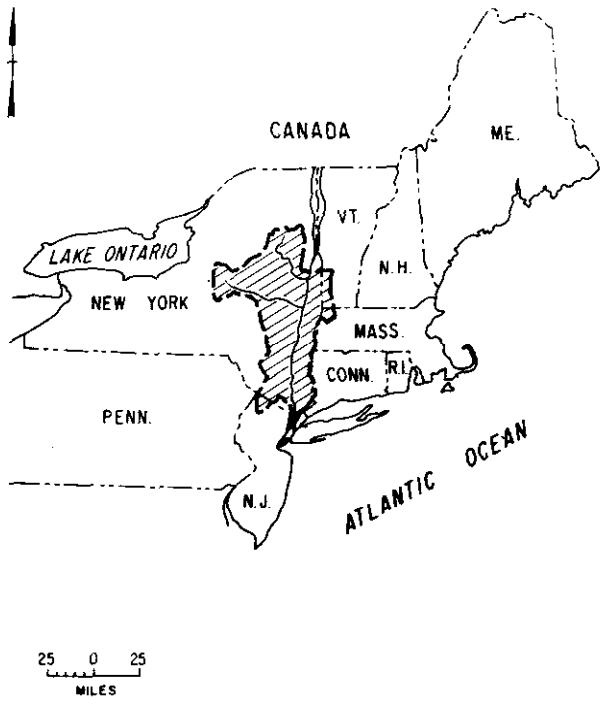


FIGURE 1. — Watershed of Hudson River.

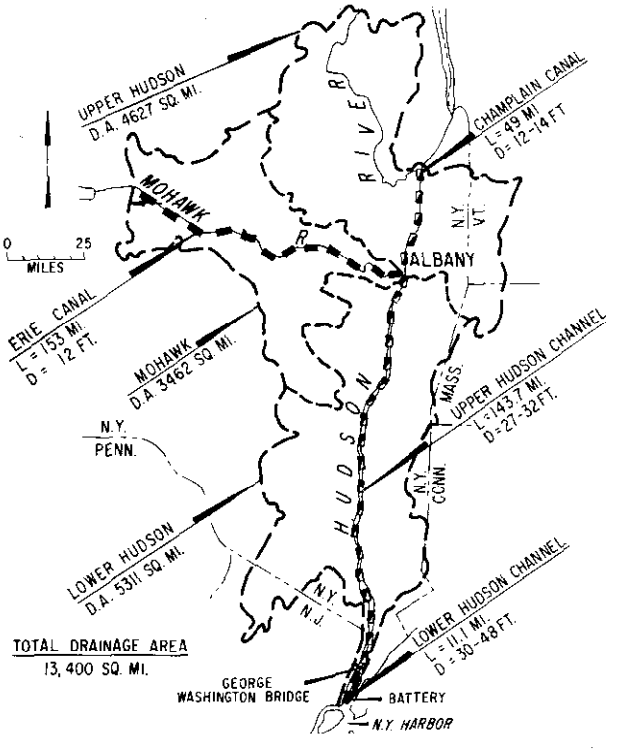


FIGURE 2. — Characteristics of Hudson River basin.

rows of Lake Champlain. West of Albany, 153 miles of the Mohawk River have been canalized to a depth of 14 feet to connect with Lakes Erie and Ontario.

The average annual precipitation over the basin is 43 inches, of which 24 inches appears as annual runoff at the mouth to produce an average fresh water runoff of 23,000 cubic feet per second.

The tidewater in the Hudson River extends 154 miles from its mouth at the Battery to the Federal Lock and Dam at Troy, 10 miles upstream of Albany, N.Y. The mean tidal range decreased from 4.4 feet at the Battery to a minimum of 3.0 feet at Beacon, N.Y., 62 miles upstream of the Battery, and then increases to 4.8 feet at Troy.

Basic Hydraulic Mechanism for Sedimentation

The New York Harbor, from the viewpoint of circulation, represents probably the most complex major estuarine system in the United States. The principal hydraulic components (fig. 3) are Lower New York Bay, Sandy Hook Bay, Raritan Bay, Newark Bay, Arthur Kill, Kill Van Kull, Hudson River, East River, Harlem River, the lower reaches of Hackensack and Passaic Rivers, the Atlantic Ocean, and Long Island Sound.

Most of the fresh water entering the system is contributed by the Hudson River (13,400 sq. mi.) with some contribution from the Raritan, Hackensack, and Passaic Rivers (2,300 sq. mi.) that for short periods may be significant. The tides in the principal components are controlled primarily by the ocean tide entering directly into the Lower Bay, and indirectly through the Long Island Sound into the East River (fig. 3). The degree of mixing that influences the circulation pattern varies from highly stratified to well mixed, depending on the magnitude of the tidal flows as compared to the fresh waterflows.

Studies of the New York Harbor in nature and models revealed many valuable aspects of circulation.[1]

The plan of circulation in the New York Harbor is extremely complicated. The Hudson River, upstream of the Battery, is a system in which the inertia of a long column of moving water must be overcome by the rising or falling

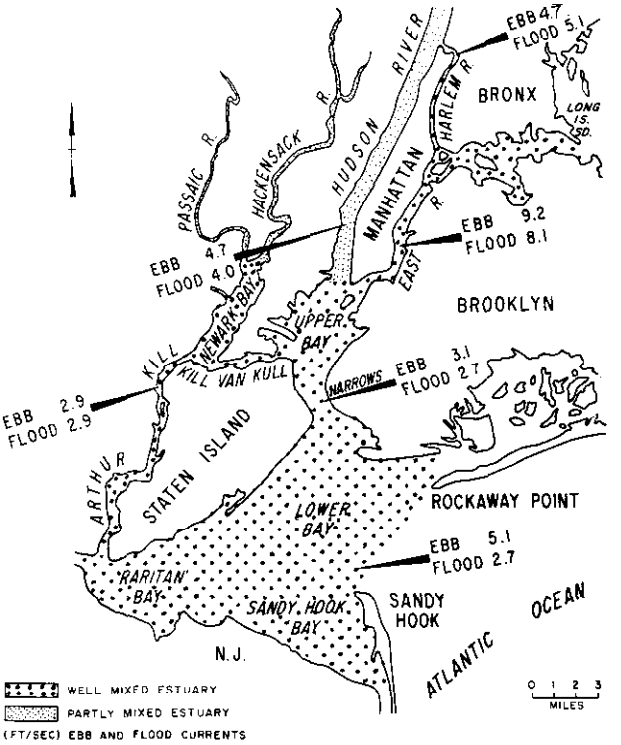


FIGURE 3. — New York Harbor estuarine system.

tide head before slack current can be effected. This is best demonstrated by the current pattern 2 hours after low water at the Battery. At this time (fig. 3) the Hudson River is ebbing at the surface and receiving flood discharge of the Upper New York Bay at the bottom; the East River is receiving ebb discharge of the Hudson River at the surface and flood discharge of the Upper New York Bay at the bottom; Newark Bay is receiving ebb discharge of Hudson River and flood discharge of the Upper New York Bay, and the upper New York Bay is receiving flood discharges of the Lower Bay through the Narrows.

This difference in phasing of currents to some extent affects the sedimentation pattern in the Lower Hudson River. However, it has been found from model observations of the basic flow mechanism operating in the Harbor, that the Hudson River is to some extent insulated from the rest of New York Harbor as far as receiving sediments is concerned. The fresh waterflow of the Hudson River constitutes a sufficiently large percentage of its total flow to be important in the establishment of density currents. However, in the Lower Bay, Raritan and Sandy Hook Bays, and the Upper New York Bay, the fresh water discharges represent a small percentage of the total tidal discharge, so

[1] DUKE, CHARLES M. SHOALING IN THE LOWER HUDSON RIVER. Amer. Soc. Civil Engin. 2730. February 1961.

FORTSON, E. P. JR., and SIMMONS, HENRY B. SOME ASPECTS OF CIRCULATION REVEALED BY THE NEW YORK HARBOR MODEL. Amer. Soc. Civil Engin. meeting. 12 pp. October 1959.

NEW YORK HARBOR COMMITTEE ON TIDAL HYDRAULIC. REVIEW OF SHOALING PROBLEMS IN HUDSON RIVER. U.S. Army Corps of Engineers. 24 pages. July 1961.

that the water is well mixed vertically. Thus, the density currents in Upper New York Bay are relatively weaker than those in the Hudson River proper. Therefore, it is improbable, though not impossible, that any considerable source of shoaling to the Hudson River could be located below the Battery.

The difference in phasing of the currents in East River and Hudson River causes the ebb flow in the East River to flow upstream into the Hudson River for about 1½ hours before the Hudson reaches flood slack and begins to ebb. This could make it possible for the East River to be a source of shoaling material for the Hudson River.

The part of the Hudson River within the New York Harbor area from the Battery to about Haverstraw Bay is normally classified as a partly mixed estuary (figs. 3 and 4) Tidal currents go through a complete reversal from ebb

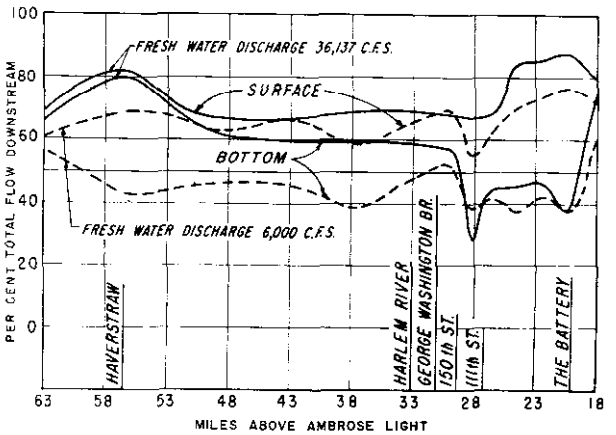


FIGURE 4. — Flow predominance at surface and bottom of Hudson River within the New York Harbor.

to flood at all depths throughout this reach, but vertical mixing is not complete. For favorable conditions of location, fresh water flow, and phase of tide, salinities at the bottom may be as much as 10 parts per thousand greater than at the surface (fig. 4). This large salinity gradient causes the reversal of the current from ebb to flood to occur by as much as 2 hours earlier at the bottom than at the surface, so that the strengths and durations of the bottom flood currents are greater than those of the bottom ebb currents. As a result, more water flows upstream than downstream at the bottom; and, conversely, more water flows downstream than upstream at the surface. In general, the salinity-induced upstream flow predominates over the downstream flow in about the lower half of the stream, and the fresh-water induced downstream flow predominates over the upstream flow in the upper half.

The magnitude and extent of the upstream predominant bottom currents vary with the amount of fresh water flow in the Hudson River (fig. 4). At high flows these currents are increased in magnitude but reduced in extent, so that the upstream limit is downstream from the George Washington Bridge. At medium and low fresh waterflows, they extend well upstream of the George Washington Bridge, but are interrupted locally just downstream of the bridge.

The interruption of the upstream predominant bottom currents during medium and low fresh waterflows in the Hudson River is due to a combination of two conditions that are considered to effect sedimentation significantly in the Lower Hudson River.

One condition is the abrupt change in cross-sectional area in the vicinity of the George Washington Bridge (fig. 5). The constricted

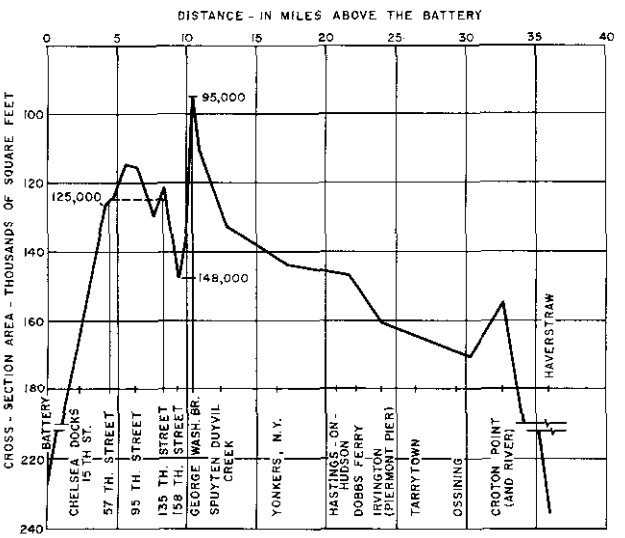


FIGURE 5. — Lower Hudson River: cross sectional areas at mean low water.

cross-sectional area near the bridge causes an intense turbulence that effectively interrupts the upstream predominant bottom currents but still permits the movement upstream of saline water that reforms the upstream predominant bottom currents upstream of the George Washington Bridge (fig. 4). The excessive cross-sectional area occurring downstream of the bridge where the bottom flood predominance is either enfeebled or eliminated, depending on the fresh water discharge, tends to receive sediment that otherwise might be induced to proceed farther upstream before depositing.

The other condition is that the discharge of large quantities of heavy saline water from the Harlem River into the Hudson River upstream of George Washington Bridge (figs. 3 and 4) during certain phases of the tide reduces the vertical salinity gradient and interferes with the upstream bottom predominance.

Thus, there are two separate and independent reaches in the Hudson River where upstream predominant bottom currents exist during medium and low flows, one upstream and the other downstream of the George Washington Bridge.

The movement of silt within the lower Hudson River is related to and controlled by the vertical distribution and pattern of currents as herein described. Thus, suspended silt entering the area is first carried downstream in suspension in the upper fresh water, predominantly downstream layers, then gradually dropped to the bottom layers through flocculation and settling processes. Material reaching the bottom layer gradually moves upstream rather than downstream until it is deposited in areas where the bottom current velocities are too low to move material or where the net movement of bottom currents is zero. More specifically, from the downstream end of the shoal areas (fig. 6) material is moved upstream along the navigation channels to West 59th Street (see fig. 20). From this point, the material moves upstream in the naturally deeper channel along the New York side.

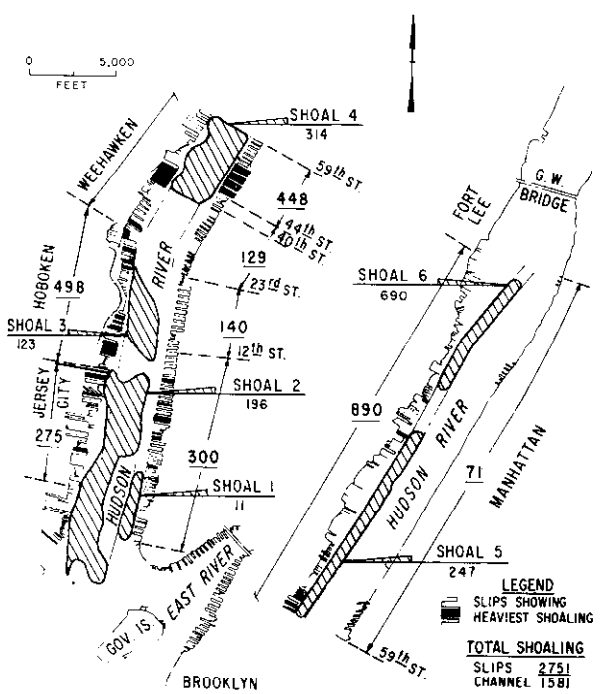


FIGURE 6. — Slip and channel shoaling (thousands of cubic yards), Hudson River.

As heretofore noted, the upstream predominance varies with depth and, in the region from 59th Street to George Washington Bridge, is confined to the east (New York) side of the Hudson with natural depths of 40 to 50 feet as compared to 30-foot depth on the west (New Jersey) side. When the material reaches the George Washington Bridge, the turbulence throws the material into the surface layers and toward the New Jersey side. Then the material is deposited into shoal areas by ebb currents moving downstream in the 30-foot channel on the New Jersey side in which there is no predominant upstream current.

More simply, a significant part of the shoal material in the Lower Hudson River moves intermittently but progressively in a counterclockwise direction, generally upstream along the New York side to the vicinity of George Washington Bridge, then downstream along the New Jersey side.

Some material is carried upstream on floodtides through the George Washington Bridge, after which it reenters the upstream predominant bottom currents of low and medium freshwater flows and moves in naturally deep water upstream of the bridge until deposited in the shoals in vicinity of Haverstraw Bay, which is also fed by upstream materials.

The shoaling of the slips is related to this sedimentation mechanism that is responsible for the presence of shoaling materials that permeate the waters in the lower strata of the harbor. The shoaling of the slips is due to a combination of causes. The current velocities associated with filling and emptying the individual tidal prisms of the slips are very slight, but the settling rate of the suspended load is slow and small velocities are competent to carry the material. The flooding of the slips is effected principally from the bottom currents of the main stream, the greater part of it entering in the first 2 to 3 feet above the bottom. Therefore, not only is the velocity greater because it occupies a small cross section, but it occurs at a level at which the concentration of suspended load is greatest, so that large amounts of sediment are carried into slips on every floodtide. Conversely, the emptying of the slip prisms involves approximately equal flow throughout the slip cross section, and the outward moving bottom velocities from the slips during ebbtides do not have sufficient velocity to remove the silt that has come into the slips during floodtide. The net result is that silt enters the slips during each floodtide and very little is moved out on the following ebbtide.

Nature and Extent of Sedimentation

In order to understand better the factors entering into the sedimentation processes and to obtain data required to construct and to operate a model that would reproduce the basic natural sedimentation mechanism, studies of past records and measurements were made of shoaling pattern and amounts, source of sediment, and vertical distribution of salinity, velocity, and sediment discharge.

Channel Shoaling

The distribution of channel shoaling was developed from maintenance surveys and dredging records. The annual amount of material distributed in some six shoal areas (fig. 6) amounts to about 1,600,000 cubic yards annually in the navigation channels. The heaviest shoaling occurs in the 30-foot channel, shoals 5 and 6, where over 900,000 cubic yards of materials have been dredged annually.

Slip Shoaling

The distribution of slip shoaling was similarly developed from available incomplete records extrapolated to cover the entire area. The annual amount of slip shoaling was determined as about 2,700,000 cubic yards, of which 60 percent occurs on the New Jersey side (fig. 6). The problem slips, noted in black on figure 6, are generally located adjacent to the channel shoal areas. However, an increase in slip shoaling also varies directly with an increase in depth of slips, 15 to 18 feet are required for barges and ferries, 26 to 28 feet for cargo vessels, and 40 feet and more for passenger vessels and supertankers.

Spoil Dumping in Harbor

Dumping of dredged material in the New York Harbor during World War II may have modified the shoaling pattern and rate in the Lower Hudson River. During this time, over 53 million cubic yards of mud, clay, sand, and rock were dumped at 11 or more deep water or nonnavigable locations in the New York Harbor area. Of this amount about 21 million cubic yards were dumped in the Narrows, over 13 million cubic yards under George Washington Bridge, and over 7 million cubic yards at Governors Island. Recent surveys indicate that only a small amount of the dumped material remains, except at George Washington Bridge where over 40 percent of the dumped material, believed to be rock, still remains.

The effect on shoaling due to the change in geometry at the George Washington Bridge has been appraised, based on studies of maintenance dredging records, corrected for new work, before and after the dumping operation. Fifteen years of maintenance records before dumping (1926-40) and after dumping (1946-60) were corrected and plotted as mass curves (fig. 7).

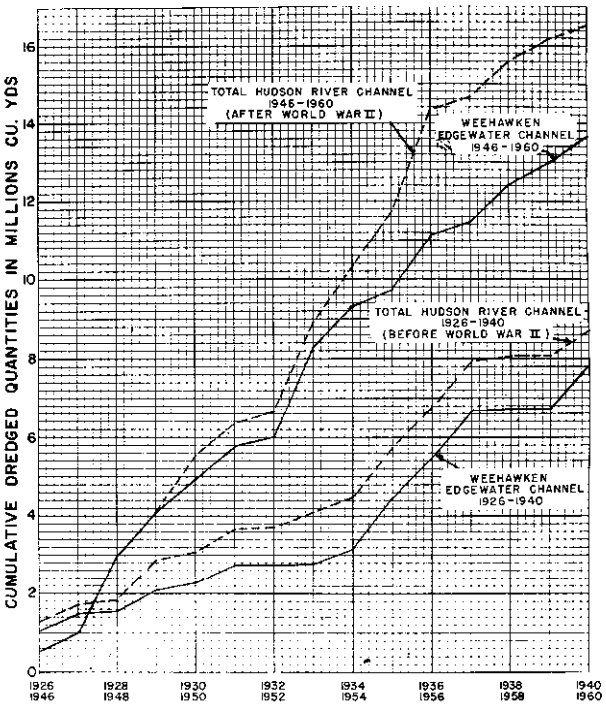


FIGURE 7.—Mass curve of dredged quantities before and after World War II spoil operations, New York Harbor.

Between 1926 and 1940, an average of 500,000 cubic yards were dredged annually for maintenance, of which 490,000 cubic yards represent the quantity dredged from the Weehawken-Edgewater channel and 90,000 cubic yards from the rest of the project. During 1946-60, the 15 years after dumping in the harbor, an average of 1,130,000 cubic yards were dredged annually for maintenance, of which about 930,000 cubic yards were in the Weehawken-Edgewater channel and 200,000 cubic yards in the rest of the project. These figures represent an increase in shoaling of about 100 percent.

Source of Shoaling Material

Even though, as previously described, the hydraulics of the harbor system results in a mechanism for moving and depositing suspended material that is independent of the source of shoal material, limited measurements of suspended sediment and an accounting of other material entering the lower Hudson River were made. The probable sources of shoaling material considered were the Hudson River watershed; local

dumping through sanitary and storm sewers, industrial, and others; other watersheds entering New York Harbor, Long Island Sound, and the Atlantic Ocean.

Sediments from Hudson River Watershed

The Hudson River watershed furnished sediment from erosion of the land areas, streambeds, and streambanks (fig. 8). The material is

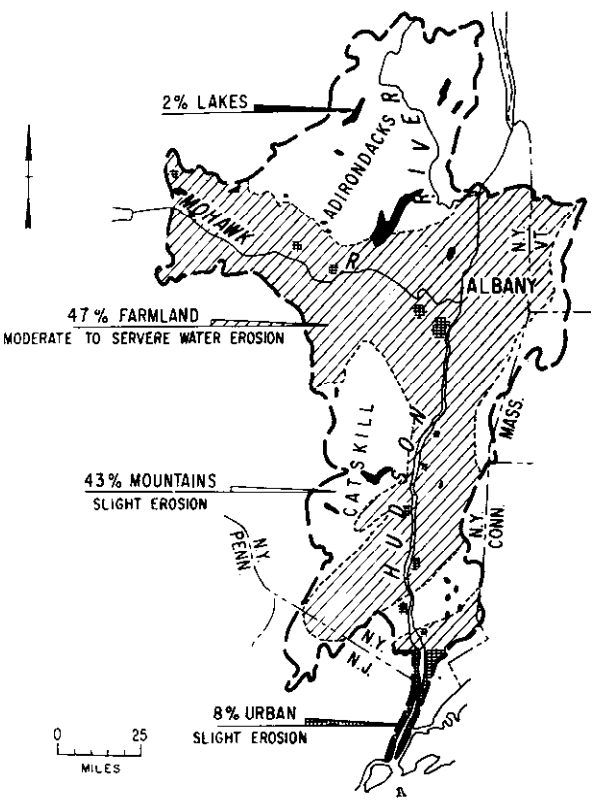


FIGURE 8. — Physiography and erosion of Hudson River watershed.

carried by the river in suspension or is moved by the current along the streambed. In this investigation, no attempt was made to measure the bedload. The suspended load was measured by means of daily water samples taken at Poughkeepsie, N. Y., which is 75 miles upstream from New York Harbor and above the limit of salt water intrusion. The Department of Public Works at Poughkeepsie cooperated in obtaining twice-daily water samples for the period 3 July 1959 through 15 September 1960. The sampling period encompassed the seasonal fluctuations of fresh water discharge and sediment concentration and provided a complete year's record of sediment discharge for comparison with the annual prototype shoaling volume in the problem area. Salinity as measured in the water samples at Poughkeepsie ranged from 8 to 35 p.p.m., averaging 15 p.p.m. for the test period.

The structure for the Poughkeepsie water intake is located at the midpoint of the river, which is about 2,000 feet wide, at depth of 54 feet below mean low water, or 5 feet above the deepest part of the cross section. Water is drawn through the manifold into a 36-inch pipe leading to a pumphouse. To insure a representative sample, a continuous discharge was maintained through an open tap where quart samples were taken and sealed in mason jars. Duplicate quart samples were obtained twice a day; one set at strength of ebb and the other at strength of flood for the period 1 September 1959 through 31 August 1960. Additional bimonthly samples were taken for chemical and mechanical analyses.

The samples of water were tested for total suspended solids by standard methods. The sediment concentration, the weight of dry solids to the weight of the sample, were recorded in parts per million. The total daily suspended sediment discharge was the product of the total fresh water discharge at Poughkeepsie in a day second-feet times the sediment concentration during ebb flow. The daily fresh waterflows were obtained from flow records of the Hudson River at Green Island, opposite Troy, N. Y., and the intermediate tributary streams, including Kinderhook, Wappinger, and Rondout Creeks. The time-interval required for fresh waterflows from Green Island, the primary gaging station, to reach Poughkeepsie was developed as 1 day. Therefore, in computing the suspended sediment discharge at Poughkeepsie from the upstream fresh waterflows a 1-day lag was applied. The suspended sediment concentration at strength of ebb current and the daily fresh waterflow at Poughkeepsie were used in computing the daily sediment load flowing to the problem area, because any sediment moving upstream during the flood cycle necessarily had to return during subsequent ebb cycles.

In order to determine the relation of the single sample intake concentration to the mean sediment concentration for the entire river cross section, sediment samples were taken at 10 various depths and locations in the cross section at the Poughkeepsie intake during the ebb flow. These samples were analyzed for sediment concentration and the results averaged 68.5 p.p.m. All the samples were taken within an hour of the two intake readings, which averaged 78 p.p.m. Based on these tests, the intake readings were found to be higher than average and a coefficient of 0.88 was derived to reduce the

intake recording to a mean suspended sediment concentration for the entire river cross section. A further adjustment was necessary, so that the sediment concentration taken at strength of ebb current could be converted to a mean for the ebb current duration equal to the ratio of maximum current 1.2 knots to mean current 0.4 knot for a coefficient of 0.33. Combining the two coefficients provided a constant of 0.3, which applied to the observed intake concentration in parts per million would provide a mean concentration representing the total suspended sediment passing Poughkeepsie each second. This study confirmed the fact that sediment concentration increases from the water surface to the bed of the river with average concentrations recorded at 0.6 depth and that distribution of suspended sediment varies in the cross section, being heaviest at the center and deeper parts of the river.

The daily sediment load, in tons passing Poughkeepsie and flowing to the lower Hudson River, was computed as follows:

Daily sediment load at Poughkeepsie (tons/day) $= Q$ (c.f.s.) $\times$ 86,400 (sec./day) $\times$ 62.4 (lb./cu. ft.) $\times \frac{1}{2{,}000}$ (ton/lb.) $\times \frac{\text{p.p.m.}}{10^6} \times 0.3$

$= 0.00808704 \times$ p.p.m. $\times Q$ (c.f.s.)

On the basis that the total suspended sediment transported past Poughkeepsie reaches the Lower Hudson River, the daily sediment load in tons was obtained by correcting sediment flow at Poughkeepsie by a factor equal to ratio of effective drainage areas of 1.1 (12,730 sq. mi./11,407 sq. mi.) or $0.008976614 \times$ p.p.m. $\times Q$ (c.f.s.).

The results of these measurements, more specifically the fresh water discharge at the problem area, average daily concentration of suspended sediment in parts per million through the ebb current at Poughkeepsie, daily suspended sediment load transported to the problem area for the period 4 July 1959 through 15 September 1960, and daily precipitation measured at Albany, N. Y., are shown on figure 9.

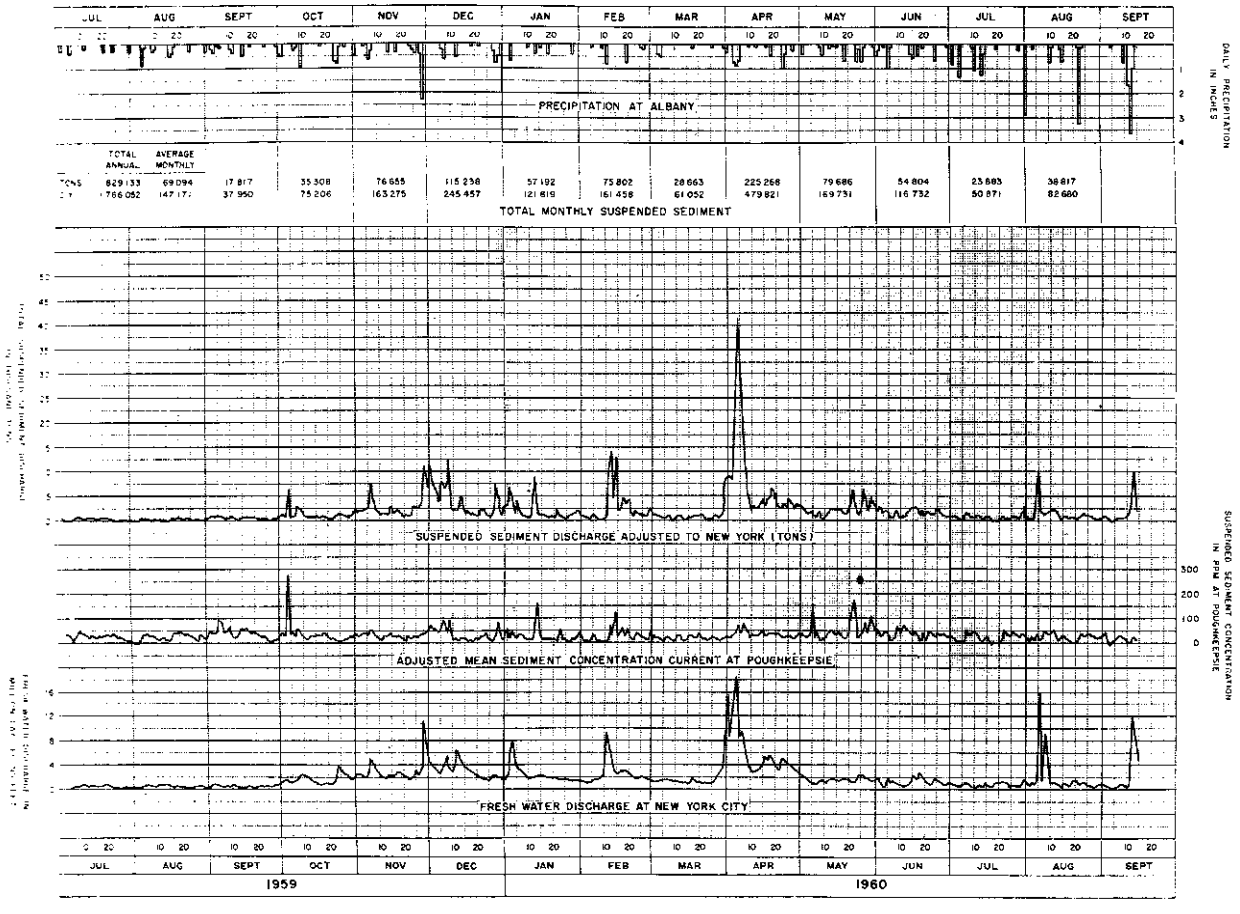


FIGURE 9. — Lower Hudson River sediment measurements, Poughkeepsie, N. Y.

Based on the foregoing, the total annual suspended sediment load transported to the Lower Hudson River for the 1-year period, September 1959 through August 1960, amounts to about 829,000 tons, or 1,766,000 cubic yards, based on an average dry density of 939 pounds per cubic yard. The total monthly loads are shown on figure 9. The total measured suspended sediment load is representative only of the period studied and cannot be considered as the average yearly load for the Hudson River. Even though the yearly fresh water discharge compares closely with the long term discharge, the sediment concentration varies considerably.

An analysis of the sedimentation measurements and computations result in the following observations:

1. The fresh water discharge at Green Island ranged from a low of 1,340 c.f.s. (5 July 1959) to a high of 216,896 c.f.s. (15 April 1960) with a period average of 16,505, which compares favorably with the 12-year mean of 14,400 c.f.s.

2. The mean daily suspended sediment concentration ranged from a low of 3 p.p.m. on 9 February 1960 to a high of 267 p.p.m. on 4 October 1959, for a daily average of 33 p.p.m.

3. The daily suspended sediment varied from a low of 59 tons (126 cu. yd.) on 28 September 1959 to a high of 41,300 tons (88,000 cu. yd.) on 5 April 1960 for an average of 2,272 tons (4,840 cu. yd.). Lower suspended sediment flows occur during the summer and fall months and higher sediment flows during the winter and spring.

4. High river discharges generally contain high sediment concentration.

5. The highest recorded sediment concentration was observed during a low flow, most probably due to an intense rainfall pattern of short duration over a particularly erodible area of the watershed.

Undisturbed samples of shoal material were obtained from the six major shoal areas and six pier slips where the silting is extremely heavy. These samples were analyzed to determine the in-place wet and dry densities and to identify the major chemical components and grain size of the shoal material for comparison with the upstream sediment samples obtained at Poughkeepsie. Fifty samples were obtained in the locations indicated on figure 10. At least two samples were taken in each shoal area. The samples were taken in the vertical with a 3-inch Charleston-type window sampler at depths ranging from 2 to 12 feet below the bottom, the depth to which the sampler penetrated due to its own weight. The Savannah fish-type sampler was used to obtain the "soupy" material at the top of the shoal, which was too fine or fluid to be collected by the window-type sampler.

The shoal samples were analyzed to determine the wet and dry densities, water content, specific gravity, and grain-size distribution (table 1). Twelve suspended sediment samples, taken at Poughkeepsie, N. Y. were similarly analyzed (table 1). The dry and wet densities were found to be generally constant in the vertical and not to increase with the depth. The average of the in-place dry densities of the upper layers of bottom material varied from 38.46 pounds per cubic foot in the channel to 31.12 pounds per cubic foot

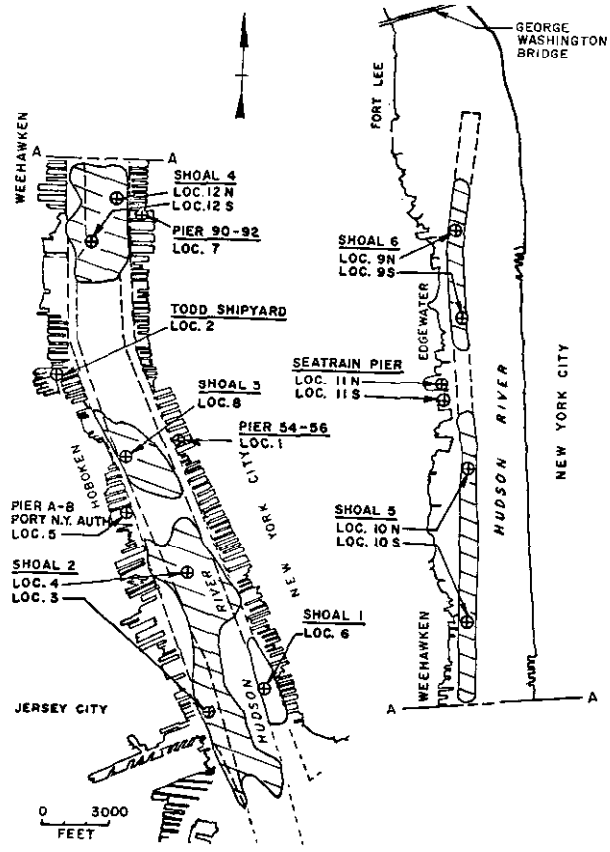


FIGURE 10. — Location of shoaling sampling in lower Hudson River.

in the slips. The average of these densities, 34.79 pounds per cubic foot, was used to convert total suspended sediment weight to volumetric units.

In order to identify further the similarity of shoal material and the upstream sediment source, grain size distribution was determined by mechanical analysis of the following: bottom samples taken from the channel shoal areas in 1960, a 180-gallon composite water-sediment sample taken at Poughkeepsie during 1959-60, and hopper dredge bin samples of material dredged from the Hudson River 30-foot project

TABLE 1.—*Physical and mechanical properties of Hudson River channel and slip samples and Poughkeepsie samples*

Location of sample	Depth of shoal surface	Depth of sample below shoal surface	Density			Wt. of dry solids	Analysis					
			Wet		Dry (salt-free)		Water content	Specific gravity	Organics	Gravel	Sand	Fines (clay or silts)
	Ft. m.l.w.	*Ft.*	*G/cc.*	*(Lb./ cu. ft.)*	*(Lb./ cu. ft.)*	*Pct.*	*Pct.*		*Pct.*	*Pct.*	*Pct.*	*Pct.*
Shoal 1, location 6	45.0	*2	1.430	89.23	44.74	50.14	85.1	2.62	9.2	15	34	51
Shoal 2, location 3	37.1	4	.....	.....	.....	.....	174.3	2.53	12.0	0	1	99
		6	1.302	81.24	45.18	55.61						
		8	1.494	93.23	53.13	56.99						
		10	1.507	94.04	24.94	26.52						
Shoal 2, location 4	48.0	*2	1.515	94.54	48.19	50.97	75.9	2.70	6.3	0	25	75
Pier A-B, N.J., location 5, Port of N.Y. Authority	26.1	**+0.1	1.244	77.62	26.08	33.60						
		1.9	1.318	82.24	30.80	37.45						
		3.9	1.245	77.69	29.60	38.10						
		5.9	1.186	74.01	25.52	34.48						
		7.9	1.225	76.44	27.83	36.41						
Piers 54-56, N.Y., location 1	24.7	**+0.4	1.201	74.94	22.11	29.50						
		1.6	1.326	82.74	32.86	39.71						
		3.6	1.228	76.62	28.00	36.54						
		5.6	1.231	76.81	28.30	36.84						
		7.6	1.275	79.56	35.50	44.62						
		9.6	1.265	78.90	19.67	24.93						
Shoal 3, location 8	50.2	3	.....	.....	.....	.....	108.8	2.70	7.3	18	13	69
		5	1.394	86.98	39.51	45.42						
		**2.5	1.264	78.87	29.90	37.91						
Todd Shipyards, location 2	25.8	**4.5	1.241	77.44	28.60	36.93						
		6.5	1.274	79.50	33.00	41.51						
		8.5	1.303	81.31	30.30	37.26						
		10.5	1.365	85.18	37.78	44.35						
Shoal 4, location 12N	51.1	*2.2	1.458	90.98	41.95	46.11	75.1	2.72	5.5	0	2	98
Shoal 4, location 12S	52.4	*0	1.381	86.17	37.30	43.29						
		2	1.489	92.91	48.71	52.43						
Piers 90-92, N.Y., location 7	43.8	**+0.8	1.178	73.50	50.70	68.98						
		3.2	1.251	78.06	26.72	34.23						
		5.2	1.309	81.68	32.86	40.23						
		7.2	1.270	79.25	34.80	43.91						
		9.2	1.298	80.99	33.47	41.33						
Shoal 5, location 10S	40.0	*3	1.336	83.37	25.84	30.99	77.4	2.71	7.3	0	5	95
Shoal 5, location 10N	33.5	*0	1.441	89.92	58.51	65.07						
		*2	1.459	91.04	34.37	37.75						
Seatrain Pier, location 11S	20.7	**1.5	1.263	78.81	25.46	32.30						
		3.6	1.300	81.12	31.01	38.23						
		5.6	1.289	80.43	28.38	35.28						
		**2	1.338	83.49	34.66	41.54						
		**4	1.317	82.18	31.54	38.37						
Seatrain Pier, location 11N	27.4	6	1.294	80.74	30.36	37.60						
		8	1.335	83.30	36.40	43.70						
		10	1.323	82.55	33.36	40.41						
		12	1.319	82.30	31.73	38.55						
		*2	1.383	86.30	35.50	41.13	132.2	2.70	10.9	0	2	98
Shoal 6, location 9S	32.5	*4.5	1.291	80.56	30.17	37.45						
		6.5	1.360	84.86	37.07	43.68						
		8	1,392	86.86	39.64	45.64						
		10	1.372	85.61	35.38	41.33						
Shoal 6, location 9N	.....	*+0.2	1.274	79.50	30.39	38.23						
		1.8	1.414	88.23	41.73	47.30						
		2.8	1.368	85.36	39.35	46.10						
Average	.....	.....	.....	.....	.....	.....	104.1	2.67	8.4	5	12	83
Average, channel*	.....	.....	1.404	87.58	38.46	43.91	.....	.....	.....	.....	.....	.....
Average, slips**	.....	.....	1.244	77.64	31.12	40.08	.....	.....	.....	.....	.....	.....
Average, slips and channels	.....	.....	1.324	82.61	34.79	42.11	.....	.....	.....	.....	.....	.....
Average, Poughkeepsie waterworks (12 samples)	.....	.....	1.004	62.63	.111	.18	142.0	2.53	2.28	0	8	92
Average, 30-ft. project (1952-57)	.....	.....	1.427	89.04	.....	.....	111.1	2.63	9.8	0	6	94
Average, 40-ft. project (1954-55)	.....	.....	1.463	91.29	.....	.....	108.0	2.64	.....	0	15	85

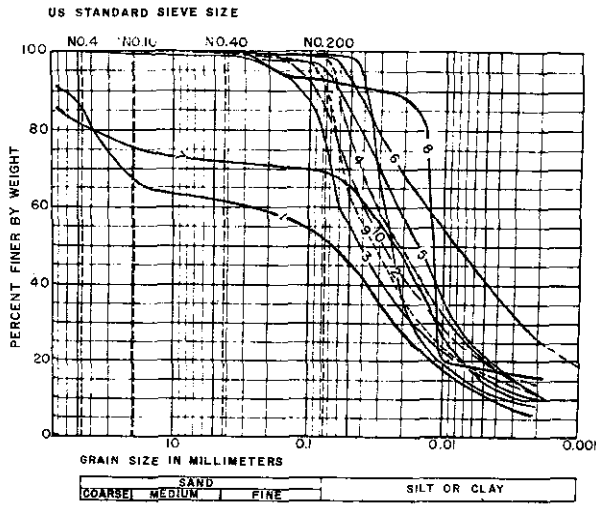
channel in Weehawken-Edgewater between 1952 and 1957 and the Hudson River 40-foot project channel in 1954-55. The results, summarized in table 2 and shown in figure 11, indicate that the Poughkeepsie sediments are generally finer than the channel shoal materials. In the upper particle-size range the shoal samples show a large amount of coarser materials. However, in the lower particle-size range, the shoal material is similar to the Poughkeepsie sediments. Overall, the lower Hudson River shoal materials have physical characteristics similar to those of the Poughkeepsie sediments, indicating a definite upstream source of sediment.

Standard chemical tests were made on bottom samples taken from the slip and channel shoals and the composite 180-gallon water-sediment sample at Poughkeepsie. The results are shown in table 3. In addition, a spectographic analysis was made of the Poughkeepsie composite sample and a sample taken from the south side of the Seatrain Pier at Weehawken with the following results.

Seatrain slip, Weehawken, southside: over 10 percent silicon; 1 to 10 percent calcium, aluminum, magnesium, and iron; 0.1 to 1 percent sodium. Not detected, potassium.

Poughkeepsie composite: over 10 percent iron, silicon; 1 to 10 percent calcium, aluminum, and magnesium; 0.01 to 1 percent sodium. Not detected, potassium.

Comparison of these test results, while not



LOCATION: HUDSON RIVER SHOALS & POUGKEEPSIE

SAMPLE NO.	LOCATION	DATE
1	SHOAL NO. 1, LOC. 6	JULY 13, 1960
2	SHOAL NO. 2 LOC. 3	JULY 7 1960
3	SHOAL NO. 2 LOC. 4	JULY 7, 1960
4	SHOAL NO. 4 LOC. 12N	JULY 13, 1960
5	SHOAL NO. 5 LOC. 10S	JULY 12, 1960
6	SHOAL NO 6 LOC. 9S	JULY 11, 1960
7	SHOAL NO. 3 LOC. 8	JULY 7, 1960
8	POUGHKEEPSIE WATER WORKS	OCT. 59-JULY 60
9	WEEHAWKEN-EDGEWATER 30 FT. CHANNEL	1952-1957
10	HUDSON RIVER 40 FT. CHANNEL	1954-1955

FIGURE 11. — Grain-size distribution curve.

TABLE 2.—*Comparative grain size of samples from Poughkeepsie, Hudson River shoals 1 to 6, and 30-foot and 40-foot project channels*

Place and location of sample	Date	Grain-size (millimeters)				Diameter size	
		0.001 to 0.01	0.01 to 0.1	0.1 to 1.0	1.0 and Over	D_{50}	D_{90}/D_{20}
		Pct.	*Pct.*	*Pct.*	*Pct.*	*Mm.*	*Mm.*
Poughkeepsie waterworks intake: Composite samples (fines 92 percent, sand 8 percent)	Oct. 59–July 60	20	73	7	0	0.013	1.5
Project channels, shoal areas:							
Shoal No. 1 Location 6	July 60	18	37	8	37	.066	308.0
2 3	July 60	19	80	1	0	.025	3.1
2 4	July 60	23	64	12	1	.041	10.0
3 8	July 60	27	43	3	27	.023	571.0
4 12	July 60	32	66	2	0	.021	10.2
5 10	July 60	35	62	3	0	.016	9.1
6 9	July 60	55	43	2	0	.008	3.0
Average (fines 83 percent, sand 12 percent, gravel 5 percent)	.....	30	57	4	9	.027	7.0
Edgewater-Weehawken-Hudson River, 30-foot project channel:							
Hopper dredge bin.....	1952	30	64	6	0	.019	8.0
Hopper dredge bin.....	1953	29	69	2	0	.026	11.2
Hopper dredge bin.....	1956	19	77	4	0	.040	5.0
Hopper dredge bin.....	1957	36	62	2	0	.030	20.0
Average, 12 samples (fines 94 percent, sand 6 percent)	.....	28	68	4	0	.033	8.7
Hudson River, 40-foot project channel: Average, 14 samples (fines 85 percent, sand 15 percent)	1954–55	29	65	6	0	.029	9.4

TABLE 3.—*Chemical analysis of upstream and downstream shoaling material, 1960, and composite at Poughkeepsie, 1959–60*

Sample Location	Sample Date	Chlorides	Sulfates	Alkalinity methyl orange	Carbonate hardness	Non-carbonate hardness	Iron soluble	Silicon dioxide	Aluminum
		Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
Shoal 1	July 13	0.635	0.337	0.152	0.152	15.078	0.00300	52.000	1.055
Shoal 2, south	July 7	.834	.257	.097	.097	16.454	.00450	23.600	1.682
Shoal 2, north	July 7	.612	.308	.180	.180	16.182	.01200	46.000	1.468
Between piers A-B, N.J.	July 7	.952	.007	.107	.107	4.126	.00019	8.260	.230
Between piers A-B, N.J.	July 7	.750	.148	.097	.097	15.297	.01250	23.200	1.582
Shoal 3	July 7	.690	.216	.097	.097	15.966	.00300	43.400	1.638
Between piers 54-56	June 7	1.166	.075	.074	.074	2.639	.00007	4.700	.185
Between piers 54-56	June 7	.875	.453	.111	.111	14.828	.00400	30.000	1.557
Between piers 16-H, N.J.	June 7	.815	.108	.025	.025	2.298	.00015	.812	.080
Between piers 90-92, N.Y.	July 7	1.202	.043	.118	.118	2.596	.00008	4.890	.160
Between piers 90-92, N.Y.	July 7	.850	.123	.180	.180	11.996	.00750	20.900	1.211
Shoal 4, north	July 13	.472	.230	.097	.097	18.038	.01150	48.300	1.772
Shoal 5, south	July 12	.547	.288	.070	.070	17.694	.01350	44.200	1.720
Seatrain pier, south	July 8	.544	.158	.097	.097	16.741	.00350	23.300	1.700
Seatrain pier, north	July 8	.529	.009	.066	.066	11.801	.00044	20.250	1.177
Seatrain pier, north	July 8	.547	.070	.111	.111	15.104	.00750	30.700	1.542
Shoal 6, south	July 11	.373	.123	.111	.111	15.300	.00500	44.200	1.490
Shoal 6, south	July 11	.662	.015	.077	.077	11.558	.00021	15.700	1.254
Average	.....	.725	.165	.104	.104	12.428	.005	26.912	1.195
Composite, Poughkeepsie	Oct. 59 to July 60	.025	.029	.097	.097	32.629	.020	16.600	.952

Sample Location	Sample Date	Iron total	Calcium	Magnesium	Sodium	pH[1]	Depth of sample below top of shoal
		Pct.	*Pct.*	*Pct.*	*Pct.*		
Shoal 1	July 13	1.421	1.740	0.290	0.161	6.7	2 ft. below bottom.
Shoal 2, south	July 7	1.747	.303	.422	.177	6.5	4 ft. below bottom.
Shoal 2, north	July 7	1.557	.944	.403	.155	6.7	2 ft. below bottom.
Between piers A-B, N.J.	July 7	.695	.109	.222	.172	7.4	.5 ft. above bottom.[2]
Between piers A-B, N.J.	July 7	1.601	.322	.385	.150	6.9	.1 ft. above bottom.
Shoal 3	July 7	1.748	.172	.447	.149	6.8	3 ft. below bottom.
Between piers 54-56	June 7	.327	.064	.156	.459	7.5	.5 ft. below bottom.[2]
Between piers 54-56	June 7	1.523	.332	.329	.209	6.7	1.6 ft. below bottom.
Between piers 16-H, N.J.	June 7	.056	.024	.074	.427	7.0	.5 ft. below bottom.[2]
Between piers 90-92, N.Y.	July 7	.370	.062	.164	.430	7.6	.5 ft. above bottom.[2]
Between piers 90-92, N.Y.	July 7	1.288	.237	.319	.160	6.8	.8 ft. above bottom.
Shoal 4, north	July 13	1.995	.286	.536	.104	6.5	2.2 ft. below bottom.
Shoal 5, south	July 12	1.937	.349	.517	.100	6.3	3.0 ft. below bottom.
Seatrain pier, south	July 8	1.893	.209	.455	.120	6.5	3.6 ft. below bottom.
Seatrain pier, north	July 8	1.362	.151	.311	.378	7.3	.5 ft. below bottom.[2]
Seatrain pier, north	July 8	1.768	.172	.355	.110	6.7	2 ft. below bottom.
Shoal 6, south	July 11	1.870	.226	.382	.088	6.8	2 ft. below bottom.
Shoal 6, south	July 11	1.160	.111	.309	.461	7.4	10 ft. below bottom.
Average	.....	1.351	.323	.338	.501	6.9	
Composite, Poughkeepsie[3]	Oct. 59 to July 60	9.860	.097	.169	.020	6.6	

[1] 20 percent solution.
[2] Sample obtained with fish type sampler.
[3] Potassium not detected.

entirely conclusive, indicated a reasonable similarity between the sediments from the upstream Hudson River drainage area and the shoaling material found in the lower Hudson River.

Sanitary and storm sewer and industrial and other waste material are discharged or dumped into the Hudson River and other waterways tributary to the Harbor.

An inventory of data from local agencies revealed that 159,000 cubic yards of dry solid industrial waste and sewage have been discharged annually into the Hudson River between the Battery, Manhattan, and Bear Mountain, and 473,000 cubic yards of dry solids into other portions of New York Harbor for a total source of 632,000 cubic yards annually. Although a part of these solids would be reduced by natural processes and another part would never settle to the bottom due to its specific gravity, the rest constitutes a consideral potential source of shoal material in the harbor. This source of shoal material is estimated at 412,000

cubic yards, about 65 percent of the annual discharge or about 10 percent of the annual shoaling in the Lower Hudson River, including the slips. Although these studies do not represent a complete inventory and accounting of material source into the Lower Hudson River, certain general findings are evident.

The Hudson River watershed is a definite source of sediment moving into the Lower Hudson River. This source represents an estimated potential shoaling material equal to about 1¾ million cubic yards annually. This represents a suspended sediment load of 43 percent of the annual shoaling in the Lower Hudson River. Some of these sediments would probably settle in the Haverstraw Bay and Tappan Zee area because of predominant upstream bottom currents north of the George Washington Bridge during medium and low fresh waterflows in the Hudson River. However, it is reasonable to consider that losses in this area would be offset by sediment discharge entering the Lower Hudson River.

These two sources do not account for all the sediment deposited in shoals. Measurements were not made of other possible sources, such as the 3,000-square mile area tributary to the New York Harbor downstream of the Hudson River and the Atlantic Ocean on the floodtides.

Cross-Sectional Distribution of Velocity, Salinity, and Sediment Discharge

For a better evaluation of the factors that influence the shoaling and to obtain prototype data for construction and verification of the models, tidal observations, velocity measurements and water samples for salinity, and sediment concentration, distribution, and discharge were taken at stations established on each of five ranges (fig. 12). These ranges were located across the river, spaced about 2 miles apart along the 11-mile reach of the Lower Hudson River between the Battery and George Washington Bridge. A single master control station was established on range A, the farthest downstream, and three stations were established at each of the other ranges, B through E.

Ideally, the measurements were designed for a single complete tidal cycle during average conditions for all stations, simultaneously. However, since this was not possible because of field conditions, personnel, and equipment, the measurements were made on successive days beginning with range B on 23 April 1957 and continuing for one tidal cycle on each of the upstream ranges in succession until range E was completed on 26 April. The master control station was operated throughout the time that the measurements were performed on any of the other stations, to measure the similarity of

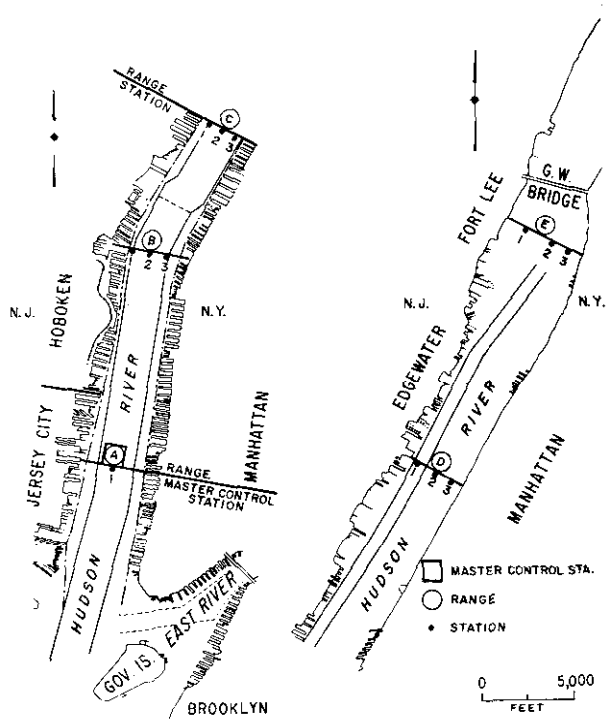


FIGURE 12. — Location of ranges in lower Hudson River.

conditions under which the measurements for each station were taken.

The measurements were taken from small pontoons anchored at each station. The field party on each of the four pontoons, three on the upstream range and one on the control station, consisted of two men. The work was performed only on alternate tidal cycles, and the men worked in two shifts of ten men each to take advantage as much as possible of daylight hours. Velocity measurements were obtained by Price current meter. Steel vanes suspended from a cable served as current direction indicators. Pint-size milk bottles and a mechanism that permitted opening of the bottles at the desired depth were used to obtain water samples.

At each range, the velocity and direction of current were measured simultaneously at the three stations and the master control station at half-hour intervals for 5 depths: 1.5 feet below the surface, at ¼, ½, and ¾ depths, and 2 feet above the bottom. Water samples were collected hourly at each station for every depth simultaneously with the velocity measurements. The results of these measurements in terms of cross-sectional distribution of velocity, salinity, tide level, sediment discharge at minimum, mid, and maximum velocity are shown for each range (B through E) on figures 13, 14, and 15. The vertical variation in velocity, salinity, and sediment in the midstation of range A over a

tidal cycle is shown on figure 16. The vertical and horizontal variation of sediment discharge across range B during one tidal cycle is shown on figure 17. The variation in the average velocity, salinity, and sediment at midstation of range A and at range E over a tidal cycle is shown on figures 18 and 19. The net sediment discharge in cubic feet per second for all ranges (A to E) is shown in table 4.

The cross-sectional channel geometry varies

TABLE 4.—*Lower Hudson River: net sediment discharge, April 23 to 26, 1957* [1]

Range	Station	Date	Sediment discharge at indicated depth					Net sediment total
			Top −1.5 ft.	¼ depth	½ depth	¾ depth	Bottom +2 ft.	
			C.f.s.	*C.f.s.*	*C.f.s.*	*C.f.s.*	*C.f.s.*	*C.f.s.*
A	2	23	−16.99	− 4.60	+ 5.24	+17.50	+ 3.30	+ 4.45
	2	24	−30.38	−30.94	− 3.48	+ 1.10	+ 6.99	56.71
	2	25	−39.07	−65.20	+17.33	+20.76	+ 6.85	−59.33
	2	26	−14.94	−18.25	+16.33	+ 8.72	+ 2.03	− 6.11
B	1	23	− 3.94	− 3.91	+ 2.97	− 0.47	− 0.15	− 5.50
	2	23	− 8.29	− 7.46	− 2.05	+ 0.88	− 0.41	−17.33
	3	23	− 0.42	− 6.87	− 3.63	+ 0.71	+ 2.92	− 7.29
Total			−12.65	−18.24	− 2.71	+ 1.12	+ 2.36	−30.12
C	1	24	+ 2.60	+ 3.72	+ 0.54	+ 2.66	+ 2.38	+11.90
	2	24	− 1.75	− 1.93	− 2.91	+ 1.51	+ 0.05	− 5.03
	3	24	− 8.66	− 1.15	− 3.00	− 0.44	− 0.52	−13.77
Total			− 7.81	+ 0.64	− 5.47	+ 3.83	+ 1.91	− 6.90
D	1	25	− 9.24	− 2.96	− 0.97	+ 0.89	+ 1.46	−10.91
	2	25	− 1.66	+ 1.42	+ 4.46	+ 6.25	+ 0.67	+11.14
	3	25	− 1.12	− 3.10	+ 5.72	+ 2.55	+ 0.55	+ 4.60
Total			−12.02	− 4.64	+ 9.21	+ 9.60	+ 2.68	+ 4.83
E	1	26	− 1.64	− 5.57	− 7.46	− 0.86	− 1.82	−17.35
	2	26	+10.00	− 9.85	− 1.82	+ 1.61	+ 1.35	+ 1.29
	3	26	− 4.28	+ 2.94	+ 7.20	+ 4.50	+ 2.34	+12.70
Total			+ 4.08	−12.48	− 2.08	+ 5.25	+ 1.87	− 3.36
Total of stations B–E			−28.40	−34.72	− 1.05	+19.80	+ 8.82	−35.55
Average			− 7.10	− 8.68	− 0.26	+ 4.95	+ 2.21	− 8.88

[1] − = downstream flow; + = upstream flow.

Average discharge for 11 years, Green Island, N.Y. = 13,930 c.f.s.

Average discharge for April 1957 = 15,730 c.f.s.

Computation of annual sediment yield = $\frac{-8.88 \text{ c.f.s.} \times 43{,}300 \text{ sec. per cycle}}{27 \text{ (cu. ft. per cu. yd.)}}$ = 14,200 cu. yd. per cycle

14,200 × 700 cycles = 9,940,000 cu. yd./yr.

from a horizontal bottom in ranges B and C to the gorgelike bottom on the New York side in ranges D and E (fig. 13). The part of the river downstream of range C is maintained to depths of 40 to 48 feet by dredging practically throughout its width to the pierhead lines, whereas the river upstream has a natural deep section along the New York side and a shallow depth on the New Jersey side in which a 30-foot navigable channel is maintained.

The velocities generally decrease with the depth (fig. 16) and are higher on the New York side except for the upstream range E (figs. 13 to 15). However, the maximum velocities are about equal. The velocities are higher during the ebbtide than during the floodtide (figs. 18-19) with the maximum ebb velocity recorded at 5.7 feet per second and with the maximum flood velocity recorded at 4.0 feet per second. The slack current lags the slack water by about 3 hours at ebb and floodtides (figs. 18 and 19).

The salinity increases with depth and varies at some time as much as 22,000 p.p.m. from top to bottom (figs. 13 to 16). The saline density layers for all ranges of velocities generally increase toward the bottom and the New York side of the river, not only in the upstream ranges where the natural deep water is on the New York side but also in the downstream ranges where the depth is uniform across the river (figs. 13 to 15).

The greatest sediment discharge for all periods of flow was found in suspension in the upper half of the depth where not only is the sediment concentration fairly high but also the velocities are closest to maximum (figs. 13

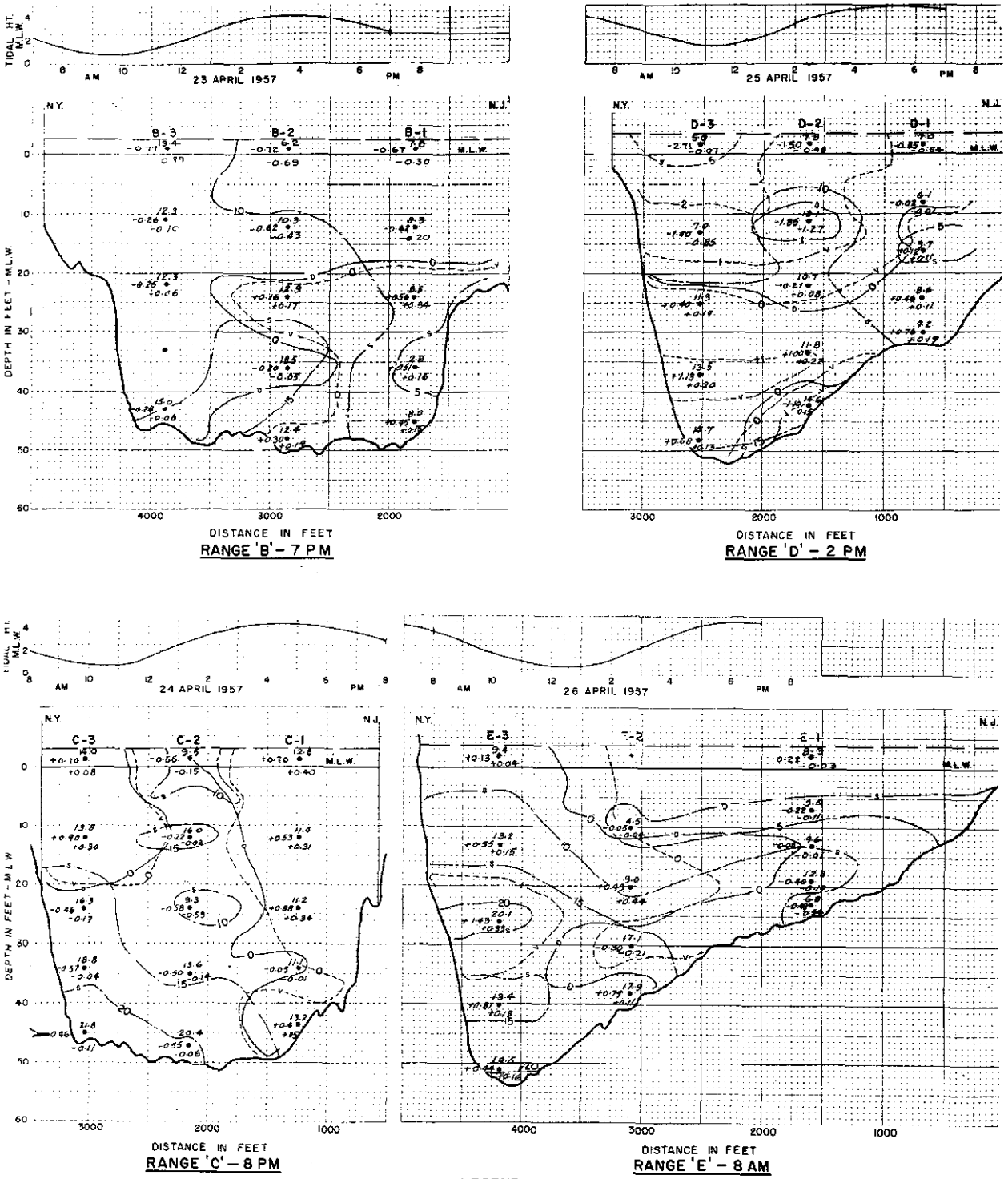


FIGURE 13. — Current velocity, sediment discharge, and salinity at *minimum velocity*.

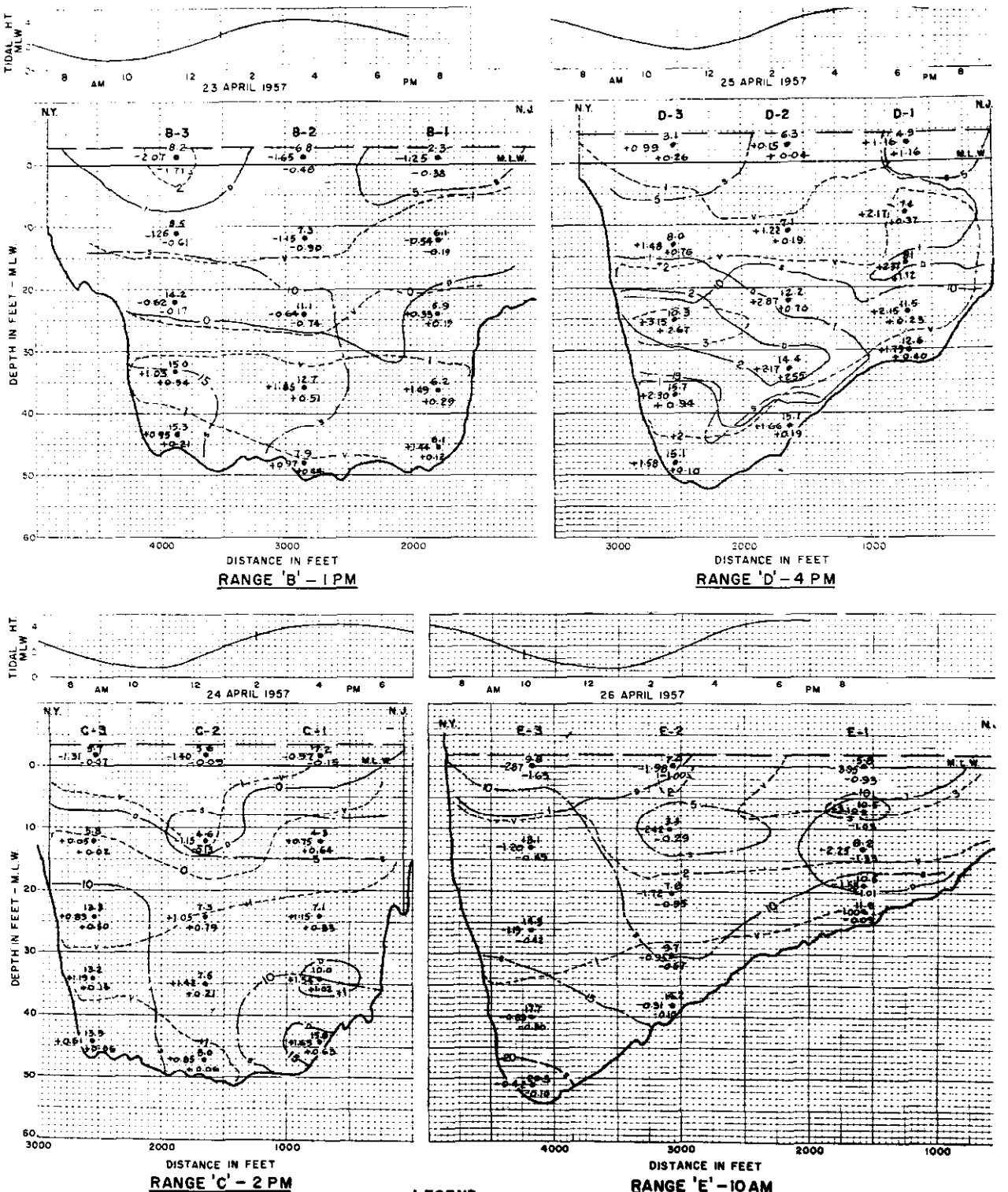


FIGURE 14. — Current velocity, sediment discharge, and salinity at mid-velocity.

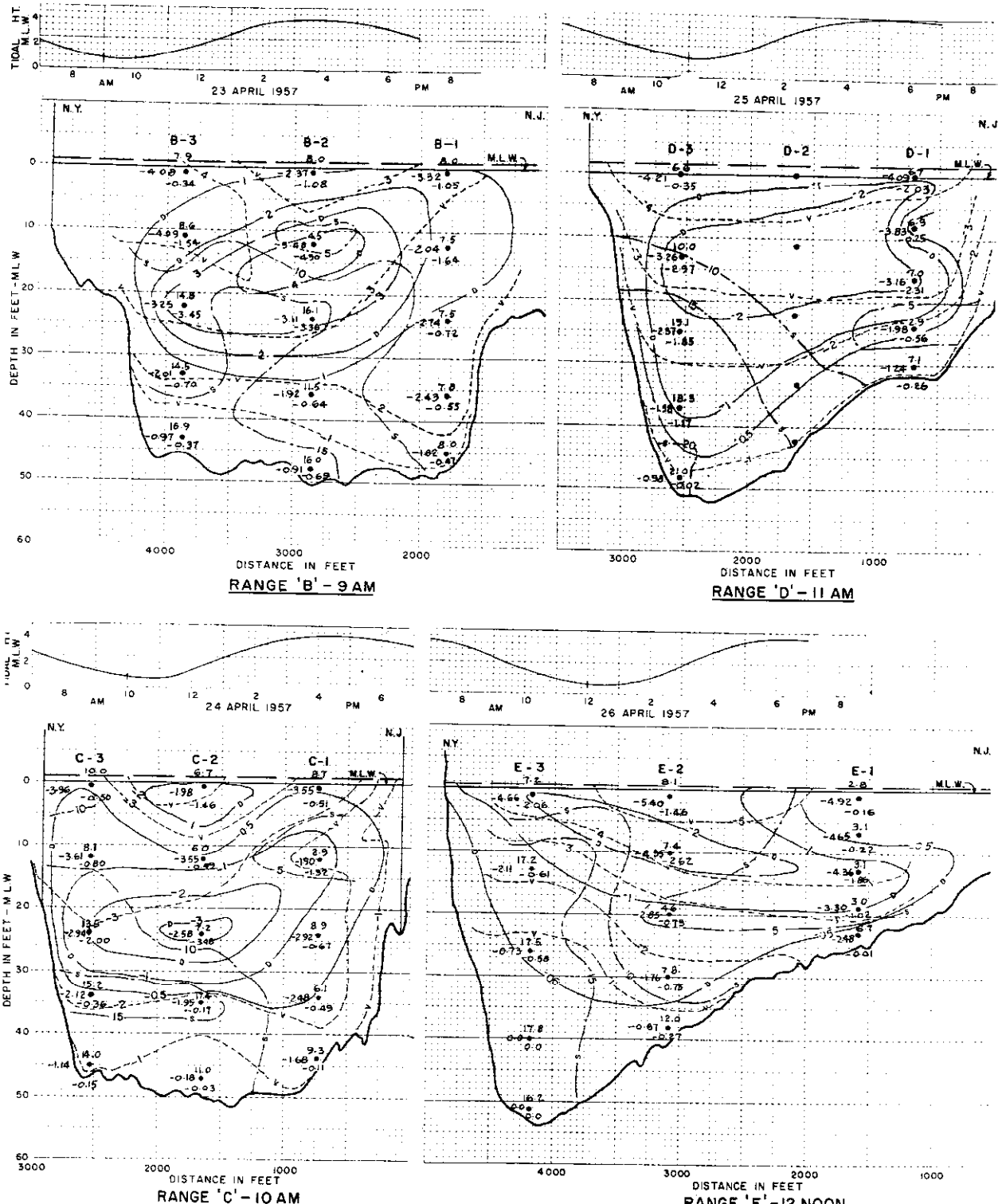


FIGURE 15. — Current velocity, sediment discharge, and salinity at maximum velocity.

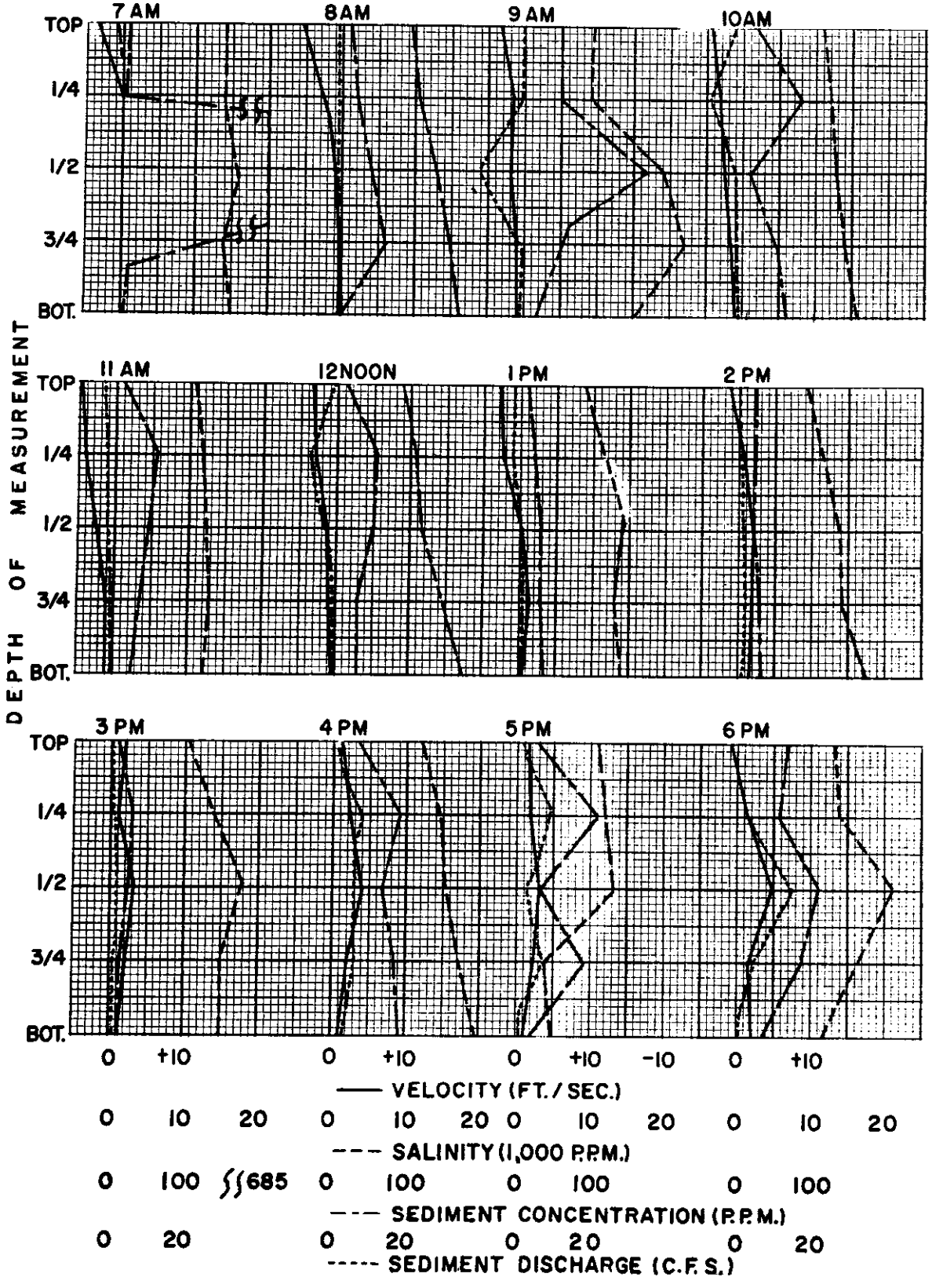


FIGURE 16. — Vertical variation in velocity, salinity, and sediment, lower Hudson River range A, 23 April 1957.

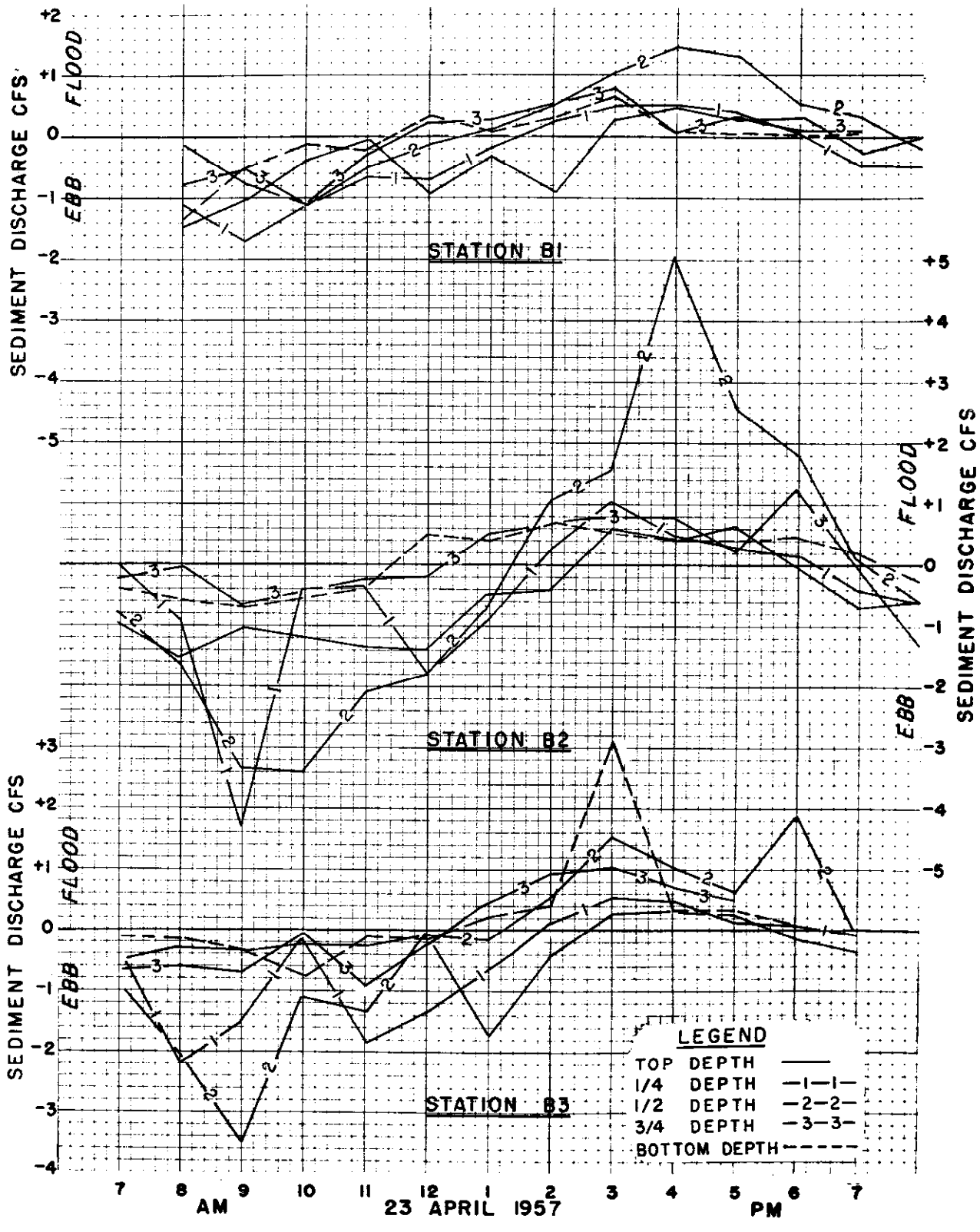


FIGURE 17.—Vertical and horizontal variation in sediment discharge over one tidal cycle.

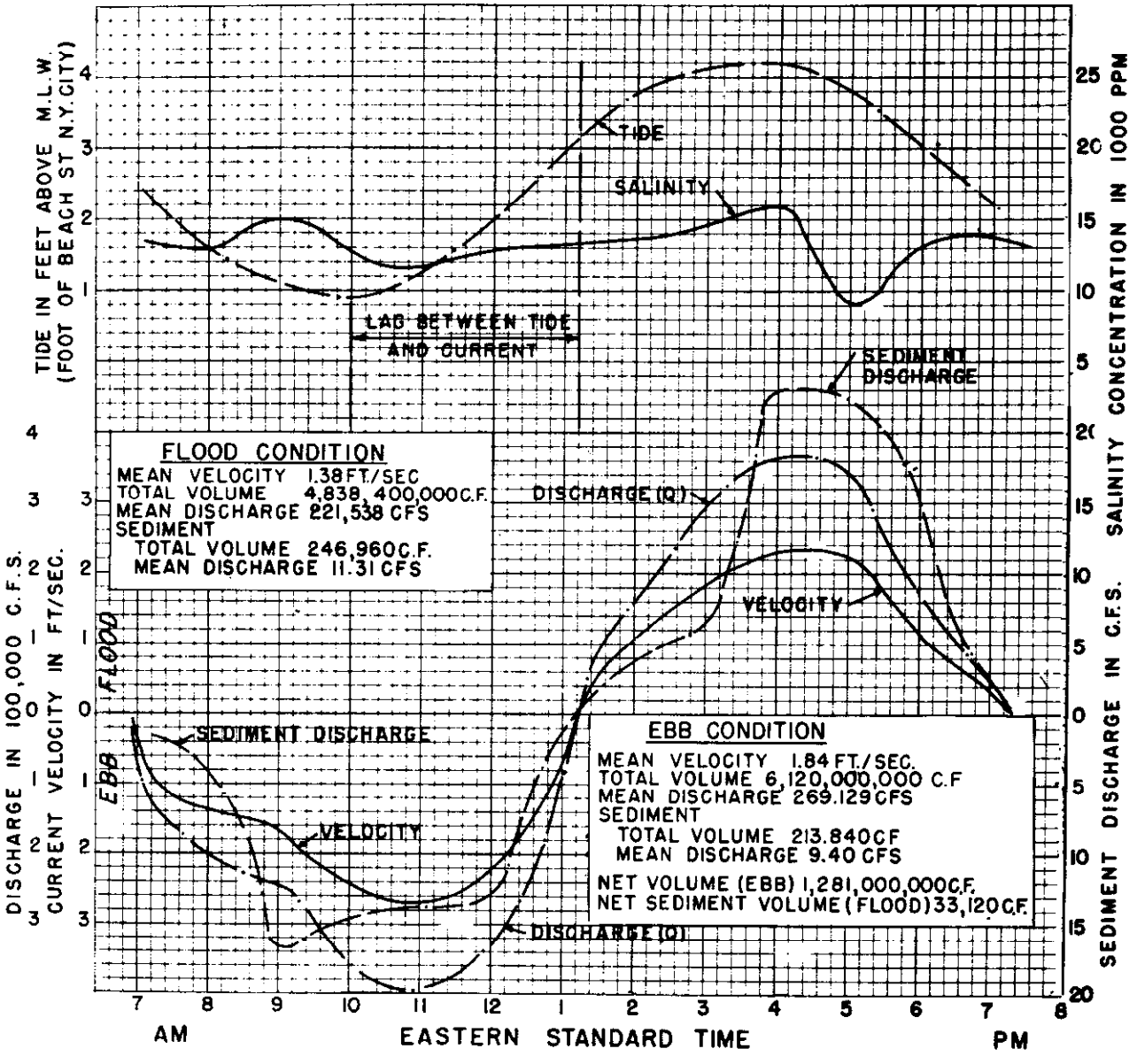


FIGURE 18.—Variation in average velocity, salinity, and sediment, range A, 23 April 1957.

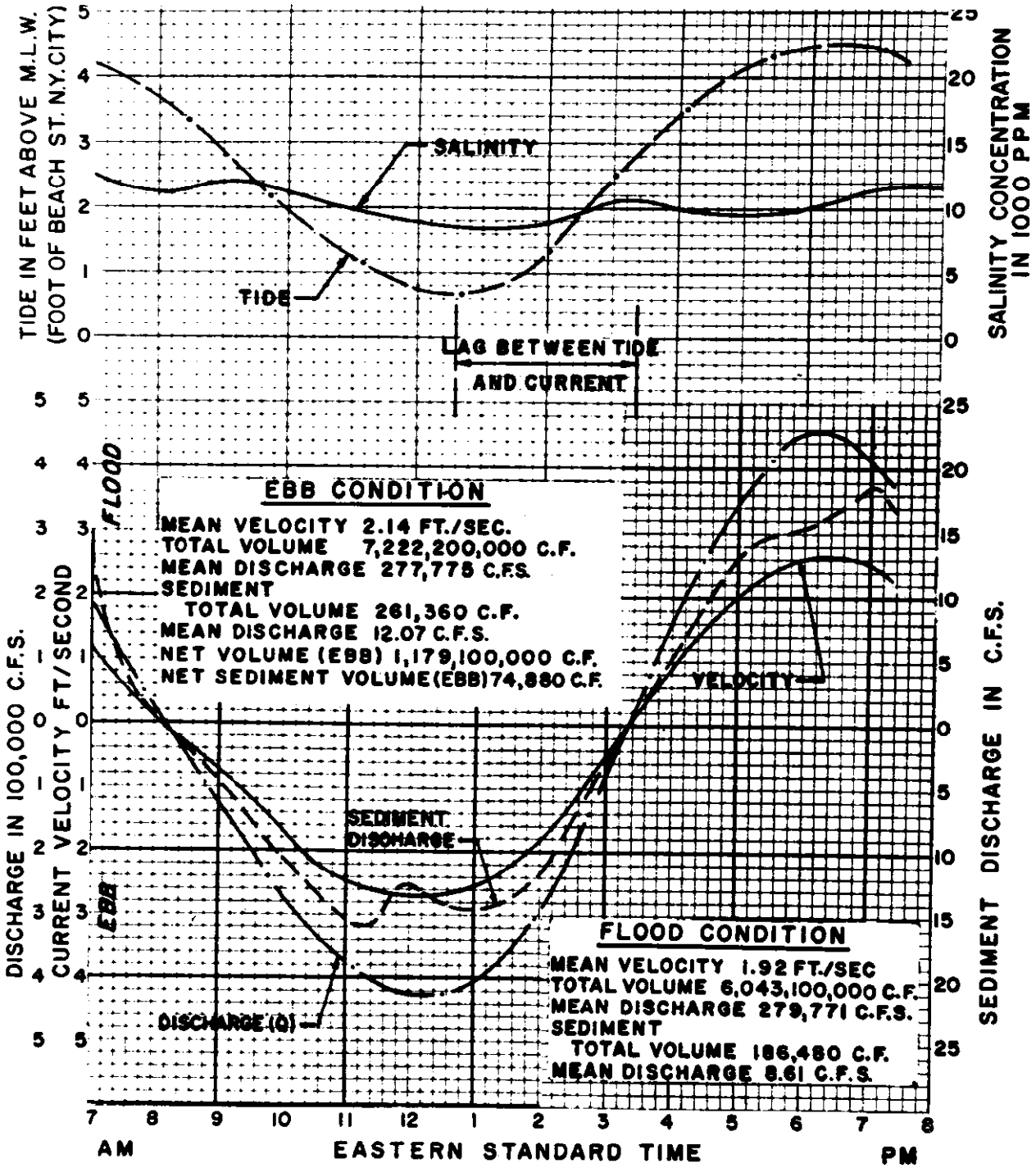


FIGURE 19. — Variation in average velocity, salinity, and sediment, range E, 26 April 1957.

through 17). During the time when the currents are changing from ebb to flood, with the ebb current still flowing on the top and the flood currents flowing in on the bottom, a definitely greater concentration of sediment is found in the bottom layers. It was also found that the greater sediment concentration existed in the boundary zone between the salt and brackish layer (table 5).

The sediment discharge was determined by

TABLE 5.—*Typical prototype observations and computations of salinity, velocity, and sediments, Lower Hudson River at range B, station 1, April 23, 1957*[1]

Current meter depth	Weighted area	Velocity	Q	Sediment concentration	Sediment discharge		Salinity
Depth 1.5 feet from top:	*Ft.*²	*Ft./sec.*	*C.f.s.*	*P.p.m.*	*Lb./sec.*	*C.f.s.*	*P.p.m.*
7 a.m.	×	—	—	—	—	—	—
8	7,320	−3.41	−24,961	54.9	− 85.5	−1.37	7,600
9	6,150	−3.32	−20,418	51.4	− 65.5	−1.05	8,000
10	6,150	−3.63	−22,324	17.2	− 24.0	− .38	6,700
11	7,080	−3.35	−23,718	2.6	− 3.8	− .06	4,000
12	8,720	−2.40	−20,928	45.1	− 58.9	− .94	1,200
1	10,830	−1.25	−13,538	27.9	− 23.6	− .38	2,300
2	12,700	− .84	−10,668	82.1	− 54.6	− .88	7,500
3	13,630	+ .96	+13,085	28.1	+ 22.9	+ .37	6,800
4	13,630	+ .68	+ 9,268	49.5	+ 28.6	+ .45	7,700
5	12,930	+ .89	+11,508	33.3	+ 23.9	+ .38	8,800
6	11,760	+ .65	+ 7,644	38.1	+ 18.2	+ .29	8,600
7	10,360	− .67	− 6,941	43.9	− 19.0	− .30	7,800
8 p.m.	8,720	− .82	− 7,150	9.6	− 4.3	− .07	9,200
¼ depth:							
7 a.m.	×	—	—	—	—	—	—
8	10,080	−1.79	−18,043	57.9	− 65.2	−1.04	7,200
9	10,080	−2.04	−20,563	79.7	−102.3	−1.64	7,500
10	10,080	−2.03	−20,462	55.0	− 70.2	−1.13	7,500
11	10,080	−1.75	−17,640	35.1	− 38.6	− .62	5,800
12	10,080	− .95	− 9,576	65.6	− 39.2	− .63	6,600
1	10,080	− .54	− 5,443	34.4	− 11.7	− .19	6,100
2	10,080	+ .82	+ 8,266	33.9	+ 17.5	+ .28	8,000
3	10,080	+1.13	+11,390	41.7	+ 29.6	+ .47	7,500
4	10,080	+1.75	+17,640	31.8	+ 35.0	+ .56	10,400
5	10,080	+1.55	+15,624	26.2	+ 25.5	+ .41	10,900
6	10,080	+ .70	+ 7,056	10.3	+ 4.5	+ .07	7,500
7	10,080	− .62	− 6,250	31.3	− 12.2	− .20	8,300
8 p.m.	10,080	− .98	− 9,878	25.6	− 15.8	− .25	9,200
½ depth:							
7 a.m.	×	—	—	—	—	—	—
8	10,000	−1.75	−17,500	4.6	− 5.0	− .08	7,800
9	10,000	−2.74	−27,400	26.2	− 44.8	− .74	7,500
10	10,000	−2.41	−24,100	47.0	− 70.7	−1.13	7,000
11	10,000	− .63	− 6,300	.....	.....	.....	.....
12	10,000	− .52	− 5,200	23.3	− 7.6	− .12	6,000
1	10,000	+ .33	+ 3,300	45.3	+ 9.3	+ .15	5,900
2	10,000	+1.98	+19,800	23.9	+ 29.5	+ .47	6,500
3	10,000	+2.72	+27,200	38.0	+ 64.5	+1.03	9,800
4	10,000	+2.91	+29,100	48.9	+ 88.8	+1.42	16,500
5	10,000	+2.15	+21,500	61.2	+ 82.1	+1.32	16,200
6	10,000	+1.18	+11,800	41.7	+ 30.7	+ .49	9,500
7	10,000	+ .56	+ 5,600	60.2	+ 21.0	+ .34	8,500
8 p.m.	10,000	− .54	− 5,400	37.1	− 12.5	− .20	9,700
¾ depth:							
8 a.m.	7,800	−2.32	−18,096	42.9	− 48.6	− .78	7,200
9	7,800	−2.43	−18,954	28.8	− 34.1	− .55	7,800
10	7,800	−1.97	−15,366	70.5	− 67.7	−1.08	7,400
11	7,800	− .85	− 6,630	53.8	− 22.3	− .36	12,500
12	7,800	+ .41	+ 3,198	67.3	+ 13.4	+ .22	6,500
1	7,800	+1.49	+11,622	25.2	+ 18.3	+ .29	6,200
2	7,800	+1.81	+14,118	39.1	+ 34.5	+ .55	14,300
3	7,800	+1.65	+12,870	62.2	+ 50.0	+ .80	14,200
4	7,800	+1.63	+12,714	8.3	+ 6.6	+ .11	17,100
5	7,800	+1.30	+10,140	34.1	+ 21.6	+ .35	18,000
6	7,800	+1.29	+10,062	.....	.....	.....	.....
7	7,800	+ .51	+ 3,978	39.7	+ 9.9	+ .16	2,800
8 p.m.	7,800	− .81	− 6,318	28.7	− 11.3	− .18	8,100
2 feet from bottom:							
8 a.m.	5,760	−1.73	− 9,964	133.3	− 83.0	−1.33	14,300
9	5,760	−1.82	−10,483	45.3	− 29.7	− .47	8,000
10	5,760	−1.36	− 7,834	15.8	− 7.7	− .12	6,800

TABLE 5.—*Typical prototype observations and computations of salinity, velocity, and sediments, Lower Hudson River at range B, station 1, April 23, 1957 —Continued*

Current meter depth (Ft.)	Weighted area	Velocity	Q	Sediment concentration	Sediment discharge		Salinity
	Ft.²	*Ft./sec.*	*C.f.s.*	*P.p.m.*	*Lb./sec.*	*C.f.s.*	*P.p.m.*
11	5,760	− .48	− 2,764	81.3	− 14.0	− .23	8,700
12	5,760	+ .51	+ 2,938	91.6	+ 16.8	+ .27	1,200
1	5,760	+1.44	+ 8,294	14.7	+ 7.6	+ .12	6,100
2	5,760	+1.33	+ 7,661	48.2	+ 23.1	+ .37	2,300
3	5,760	+1.20	+ 6,912	100.8	+ 43.5	+ .70	15,600
4	5,760	+ .75	+ 4,320	38.7	+ 10.4	+ .17	12,700
5	5,760	+ .81	+ 4,666	26.3	+ 7.7	+ .12	15,600
6	5,760	+ .59	+ 3,398	47.1	+ 10.0	+ .16	8,200
7	5,760	+ .45	+ 2,592	59.5	+ 9.6	+ .15	8,000
8 p.m.	5,760	+ .62	− 3,571	17.5	− 3.9	− .06	9,100

[1] − = downstream flow; + = upstream flow.

multiplying the sediment concentration in parts per million, as determined from the analysis of the water samples and by the velocity and area for applicable parts of the cross-sectional area (table 5 and figures 9, 13 through 19).

The net sediment discharge results are shown in table 4. The net total flows of sediment are shown on figure 20 for each station in each range in order to demonstrate the apparent circulation of the sediment in the Lower Hudson River. The plot indicates that at the upstream ranges D and E, the net flow on the New York side is upstream (stations 2 and 3, ranges D and E, fig. 20). Because of the constriction at the George Washington Bridge, the net flow and much of the sediment do not pass upstream but change direction in the region of range E and flow downstream on the New Jersey side (station 3, ranges D and E, fig. 20). On the New York side in the downstream ranges A, B, and C, the net flow is downstream. It therefore can be presumed that scour is occurring on the New York side in the upstream half of the river and that, in part, it accounts for the natural deep water found here.

On the New Jersey side, in the upstream half of the river along the 30-foot navigable channels, the flow is downstream except at the downstream point of the 30-foot channel, where the net flow is upstream (station 1, range C, fig. 20). Therefore, deposition should be expected and does occur here, as evidenced by the location of shoal 4, an area of heavy shoaling in the Lower Hudson River.

If the total discharge of sediment for the tidal cycle at each range is taken as the net amount in the indicated direction, the net volume of materials moving up or down the river would follow:

At range B, —30.12 c.f.s. would represent 48,300 cubic yards per tidal cycle, or about 34 million cubic yards annually moving downstream.

At range C, —6.90 c.f.s. would represent 11,070 cubic yards per tidal cycle, or about 7.7 million cubic yards annually moving downstream.

At range D, +4.83 c.f.s. would represent 7,800 cubic yards per tidal cycle, or about 5.4 million cubic yards annually going upstream.

At range E, —3.36 c.f.s. would represent 5,400 cubic yards per tidal cycle, or about 3.8 million cubic yards annually going downstream.

As to the differentiation of net flow between upper and lower layers of the river, the measured results indicate that there is a downstream (—) predominance at the top and an upstream (+) predominance at the bottom (figs. 13, 14, 16, and 17). Where the boundary between the saline density layers could be distinguished, there appeared to be greater sediment concentration in the saline layer just below the boundary (table 5).

Model Measurements

The movement of material in a tidal waterway leading to the formation of a shoal is not the simple process of a single deposit of material in a given location but the end result of a complicated series of pickups and deposits and movements to and fro. This action is complex as to the varied interaction of many forces in the limited understanding of the many aspects of tidal hydraulics. Under such circumstances, the model is a necessary and effective tool to supplement prototype measurements in the study of this problem, because it effectively integrates the many forces and influences.[2]

Models

Two models were constructed and tested at the Waterways Experiment Station; one a comprehensive model of the Lower Hudson River with all the adjacent channels that affect the tidal regimen, and the other a larger scale model of the Lower Hudson River from the Battery to the George Washington Bridge (figs. 21, 22, and 23). The comprehensive model was

[2] DOUMA, J. H. ENGINEERING PROBLEMS IN U.S. TIDAL WATERWAYS. Amer. Soc. Civil Engin. Paper 789. 1955.

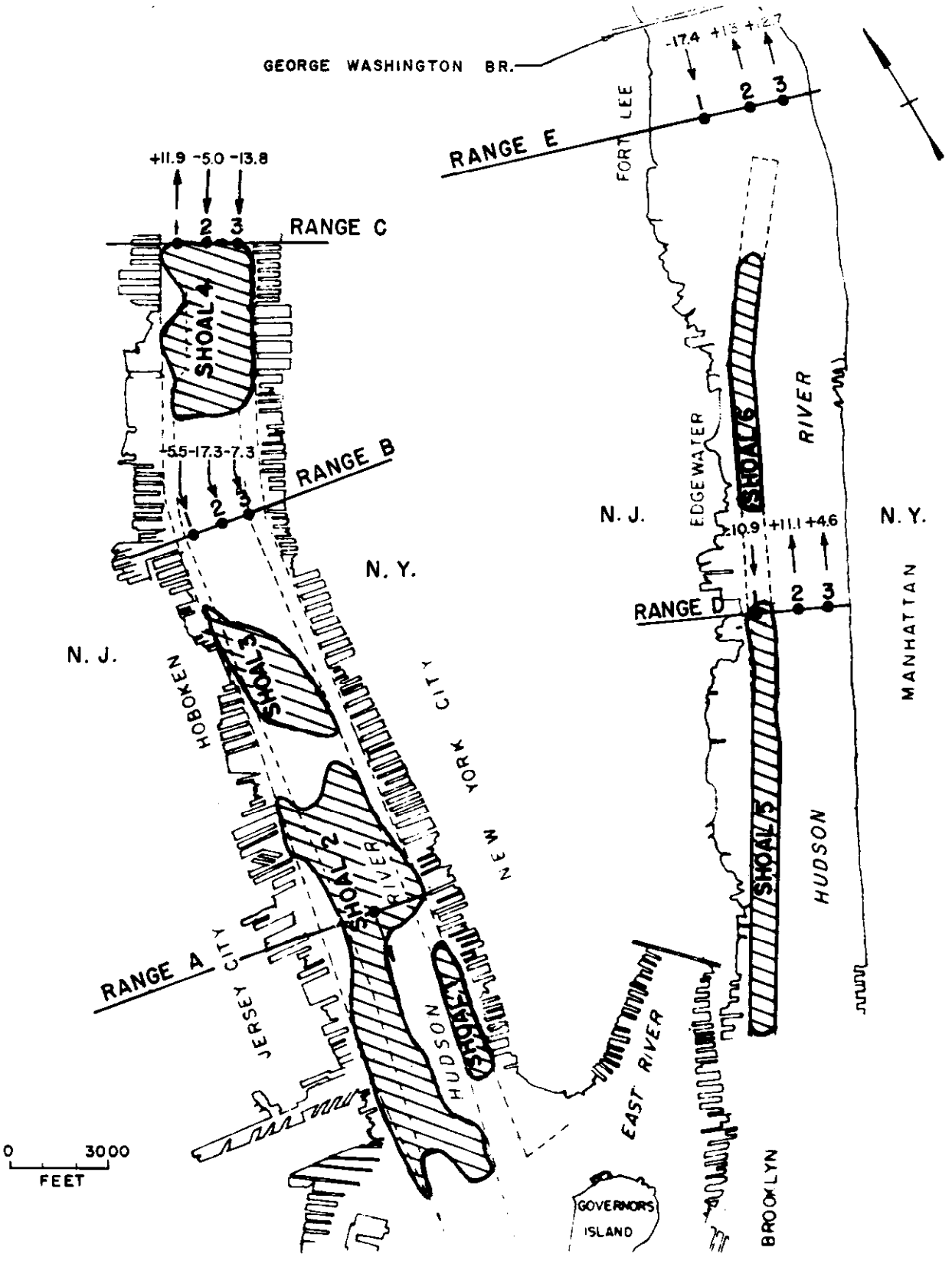


FIGURE 20. — Location of prototype sediment ranges.

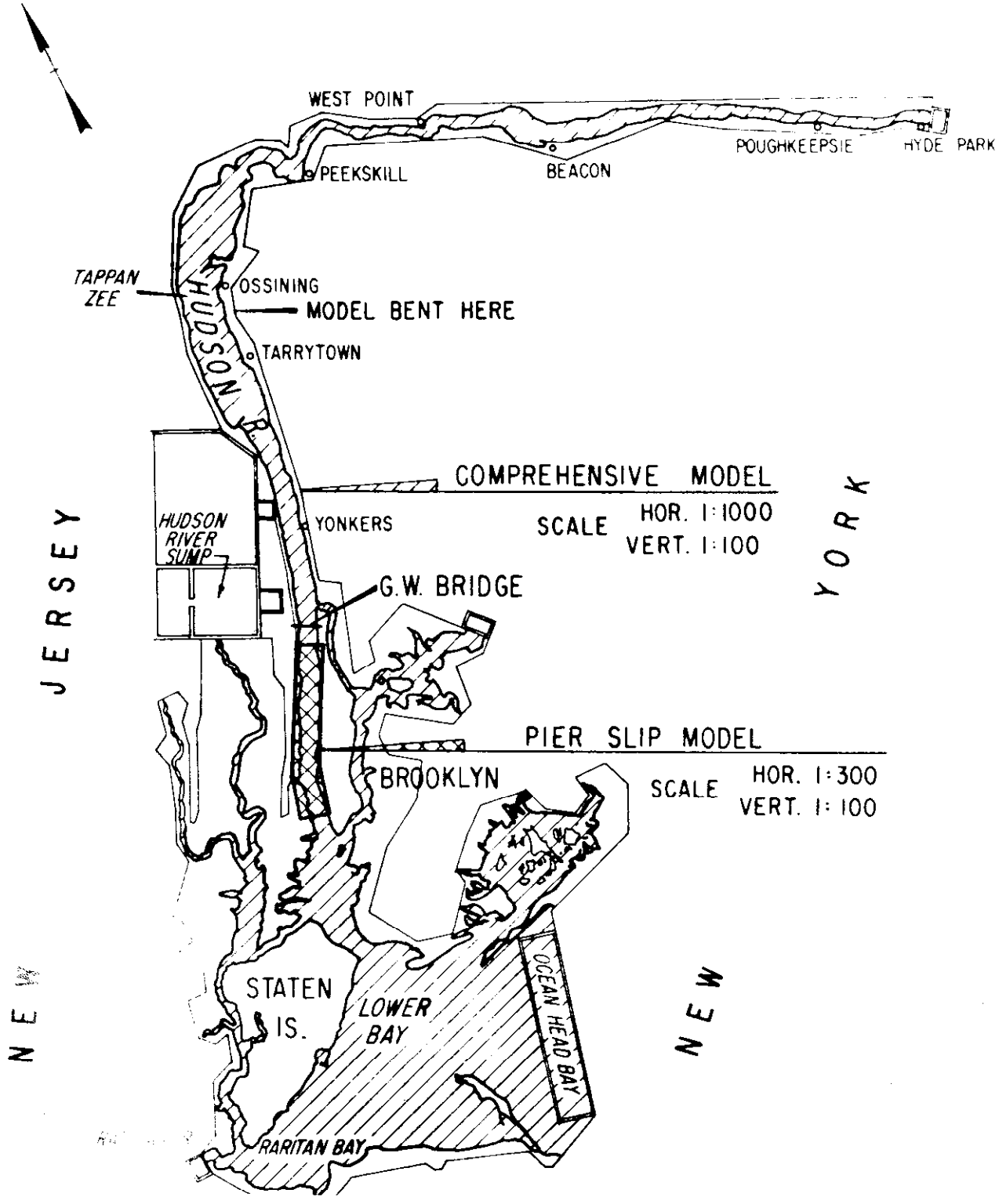


FIGURE 21. — Lower Hudson River model layouts.

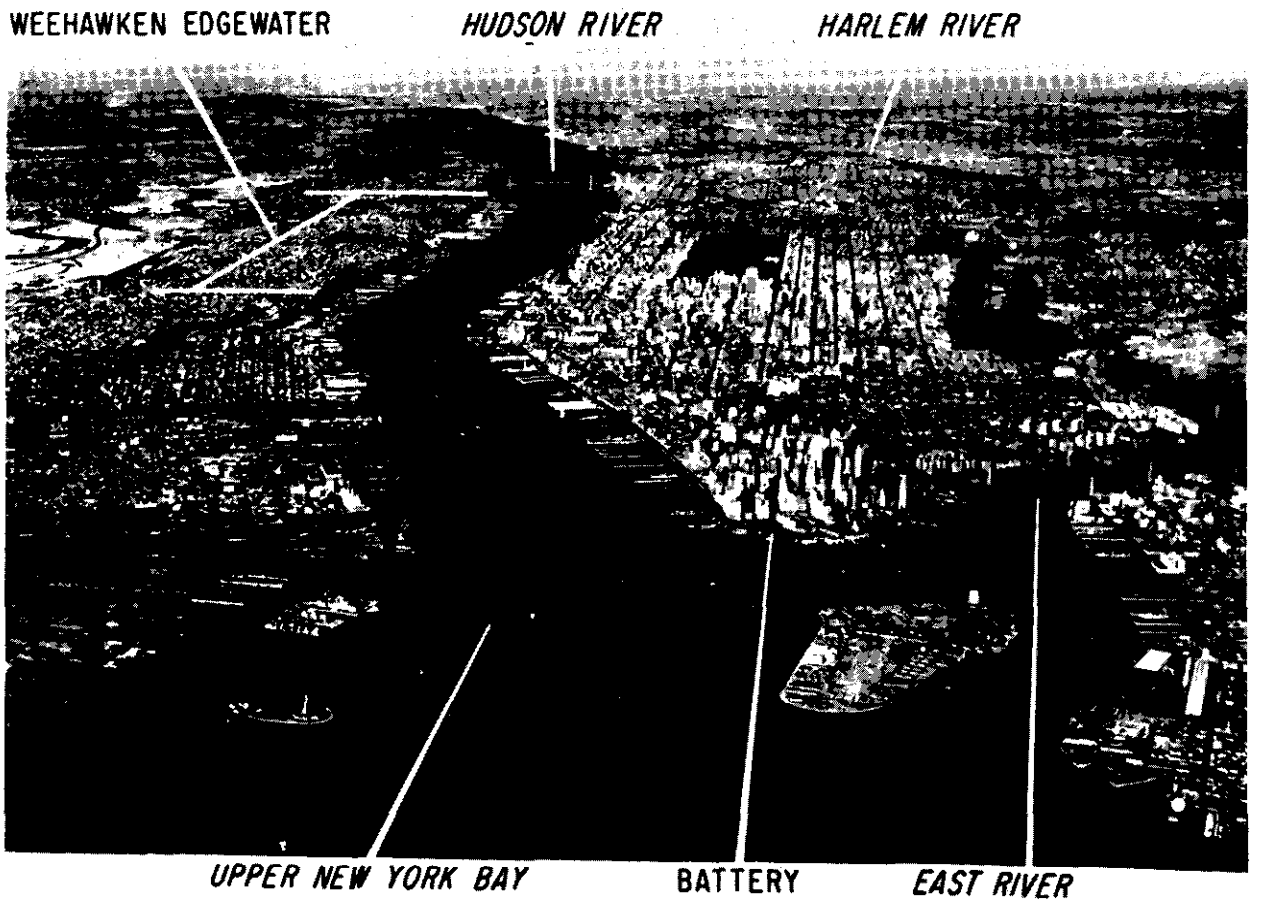


FIGURE 22. — Aerial photograph of lower New York Harbor.

used to test the effectiveness of several different types of proposed channel improvement plans, in decreasing the shoaling in the channel and slips. The larger scale model of the limited area was used to test the effectiveness of certain proposed slip improvement plans for decreasing the shoaling of the slips.

The comprehensive model is quantitatively accurate as to the reduction of the tides, the tidal currents, salinity distribution, and the complex relations of bottom and top currents that occur in tidal estuaries as a result of the interplay of salt water and fresh water. The section model is quantitatively accurate with respect to hydraulic conditions within the pier slips. Fresh water was used in the section model, since density current phenomena do not affect the pier slip shoaling problem in the same degree as the channel shoaling. The techniques used to reproduce the shoaling tendencies in the models are considered qualitatively accurate and to be susceptible of reasonably quantitative interpretation. This statement is based upon the facts that (1) the same technique (of using gilsonite as a model shoal material) has been confirmed to be accurate in models of many estuaries, and (2) an accurate verification of the Lower Hudson River models was achieved. It is also considered that the model shoaling tendencies are accurate with respect to the prototype, regardless of the lack of precise knowledge as to the source of all of the material ending up as shoals in the channels and pier slips. The reason for this statement lies in the basic hydraulic mechanism of the harbor as previously described.

For the comprehensive model, a base test was established by conducting a shoaling test with the existing waterway so as to produce measured annual conditions of shoaling in nature, identified as shoal areas 1 to 6, fig. 6. The material was then retrieved from the channel and slips and measured as the base index.

The comprehensive model base tests were conducted at various fresh water flows ranging from 3,000 to 40,000 cubic feet per second. A flow of 12,000 cubic feet per second, which is approximately the median flow of the Lower Hudson River, was used as the base for all tests unless otherwise noted. An identical test was conducted with the improvements in place, and the material was retrieved from the waterway bottom. The effectiveness of each plan was

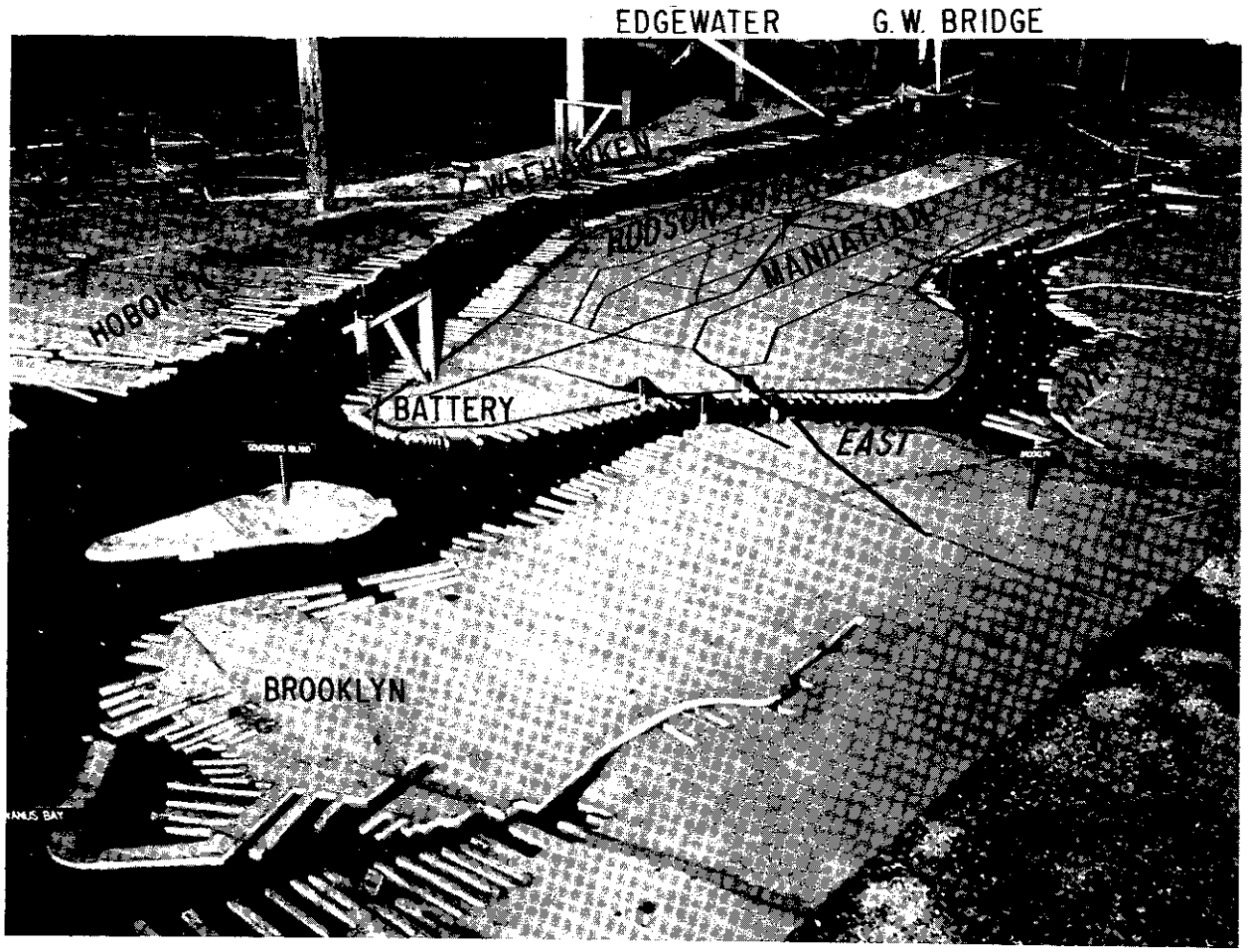


FIGURE 23.—Photograph of comprehensive model, lower New York Harbor.

evaluated on the basis of the reduction or increase in the amount of material retrieved from the channel and slips. This was recorded as a shoaling index, which is the ratio of the volume of material shoaled during the base test to the volume shoaled as a result of the plan of improvement. An index of unity indicates no change after improvement, an index less than unity indicates a reduction in shoaling, and an index greater than unity indicates an increase in shoaling. The quantity of shoaling as a result of each plan is the product of the prototype volume and the index.

Model Investigations

The basic philosophy in the reduction of dredging costs in the Lower Hudson River channels is that it could be effected in three ways, alone or in combination. The first of these would involve effecting physical changes that would alter the hydraulic mechanism so that sediments, in whole or in part, would be carried and deposited outside of the navigation facilities. The second would involve reduction of sediment entering the Lower Hudson River. The third would be the deposition of the sediment in such a way that it could be removed with a reduction in dredging costs. The plans tested in the models would fall under the first and third method.

In the comprehensive model 28 plans or variations thereof have been tested to date (table 6). Twelve of these plans involve sedimentation basins at various locations and depths (fig. 24). Two plans involve the closure of the Harlem River (fig. 25). Two plans involve the deepening of the shallow channel areas in the Tappan Zee and Haverstraw Bay Areas (fig. 26). One plan provides for a wing dike enclosing the 30-foot channel (fig. 27). Two plans provide for restoration of the pre-1941 channel cross section at the George Washington Bridge for comparison with present conditions and for determining the effect on shoaling during the period

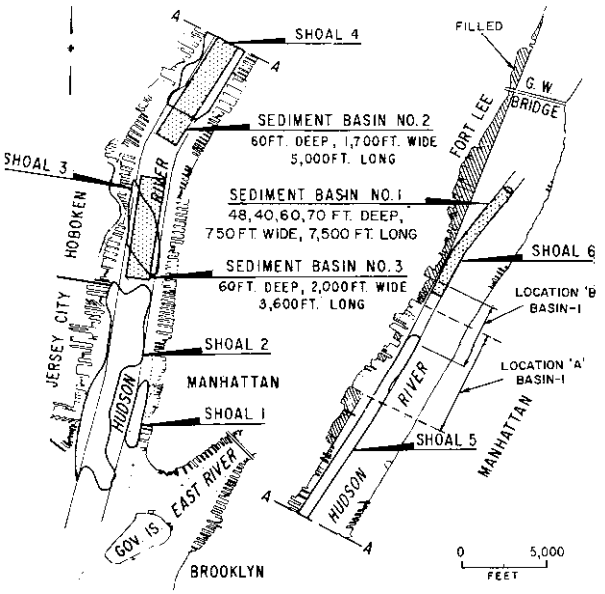


FIGURE 24. — Comprehensive model of basin series.

where dumping occurred at the bridge between 1941 and 1946.

Five plans provide for modifying the cross sections of the river at George Washington Bridge, with and without a dike parallel to the New Jersey shore (figs. 28 and 29). Four plans provide for enlarging and deepening to 50 feet the 30-foot project channel along the west, New Jersey side, and filling or constructing submerged dikes on the east, New York side of the river (fig. 30).

In the sectional (pier slip) model, 11 plans have been tested as listed in table 7 and shown on figure 31. Nine of these plans considered methods of reducing slip shoaling by physical changes to affect the shoaling mechanism by use of submerged dikes and air and water screens. Two plans considered the reduction in cost of dredging slip shoals by spoiling the material from slips into the navigable channel for removal by Federal dredging operations and by measuring the amount of material redistributed over the channels and returned to the slips during the dredging procedure.

Model Results

The results of model investigations expressed in terms of shoaling indices are summarized in tables 8 through 11 for the comprehensive model and in tables 12 and 13 for the sectional model.

The results on sediment basin indicate an increase in deposition in the basin area and a reduction in deposition in adjacent channel areas and slips previously subjected to heavy shoaling if the basin is placed in the appropriate location. The most effective basin tested was sediment basin No. 1, located at the heaviest point of shoaling, shoal 6, which is at the upstream end of the 30-foot project (fig. 24). Some plans, table 8, showed reduction in channel shoaling of more than 50 percent and in slip shoaling of 20 percent. However, plans designed for reduction of channel shoaling are not to be expected to reduce slip shoaling by more than 20 to 25 percent, therefore, a major shoaling problem would still exist in the slips after the best possible plan of channel improvement has been effected.

The complete closure of the Harlem River (fig. 25) during ebbtide discharge into the Hudson River showed significant effect in reducing shoaling in the channels by 40 percent and in slips by 32 percent (table 9). However, from the standpoint of navigation, a closure of the river would require provisions to permit the maintenance of navigation.

The deeper channels in Haverstraw Bay (fig. 26) were ineffective in reduction of shoaling (table 9) because of the local interruption of upstream predominance at the bottom flow downstream of the George Washington Bridge. These tests would probably be more effective if the local interruption of upstream predominant bottom flow could be eliminated by corrective action at the bridge and closure of the Harlem River.

Several plans of compensating dredging to approach the 1941 section at the George Washington Bridge (figs. 28 and 29) and to reduce local turbulence, showed a substantial reduction in channel and slips shoaling up to 40 percent (table 10). The variation of fresh water flow with a plan involving only compensating dredging at the bridge (plan 27) resulted in proportional variation of the shoaling (fig. 29 and table 11). Several plans involving compensating dredging at the bridge and realinement of a deep channel on the New Jersey side and a shallow channel on the New York side showed some promise (fig. 30 and table 10). These tests show that a proper enlargement of cross section at the bridge encourages upstream predominant bottom currents to carry sediment upstream of the bridge. The realinement of deep channel from the New York to the New Jersey side with construction of a sill or fill in the naturally deep water on the New York side would encourage upstream predominant currents in the deepened New Jersey channel so as to move sediment progressively upstream of the bridge.

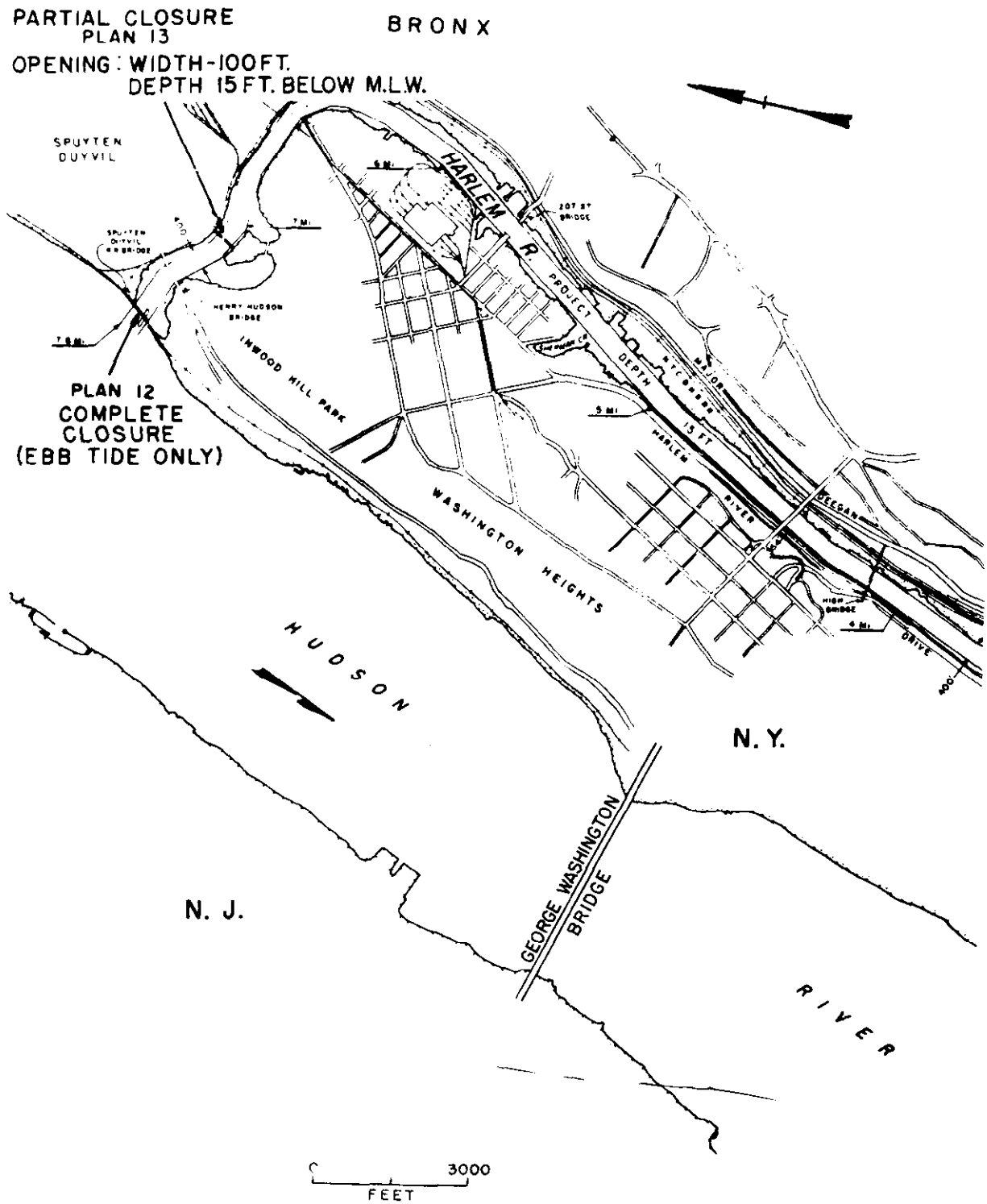


FIGURE 25. — Comprehensive model of Harlem River series.

TABLE 6.—*Lower Hudson River, comprehensive model, channel improvement plans*

SEDIMENTATION BASIN SERIES (fig. 24)

Plan No.	Basin No.	Length	Width	Depth	Capacity	Remarks
		Ft.	*Ft.*	*Ft.*	*Million cubic yards*	
1	1	7,500	750	48	4.0	
2	2	5,000	1,750	60	4.0	
3	3	3,600	2,000	60	4.0	
4	1 and 2				8.0	
5	1, loc A	7,000	750	70	9.1	End slopes 1 on ½.
6	1, loc A	7,000	750	70	9.4	End slopes 1 on 5.
7	1, loc C	4,350	750	70	5.7	End slopes 1 on 1.
8	1	7,500	750	40	2.2	
9	1	7,500	750	55	5.8	
10	1	7,500	750	70	9.8	Varying depth.
11	1	7,500	750	70	9.8	Fill to U.S. pier and bulkhead line.
28	1D	7,000	750	70	9.2	Divided at pipeline crossing.
29	1D	7,000	750	50	4.2	

HARLEM RIVER SERIES (fig. 25)

Plan No.	Description
12	Complete closure for Harlem River Ebbflow.
13	Partial closure, opening: width—100 feet; depth −15 feet mean low water.

HAVERSTRAW BAY SERIES (fig. 26)

14	Channel, Tappan Zee and Haverstraw Bay, −35 feet mean low water, 600 feet wide.
15	Channel, Tappan Zee and Haverstraw Bay, −40 feet mean low water, 1,000 feet wide.

WING DIKE SERIES (fig. 27)

16	Dike: 13,600 feet long, top elevation at mean high water from Fort Lee to and along 30-foot project.

GEORGE WASHINGTON BRIDGE SERIES (figs. 28 and 29)

17	Restoration of 1941 section prior to spoil dumping; excavation 7.5 million cubic yards.
18	Plan 17 with spoil simulation, 1942–45.
19	Compensatory dredging to restore 1941 section with channel cut; 6,800 feet long, 200 to 1,800 feet wide, 40 to 70 feet below mean low water, 5.0 million cubic yards.
20	Adjusted cross section: 10,200-foot dike, Fort Lee to Edgewater; channel 16,400 feet long, 500 to 1,100 feet wide, 57 to 60 feet deep; dredge 10.8 million cubic yards.
21	Adjusted cross section: 10,500-foot dike, Fort Lee to Edgewater; channel 10,000 feet long, 500 to 1,300 feet wide, 55 to 70 feet deep; dredge 7.1 million cubic yards.
22	Adjusted cross section: 10,725-foot dike, Fort Lee to Edgewater; channel equal to plan 21.
27	Similar to plans 20 and 21 without dike.

REALINED CHANNEL SERIES (fig. 30)

23	Deepen 30-foot channel to 50 feet below mean low water, widen to 1,000 feet, and extend to 900 feet upstream of George Washington Bridge; construct 13 submerged sills on New York side with top elevation 20 to 40 feet below mean low water.
24	Same as plan 23 except place solid fill to 30 feet below mean low water on New York side.
25	Same as plan 23 except extend 50-foot channel to 4,500 feet upstream of the bridge with maximum depths to 63 feet below mean low water and width of 1,400 feet; only a single 1,500-foot dike across New York side at West 134th Street, with top elevation at 30 feet below mean low water.
26	Same as plan 25 on New Jersey side and plan 24 on New York side.

Several plans tried in the section model (fig. 31 and table 12) showed that submerged wing dikes were ineffective, that submerged movable sills at appropriate elevations across the entrance of the slips would reduce shoaling appreciably, that an air bubble screen across the entrance or longitudinally in the slip would result in appreciable reduction of shoaling, and that a water bubble screen was ineffective. Two other tests (table 13) that showed promise were those permitting the dumping of slip spoil into the channels to be removed by Federal dredging at local cost.

Because of the inconsistency of some results and the promising results shown by several plans to effect reduced channel and slip shoaling, a limited additional test program is scheduled to permit the economic selection of the most promising plan to reduce the shoaling in the Lower Hudson River.

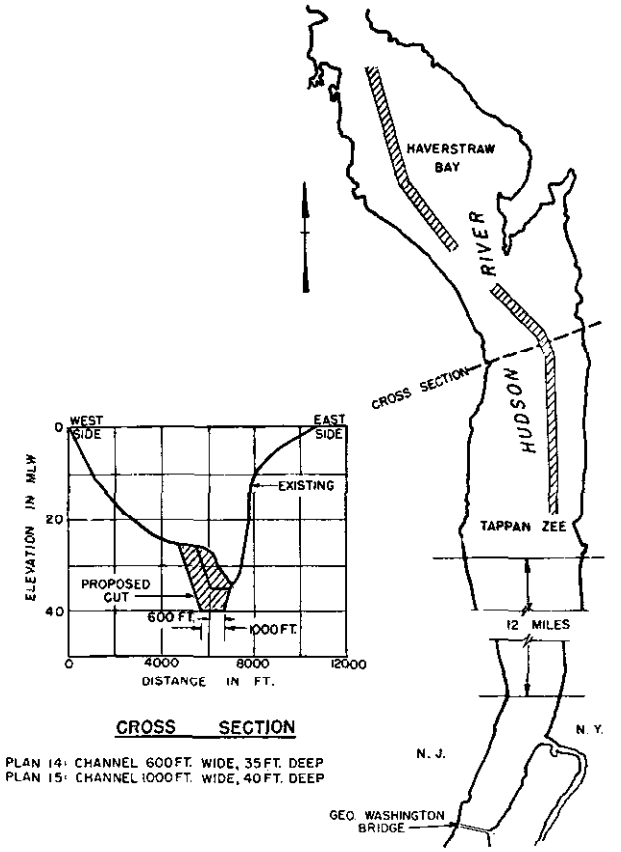


FIGURE 26. — Comprehensive model of Haverstraw Bay series.

Summary

The data presented point to following pertinent facts:

1. The Lower Hudson River is a partly mixed estuary that is a part of a complex estuarine system of the New York Harbor. This system does not lend itself to mathematical analysis and requires the use of the model tool to integrate the many influencing forces and factors.

2. The model studies have permitted a more complete understanding of the hydraulic and sediment mechanism affecting shoaling. This hydraulic and sediment mechanism is confirmed by the limited number of prototype measurements of salinity, velocities, and sediment concentration, even though variable. This better understanding is another tool to evaluate spoil and dredging operations that could affect the mechanism that results in adverse shoaling conditions.

3. The Hudson River Watershed and areas tributary to the New York Harbor are definite sources of shoal material. The part of the source that is susceptible to reduction would have little effect on the shoaling pattern.

4. These model studies have demonstrated ways of effecting reduction in channel and slip shoaling that are entwined with economics of cost of plan vs. benefits derived. The testing program cannot be predicted and fixed but must be fluid, so that it can be developed in phase with the results of the testing. Therefore, the testing program should be expanded as required until all the necessary factors have been developed. The results of the tests must be carefully examined and rechecked to assure the reliability of results.

5. The combination of limited prototype measurements supplemented by appropriate model study and analysis of results is a strong and economical tool in the study and understanding of estuary sediment problems that can lead to possible methods of solution.

Acknowledgments

The material presented herein is the result of studies conducted by the U.S. Army Engineer District, New York, in cooperation with U.S.

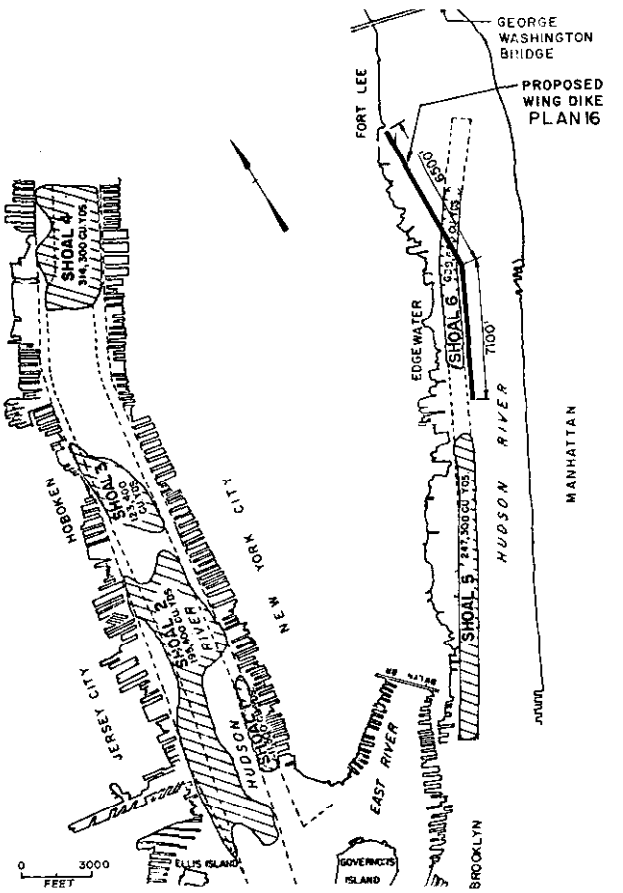


FIGURE 27. — Comprehensive model of Wing dike.

TABLE 7.—*Lower Hudson River sectional model pier slip improvement plans (fig. 31)*

Plan No.	Description
1....	Submerged dike across slip entrance, with top elevation at 35 feet below mean low water.
2....	Same as plan 1, with top elevation at 5 feet below mean low water.
3....	Wind Dike: 100-foot extension of pier 92, 20 feet deep, with top elevation at mean low water.
4....	Same as plan 3, except top elevation at 35 feet below mean low water.
5....	Same as plan 3, except dike extension of pier 90.
6....	Submerged air screen across slip entrance, with top elevation of conduit at 45 feet below mean low water (discharge, 37,500 c.f.m.).
7....	Same as plan 6, except water screen (discharge, 18,000 c.f.m.).
8....	Air bubble turbulence generator consisting of two parallel, longitudinal pipes at ⅓ points, with top elevation 45 feet below mean low water (discharge, 232,500 c.f.m.).
9....	Same as plan 8 (discharge, 30,000 c.f.m.).
10....	Simulated removing shoal in slip 90-92 by a bucket dredge filling a 1,750 cubic yard barge every 6 hours and dumping at mid-navigable channel in front of slip. Operation continued until 74,500 cubic yards (half annual shoaling) were removed.
11....	Simulated removal of same volume of shoal as in plan 10 by a hydraulic dredge discharging at the surface at 500 feet toward the channel from the face of pier 92 equal to 9 prototype days.

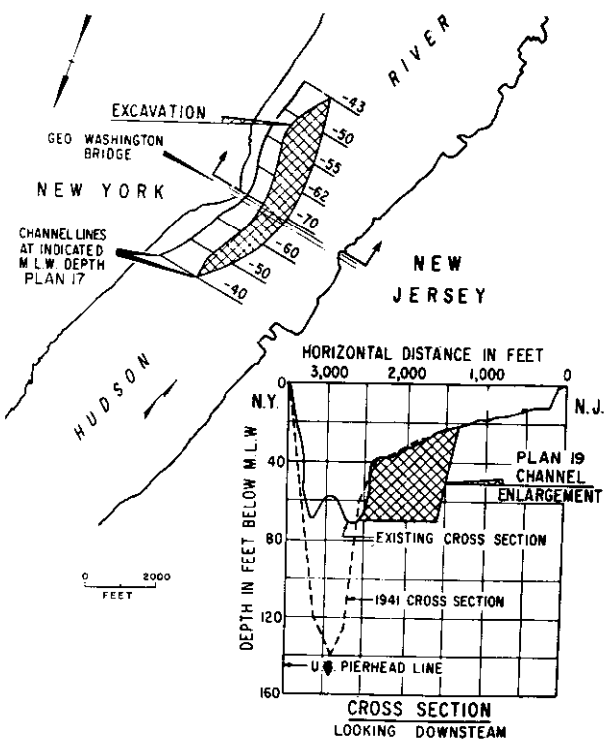


FIGURE 28. — Comprehensive dredging to 1941 cross section.

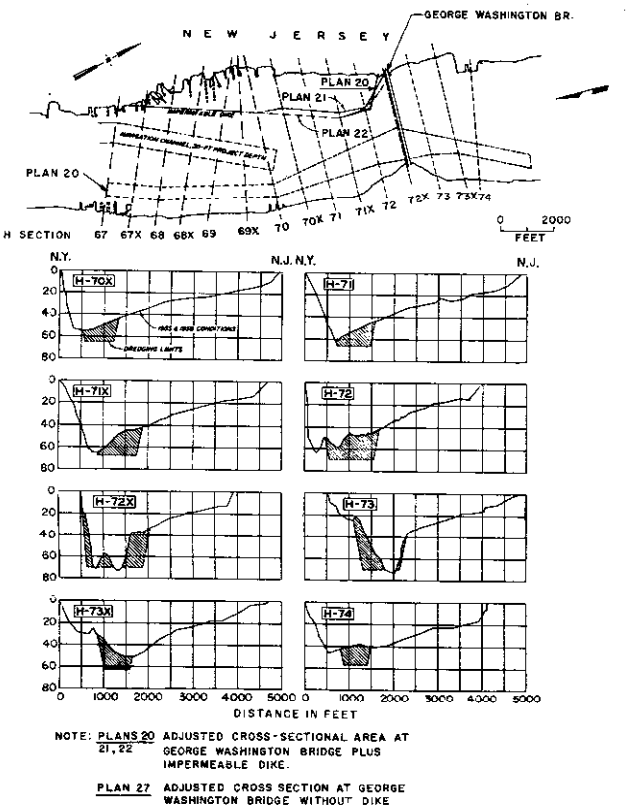
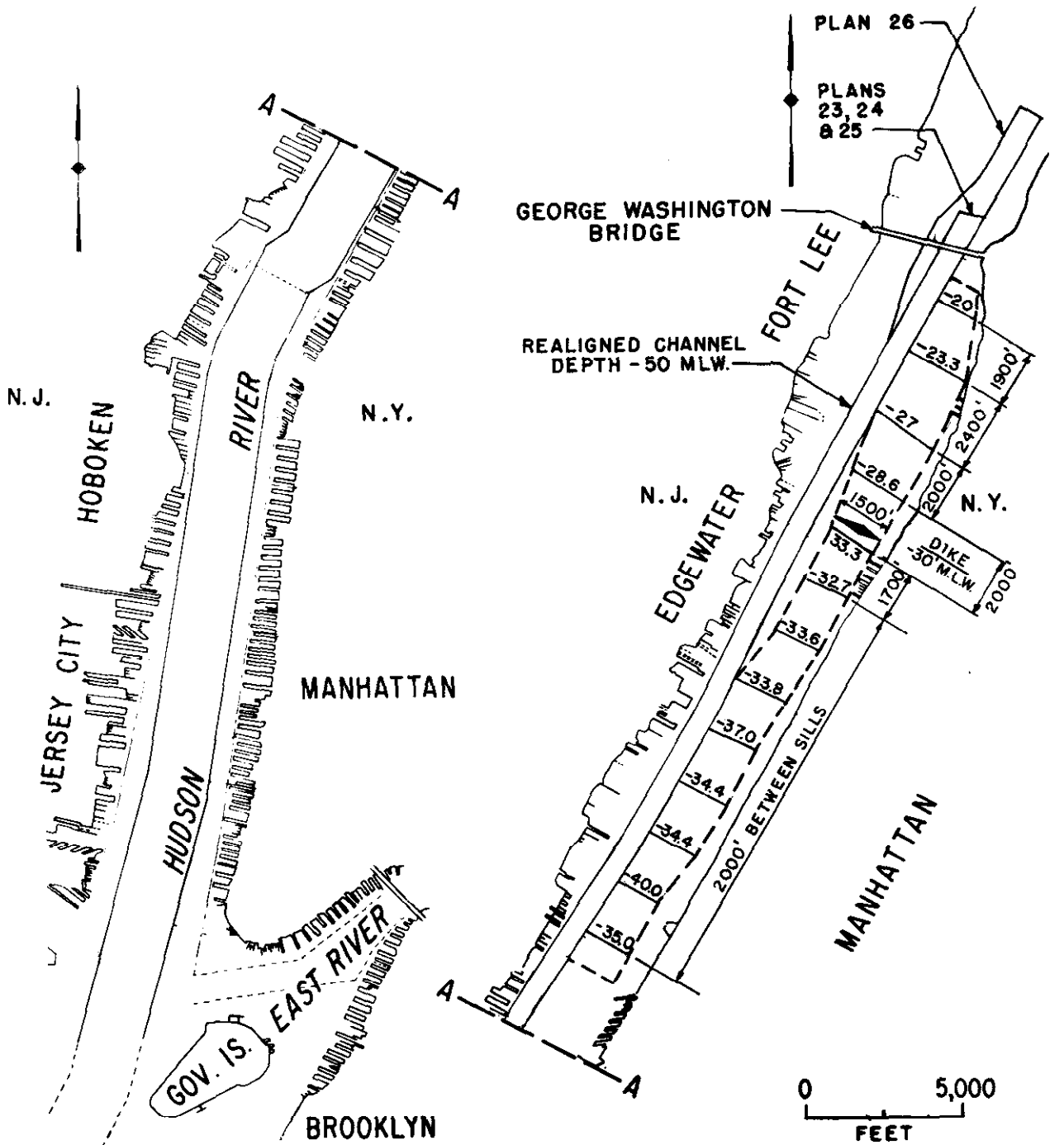


FIGURE 29. — Comprehensive model of George Washington Bridge series.

Army Engineer Waterways Experiment Station, Vicksburg, Miss. The writer acknowledges the leadership and direction of Col. Milton M. Miletich, Col. Charles M. Duke, and Brig. Gen. Thomas De F. Rogers who served as U.S. Army District Engineers, and the technical guidance of Charles K. Panish, Chief of the Engineering Division of the New York District; and Col. Alex G. Sutton, Jr. and Col. Edmund H. Lang, who served as directors, and Joseph B. Tiffany, technical director of the Waterways Experiment Station. Special acknowledgment is due to Henry B. Simmons, chief of the Estuaries Section of the Waterways Experiment Section, who was in direct charge of conducting the model studies. The writer also acknowledges the assistance in the research work and preparation of this paper of the following District personnel: Isiah Katcher, Aaron J. Addis, Melvin Mark, Charles Beatty, and Joseph Tennen.



PLAN 23 SUBMERGED SILLS TOP ELEV. -32 FT. M.L.W.
PLAN 24 CHANNEL REALIGNMENT WITH SOLID FILL ---
PLAN 25 CHANNEL REALIGNMENT WITH SINGLE SUBMERGED DIKE
PLAN 26 CHANNEL REALIGNMENT WITH SOLID FILL & ADJUSTED CROSS-SECTION AT BRIDGE

(ALL PLANS WITH REALIGNED N.J. CHANNEL - DEPTH - 50' MLW)

FIGURE 30. — Plan for realined channel.

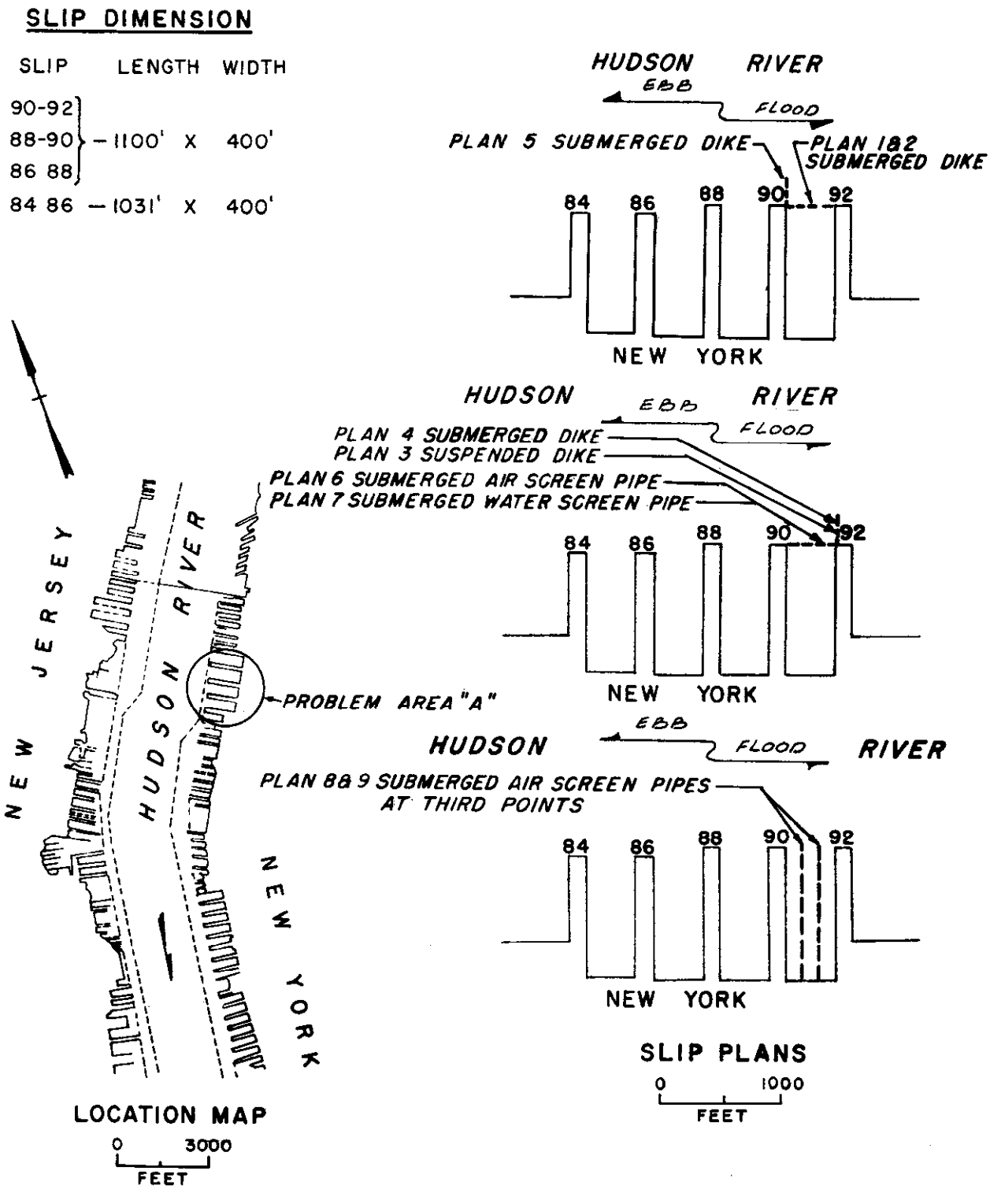


FIGURE 31. — Sectional pier slip model for shoaling studies.

TABLE 8.—*Lower Hudson River comprehensive model studies: sediment basin series*

Channel	Prototype	Plan 1, Basin No. 1		Plan 2, Basin No. 2		Plan 3, Basin No. 3		Plan 4, Basin Nos. 1 and 2	
	Cu. yd.	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*
Shoal areas:									
1	11,300	0.44	5,000	0.78	8,800	1.67	18,900	0.89	10,100
2	195,400	.91	177,800	1.37	267,700	.95	185,600	.92	179,800
3	123,400	.44	54,300	.76	93,800	.44	54,300	.50	61,700
4	314,300	1.07	336,300	.43	135,700	.51	160,300	.41	128,900
5	247,300	.31	76,700	.56	138,500	.83	205,300	.82	202,800
6	689,600	.....	.....	.73	503,400	.64	441,300	.....	.....
Shoals	1,581,300	.41	650,100	.73	1,147,900	.67	1,065,700	.37	583,300
Basins	.....	1.28	882,700	1.08	339,400	.64	79,000	.....	[1]1,654,900
Subtotal	1,581,300	.97	1,532,800	.94	1,487,300	.72	1,144,700	1.41	2,238,200
Slip areas:									
New York side	1,062,300	.98	1,041,100	.72	764,900	.81	860,500	.81	860,500
New Jersey side	1,937,700	.94	1,821,400	1.21	2,344,600	1.44	2,790,300	.97	1,879,600
Subtotal	3,000,000	.95	2,862,500	1.04	3,109,500	1.22	3,650,800	.91	2,740,100
Total	4,581,300	.96	4,395,300	1.00	4,596,800	1.05	4,795,500	1.09	4,978,300

Channel	Prototype	Plan 5, Basin No. 1, loc. A, End slope 1 on ½		Plan 6, Basin No. 1, loc. A, End slope 1 on 5		Plan 7, Basin No. 1, loc. C, End slope 1 on 1	
	Cu. yd.	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*
Shoal areas:							
1	11,300	1.50	17,000	0.75	8,500	0.75	8,500
2	195,400	1.12	218,800	1.06	207,100	1.41	275,500
3	123,400	1.08	133,300	1.17	144,400	1.83	225,800
4	314,300	.78	245,200	.89	279,700	1.40	440,000
5	247,300	.87	215,200	.67	165,700	2.33	576,200
6	689,600	.29	200,000	.08	55,200	.29	200,000
Shoals	1,581,300	.65	1,029,500	.54	860,600	1.09	1,726,000
Basins	.....	.....	1,687,000	.....	318,300	.....	530,500
Subtotal	1,581,300	1.71	2,716,500	.75	1,178,900	1.43	2,256,500
Maintained slips:							
New York side	1,062,300	.48	509,900	.86	913,600	.88	934,800
New Jersey side	1,937,700	.80	1,550,200	.93	1,802,100	1.42	2,751,500
Subtotal	3,000,000	.69	2,060,100	.91	2,715,700	1.23	3,686,300
Total	4,581,300	1.04	4,776,600	.83	3,894,600	1.30	5,942,800

Channel	Prototype	Plan 8, Basin No. 1 Depth = −40 ft.		Plan 9, Basin No. 1 Depth = −55 ft.		Plan 10, Basin No. 1 Depth = −70 ft.		Plan 11, Basin No. 1 Depth = −70 ft.	
	Cu. yd.	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*
Shoal areas:									
1	11,300	0.75	8,500	1.50	17,000	0.75	8,500	0.75	8,500
2	195,400	1.34	261,800	1.34	261,800	1.40	273,600	1.14	222,800
3	123,400	1.96	241,900	1.79	229,900	2.12	261,600	2.62	323,300
4	314,300	1.05	330,000	1.42	446,300	1.33	418,000	1.77	556,300
5	247,300	.83	205,300	.70	173,100	.73	180,500	1.10	272,000
6	689,600	.....	.....	.....	.....	.....	.....	.....	.....
Shoals	1,581,300	.66	1,047,500	.71	1,119,100	.72	1,142,200	.87	1,382,900
Basins	.....	3.36	2,317,100	4.31	2,972,200	5.70	3,930,700	5.46	3,765,200
Subtotal	1,581,300	2.13	3,364,600	2.58	4,991,300	3.21	5,072,900	3.25	5,148,100
Slip areas:									
New York side	1,062,300	.53	563,000	.74	786,100	.78	828,600	.62	658,600
New Jersey side	1,937,700	1.20	2,325,200	.78	1,511,400	.90	1,743,900	.68	1,317,600
Subtotal	3,000,000	.96	2,888,200	.77	2,297,500	.86	2,572,500	.66	1,976,200
Total	4,581,300	1.36	6,252,800	1.39	6,388,800	1.67	7,645,400	1.56	7,124,300

[1] Total in basins 1 and 2: basin No. 1, 1,268,900 cu. yd., index 1.84; basin No. 2, 386,000 cu. yd., index 1.23.

TABLE 8.—*Lower Hudson River comprehensive model studies: sediment basin series—Continued*

Channel	Prototype	Plan 28 Basin No. 1 (divided)		Plan 29, Basin No. 1 (divided)			
				Depth = −50 ft. m.l.w. 3,000 c.f.s. discharge		Depth = −50 ft. m.l.w. 40,000 c.f.s. discharge	
Shoal areas:	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*
1	11,300	1.36	15,400	0.40	4,500	1.88	21,200
2	195,400	1.09	213,000	.67	131,000	.51	99,700
3	123,400	.94	116,000	.42	51,800	.45	35,500
4	314,300	1.04	326,900	.65	204,300	.41	128,900
5	247,300	1.09	269,600	.50	123,700	.38	94,000
6	689,600	.....	.....	.01	6,900	.05	34,500
Shoals	1,581,300	.80	940,900	.33	522,200	.27	433,800
Basins	.....	4.30	2,965,300	5.50	[2] 3,792,800	8.65	[3] 5,965,000
Subtotal	1,581,300	2.47	3,906,200	2.73	4,315,000	4.04	6,398,800
Slip areas:							
New York side	1,062,300	.07	924,200	1.10	1,168,500	.99	1,051,700
New Jersey side	1,937,700	1.30	2,519,000	.72	1,395,100	.88	1,705,200
Subtotal	3,000,000	1.15	3,443,200	.85	2,563,600	.92	2,756,900
Total	4,581,300	1.60	7,349,400	1.50	6,878,600	2.00	9,146,700

[2] 57 percent in south part of basin; 43 percent in north part.
[3] 22 percent in south part of basin; 78 percent in north part.

TABLE 9.—*Lower Hudson River comprehensive model studies: Harlem, Haverstraw, realinement series and George Washington Bridge*

Channel	Prototype	Harlem River closure				Haverstraw Bay			
		Plan 12 Ebb closure		Plan 13 Partial closure		Plan 14 35-foot channel		Plan 15 40-foot channel	
Shoal areas:	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*
1	11,300	0.56	6,300	0.44	5,000	0.89	10,100	1.06	12,000
2	195,400	.80	156,300	.70	136,800	1.13	220,800	1.10	214,900
3	123,400	.62	76,500	.88	108,600	.78	96,300	1.63	201,100
4	314,300	1.14	358,300	.77	242,000	.96	301,700	1.29	405,400
5	247,300	.21	51,900	.79	195,400	.74	183,000	1.10	272,000
6	689,600	.42	289,600	1.14	786,100	1.09	751,700	.94	655,100
Subtotal	1,581,300	.59	938,900	.93	1,473,900	.99	1,563,600	1.11	1,760,500
Slip areas:									
New York side	1,062,300	.95	1,009,200	.76	807,300	1.23	1,306,600	.61	648,000
New Jersey side	1,937,700	.68	1,317,600	.89	1,724,600	1.45	2,809,700	.71	1,375,800
Subtotal	3,000,000	.78	2,326,800	.84	2,531,900	1.37	4,116,300	.68	2,023,800
Total	4,581,300	.71	3,265,700	.87	4,005,800	1.24	5,679,900	.83	3,784,300

Channel	Prototype	Realinement studies				Plan 25 Single sill		George Washington Bridge	
		Plan 23 With sills		Plan 24 Solid fill				Plan 26 Solid fill and changed section	
Shoal areas:									
1	11,300	1.00	11,300	0.56	6,300	0.67	7,600	0.33	3,700
2	195,400	1.00	195,400	.64	125,100	.65	127,000	.47	91,800
3	123,400	1.15	141,900	.60	74,000	.27	33,300	.31	38,300
4	314,300	1.40	440,000	.54	169,700	.19	59,700	.39	122,600
5	247,300	2.02	499,500	1.42	351,200	.83	205,300	.53	131,100
6	689,600	1.04	717,200	1.11	765,500	1.17	806,800	.40	275,800
Subtotal	1,581,300	1.27	2,005,300	.94	1,491,800	.78	1,239,700	.42	663,300
Slip areas:									
New York side	1,062,300	.65	690,500	.94	998,600	.75	796,700	.71	754,200
New Jersey side	1,937,700	.57	1,104,500	.65	1,259,500	.71	1,375,800	.64	1,240,100
Subtotal	3,000,000	.60	1,795,000	.75	2,258,100	.72	2,172,500	.66	1,994,300
Total	4,581,300	.83	3,800,300	.82	3,749,900	.74	3,412,200	.58	2,657,600

TABLE 10.—*Lower Hudson River comprehensive model studies: Dike and bridge series*

Channel	Prototype	Wing dike		George Washington Bridge series					
		Plan 16		Plan 17 1941 channel		Plan 18 [1] 1942-45 spoiling		Plan 19 Compensatory dredging	
Shoal Areas:	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*
1	11,300	1.89	21,400	0.56	6,300	0.80	5,000	0.44	5,000
2	195,400	1.00	195,400	1.19	232,500	1.08	251,100	.93	181,700
3	123,400	.98	120,900	1.70	209,800	.72	151,000	.63	77,700
4	314,300	1.35	424,300	1.11	348,900	1.40	488,500	.57	179,200
5	247,300	1.35	333,900	.51	126,100	2.19	276,200	.43	106,300
6	689,600	.94	648,200	.70	482,700	1.30	627,500	.48	331,000
Subtotal	1,581,300	1.10	1,744,100	.89	1,406,300	1.28	1,799,300	.56	880,900
Slip areas:									
New York side	1,062,300	1.19	1,264,100	1.35	1,434,100	1.01	1,072,900	.84	892,300
New Jersey side	1,937,700	1.39	2,693,400	.70	1,356,400	2.22	4,301,700	1.11	2,150,800
Subtotal	3,000,000	1.32	3,957,500	.93	2,790,500	1.79	5,374,600	1.01	3,043,100
Total	4,581,300	1.24	5,701,600	.92	4,196,800	1.57	7,173,900	.86	3,924,000

Channel	Prototype	Adjusted cross-sectional area at George Washington Bridge					
		Plan 20 Dike plan 1		Plan 21 Dike plan 2		Plan 22 Dike plan 3	
	Cu. yd.	*Index*	*Cu. yd.*	*Index*	*Cy. yd.*	*Index*	*Cu. yd.*
Shoal areas:							
1	11,300	0.44	5,000	0.44	5,000	0.67	7,600
2	195,400	1.46	285,300	1.44	281,400	1.39	271,600
3	123,400	1.66	204,800	1.05	129,600	1.48	182,600
4	314,300	1.20	377,200	1.09	342,600	1.23	386,600
5	247,300	.50	123,700	.74	183,000	.92	227,500
6	689,600	.59	406,900	.62	427,600	.37	255,200
Subtotal	1,581,300	.89	1,402,900	.87	1,369,200	.84	1,331,100
Maintained slips:							
New York side	1,062,300	.....	.....	.....	.....	.62	658,600
New Jersey side	1,937,700	.....	.....	.....	.....	.63	1,220,800
Subtotal	3,000,000	.....	.....	.....	.....	.63	1,879,400
Total	4,581,300	.....	.....	.....	.....	.70	3,210,500

[1] Plan 17 quantities used as base quantities for computing indices for plan 18.

TABLE 11.—*Lower Hudson River comprehensive model studies: Variable fresh water flow series*

Channel [1]	Prototype	George Washington Bridge series					
		Plan 27: no dike FWD [2] =3,000 c.f.s.		Plan 27: no dike FWD [2] =12,000 c.f.s.		Plan 27: no dike FWD [2] =40,000 c.f.s.	
Shoal areas:	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*
1	11,300	0.86	8,700	0.09	1,000	3.12	35,300
2	195,400	.95	186,600	.84	164,100	.62	121,100
3	123,400	.75	92,600	1.37	169,100	.58	71,600
4	314,300	.79	248,300	1.35	424,300	.55	172,900
5	247,300	.42	103,900	.59	145,900	.31	76,700
6	689,600	.69	475,800	.61	420,700	1.42	979,200
Subtotal	1,581,300	.71	1,115,900	.84	1,325,100	.92	1,456,800
Slip areas:							
New York side	1,062,300	.84	892,300	.90	956,100	.67	711,700
New Jersey side	1,037,700	.61	1,182,000	.78	1,511,400	.56	1,085,100
Subtotal	3,000,000	.69	2,074,300	.82	2,467,500	.60	1,796,800
Total	4,581,300	.70	3,190,200	.83	3,792,600	.71	3,253,600

Channel [1]	Base test series					
	Low flow: Hudson River discharge = 3,000 c.f.s.		*Median flow:* Hudson River discharge = 12,000 c.f.s.		*High flow:* Hudson River discharge = 40,000 c.f.s.	
Shoal areas:	*Vol. in cc.*	*Index*	*Vol. in cc.*	*Index*	*Vol. in cc.*	*Index*
1	50	2.27	22	1.0	32	1.45
2	1,165	1.82	640	1.0	816	1.27
3	480	1.92	250	1.0	816	3.26
4	1,305	1.55	840	1.0	2,614	3.11
5	120	.71	170	1.0	786	4.62
6	360	.49	740	1.0	489	.66
Subtotal	3,480	1.31	2,662	1.0	5,553	2.09
Slip areas:						
New York side	345	.86	400	1.0	416	1.04
New Jersey side	180	1.33	135	1.0	68	.50
Subtotal	525	.98	535	1.0	484	.90
Total:						
Maintained areas	4,005	1.25	3,197	1.0	6,037	1.89
Problem areas	8,309	1.14	7,263	1.0	15,839	2.18

[1] These totals for channel areas do not include maintenance of the plan channel in the vicinity of the bridge.

[2] FWD = fresh water discharge.

TABLE 12.—*Lower Hudson River sectional (pier slip) model studies: slip modification series*

Slip No.	Prototype shoaling	Plan 1 Entrance sill to −35 ft. m.l.w.		Plan 2 Entrance sill to −5 ft. m.l.w.		Plan 3 Suspended dike North side slip 90–92		Plan 4 Submerged dike North side slip 90–92		Plan 5 Submerged dike South side slip 90–92	
	Cu. yd.	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*
84–S	13,740	0.91	12,500	1.60	22,000	1.41	19,400	1.66	22,800	1.76	24,200
84–86	67,458	1.00	67,500	1.34	90,400	1.25	84,300	1.29	87,000	1.40	94,400
86–88	50,474	1.10	55,500	1.65	83,300	1.70	85,800	2.12	107,000	1.87	94,400
88–90	80,518	.82	66,000	1.21	97,500	1.33	107,100	1.59	128,000	1.57	126,400
90–92	149,301	.64	95,600	.49	73,200	1.25	186,600	1.71	255,300	1.68	250,900
92–N	17,312	1.06	18,400	1.37	23,700	1.90	32,900	1.74	30,100	2.14	37,000
Total	378,803	.83	315,500	1.03	390,100	1.36	516,100	1.66	630,200	1.65	627,300

Slip No.	Prototype shoaling	Plan 6 Entrance air screen 37,500 c.f.m. at +6.5 p.s.i.		Plan 7 Entrance water screen 300 c.f.s.		Plan 8 Longitudinal air jet 232,500 c.f.m. at +6.5 p.s.i.		Plan 9 Longitudinal air jet 30,000 c.f.m. at +2.6 p.s.i.	
	Cu. yd.	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*	*Index*	*Cu. yd.*
84–S	13,740	1.52	20,900	1.20	16,500	1.30	17,900	1.20	16,500
84–86	67,458	1.22	82,300	1.16	78,300	1.39	93,800	1.37	92,400
86–88	50,474	1.62	81,800	.89	44,900	1.39	70,200	1.40	70,700
88–90	80,518	1.33	107,100	.92	74,100	1.40	112,700	1.35	108,700
90–92	149,301	.84	125,400	1.08	161,200	.04	6,000	.78	116,500
92–N	17,312	1.03	17,800	1.38	23,900	1.36	23,500	1.33	23,000
Total	378,803	1.15	435,300	1.05	398,900	.85	324,100	1.13	427,800

TABLE 13.—*Lower Hudson River sectional (pier slip) model studies: effects of dredging operations on slip shoaling*

Slip No.	Volume retrieved in cubic yards: Prototype	
	Bucket dredge and barge operation	Hydraulic dredge and pipeline operation
Slips on New York Side:		
84—South	84	93
84—86	264	264
86—88	300	264
88—90	360	248
90—92	460	295
92—North	86	155
Total	1,554	1,319
83—North	36	24
112—113	24	15
113—114	26	21
Slips on New Jersey Side:		
149 (north and south)	64	[1]
151 (north and south)	115	[1]
188 (to pierhead line)	576	[1]

[1] No significant deposits found on the New Jersey side after the hydraulic dredging operation.

SEDIMENT MOVEMENT AS DEFINED BY RADIOACTIVE TRACERS: A PRELIMINARY REPORT[1]

[Paper No. 60]

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Introduction

An investigation of utilizing radioactive tracers in laboratory and field studies of sediment transport is now in progress at the Sedimentation Laboratory. The preliminary results of this investigation are discussed herein.

The use of radioactive tracers in sediment transport investigations has gained widespread interest in recent years. Several investigations,

[1] In cooperation with the University of Mississippi and Mississippi State University.

utilizing a variety of radioisotopes, have been conducted along coasts, in estuaries, and in maritime rivers. Considerable information has resulted from these studies. The reader is referred to the Proceedings of the 20th Permanent International Navigation Congress in Baltimore, 1961, Section II, Subject V, for the details of these investigations. Included in these proceedings are articles on radioactive tracing experiments conducted in several countries.

At the USDA Sedimentation Laboratory, research is concerned with the origin and routing of sediment in agricultural watersheds. Knowledge of the processes involved in the detachment, entrainment, transport, and deposition of the soil particles is necessary for planning the control of erosional processes. Techniques that will improve the measurement and determination of sediment transport under both overland and streamflow conditions will result in the design of more effective erosion and flood prevention structures.

The measurement of total sediment discharge in terms of hydraulic parameters and sediment properties is one of the most important objectives of research in sedimentation. Tracer techniques, including both radioactive and luminescent materials, offer a unique approach to the study of discrete particle movement, and hence to the possible evaluation of many sediment transport problems.

It is not the intention to give a thorough review of the present knowledge on this subject. It is, however, pertinent to this discussion to review some of the previous studies in terms of their objectives, materials and methods, and interpretations. Such a review will aid in the application of this method to sedimentation problems encountered in agricultural areas.

Review of Literature

Preparation of Labeled Sediment

It is axiomatic that the tracer possess the same hydraulic characteristics, i.e., transport properties, as the natural sediment. Theoretically, the tracer and the natural sediment must be equivalent in terms of these chemical and physical properties: (1) Specific gravity, (2) size distribution, (3) shape, (4) surface properties, (5) hardness, and (6) chemical stability. Any drastic differences between the tracer and sediment in question will obviously lead to differential sorting or abrasion of the tracer and, hence, to erroneous results. In practice it may be necessary to deviate somewhat from the equivalence of tracer and natural sediment. These deviations will depend largely upon the individual laboratory and its access to certain equipment and facilities necessary for the preparation and measurement of the tracer material. Such deviations must, nevertheless, be borne in mind in the final interpretation of the experimental results.

A number of radioisotopes have been used in studies of sand and silt movement. Thus far, results have been largely qualitative in nature. This is not surprising, however, in view of the inherent limitations of the method. The results, nevertheless, have been highly informative. Tracers have been prepared from both natural and artificial sediments.

Irradiation of Natural Sediments

Slow neutron irradiation has been utilized in the activation of both natural and artificial sediments. The irradiation of quartz and natural sediments has been employed by Goldberg and Inman (*7*), Inman and Chamberlain (*8*), Crickmore (*2*), and Crickmore and Lean (*3*).

Goldberg and Inman found that phosphorus-32 (beta emitter, 1.70 million electron volts (Mev), half life = 14.3 days) was the principal radioisotope resulting from the slow neutron irradiation of natural quartz sand. The beta particles emitted from phosphorus-32 are readily adsorbed (penetrating ability = 0.2 inch in water), making in situ measurements impractical. Inman and Chamberlain utilized these emissions to study the movement of beach sand at the Scripp Institute of Oceanography. Surface and core samples were taken from the experimental area and the individual samples analyzed by autoradiographic technique. In this manner, it was possible to record the presence of individual irradiated grains on photo-sensitive film.

Recently, sediment transport investigations utilizing irradiated sand were conducted by Crickmore and associates at the Hydraulics Research Station in Wallingford, England. Irradiation of untreated "Arnold 26A" sand (mean size, 170 microns) yielded both gamma and beta radiation. Autoradiography tests on this sample demonstrated the nonuniformity in activity of individual grains. Grains showing high activity were identified as the minor constituents of the sand, e.g., feldspars, micas, and magnetite. The quartz grains gave only weak activities. The sediment transport properties of the minor constituents could not be expected to be the same as quartz, the major constituent of the sand. The untreated irradiated sand was, therefore, demonstrated to be unsuitable as a tracer.

Activity measurements on various density fractions of the sand indicated the presence of a number of radioisotopes with a wide range of half-lives. The middle density fraction (specific gravity 2.60 to 2.70) was analyzed 3 months after irradiation. Scandium-46 (0.89 and 1.12 Mev) was the most active constituent, plus con-

tributions from iron-59 (1.10 and 1.29 Mev) and possibly iridium-192 (0.31, 0.47, and 0.61 Mev).

Activity distributions with respect to particle size were also performed on the different density fractions of the irradiated "Arnold 26A" sand. Analyses indicated that the specific activity was inversely proportional to particle size. This was true even for the middle density fraction (specific gravity 2.64 to 2.66), which was composed almost entirely of quartz. Decay curves for the different size fractions were similar, indicating that the radioactive constituents were the same for all fractions. Furthermore, the variation of surface activity with size conformed very closely with the variation of surface area with size. This suggested that the activity was present on the surface of the particles.

The surface label of the sand presented two problems in regard to its use as a tracer: (1) Possible losses of activity under the sediment transport conditions, i.e., from solution and abrasion effects; and (2) lack of proportionality between activity and total mass of tracer for any given point during transport.

Laboratory measurements of activity losses under severe abrasion and solution conditions showed losses of 15 to 20 percent of the activity. These activity losses occurred in the first 2 hours, with no significant subsequent change.

Crickmore desired to compare the sediment transport rates of tracer sand with the original sand in a laboratory channel. Emphasis was placed on the transport rate for the total mass of sand without concern for the movement of any definite size fraction. The variation of activity with particle size complicated this type of experimentation. In other words, for in situ measurements of the sand transported, it was desirable that the total activity at any point be proportional to the total tracer present, independent of particle size. Two possible approaches were available to correct this problem: (1) Different size fractions of the tracer could be irradiated for different lengths of time to produce the same specific activity; or (2) the amount of each size fraction could be adjusted to compensate for the lack of uniformity in specific activity. The second approach was chosen, and the tracer size distribution was weighted toward the coarser fractions to offset the bias of the activity distribution. A tracer modified in this manner can be treated as though it is labeled for bulk or volume.

The transport study was made in a large laboratory flume operating at equilibrium with a steady discharge of 10 c.f.s. At equilibrium, the sand transport was 100 pounds per hour $\pm$ 5 percent as determined by weighing the collected sediment. The middle density fraction (specific gravity 2.64 to 2.66), after irradiation and grain size modification to give a volume-labeled tracer, was added to the bed at the upstream end of the flume. Bed sand was removed from a cross section of the flume to a depth corresponding to the deepest previously observed dune trough. Active sand was mixed with this material in the ratio of 1:300; the mixture was returned to the trench and carefully molded to resemble the natural dune pattern.

Movement of the tracer was determined by a scintillation detector that could be traversed along the flume. Activity at any given point was compared to the total quantity of tracer transported for different time intervals. All measurements were referred to a standard sample to allow correction for decay of the radioisotope(s). The average velocity of transport was determined as 2.00 feet per hour from the rate of movements of the centers of gravity of the distributions. Activity measurements of the injection strip after 100 hours of flume operation gave an average thickness of movement of 0.116 foot, corresponding very closely to the depth of the deepest trough below mean bed level. A volume transport of 1.18 cubic feet per hour for the sand was computed from knowledge of the average transport rate, mean depth of movement, and width of the flume. The bulk density of sand compacted in the rippled layer was measured as 90 pounds per cubic foot. The transport rate, calculated to be 106 pounds per hour by the tracer technique, was in good agreement with the weighing method.

Further studies by Crickmore and Lean have dealt with the development of a method of measuring sand transport in rivers by means of radioactive tracers. These investigations were also made in a large laboratory flume under equilibrium flow. The transport rate was obtained from the mass distribution of tracer particles initially placed at a depth sufficient to cover all levels of movement of the bed dunes. An attempt was made to explain the shape of the activity distribution curves in terms of a model of the mechanism of transport. The model assumed that every particle has an equal chance of moving a certain hop length. They concluded that a more complicated model which considers both the statistical variation of the hop length and the time that particles spend at different levels in the ripples appears to be necessary.

Sorption of Radioactivity on the Surface of Natural Sediments

Labeling of natural sediments by sorption of radioactivity on the surface has been conducted by a number of investigators. Gilbert and

others (*5*) used sand grains labeled by adsorption (sic) of Ag-110 to investigate sand movement in the Figueira da Foy harbor (Portugal). Silver-110 added to the sand as a solution of silver-110 nitrate was subsequently reduced to metallic silver by sunlight. Under these conditions, 50 to 70 percent of the activity was fixed permanently on the grains. Eight hundred millicuries of this isotope, contained in 4,000 kilograms of sand, was placed on the harbor floor. Rapid dispersion of the tracer rendered a direct survey of the injected area to be of little value. Consequently, samples were collected, the silver was extracted from the sand and analyzed in the laboratory. The long half-life (270 days), together with the added sensitivity gained by sampling and chemical extraction of the isotope, permitted measurements to be made for several months.

No information regarding the mineral composition or the particle size distribution of the sand was included in the assay. The various density and size fractions will show differential sorption of activity. In addition, the sorptive properties of minerals vary considerably. Sand grains behave as individual entities; thus the movement of preferred density or size fractions might well result from such labeling. The conclusions were qualitative in nature. No attempt was made to estimate amounts of transported sediment.

The sorption of gold-198 on natural sediments in the San Francisco Bay was utilized by Krone and others (*15, 16, 17,* and *18*) to investigate silt and clay transport in the area. The program of research was concerned with the method of labeling of Bay sediments, detection and measurement of Bay sediments, and the application of these measurements to estuarial sediment transport problems. Three annual progress reports (*15, 16,* and *17*), and a manual, "An Underwater Scintillation Detector for Gamma Emitters" (*14*), were prepared in conjunction with these investigations. In addition, a final report (*18*) was prepared. The reader is referred to these reports for a detailed and comprehensive discussion of the use of radioactive tracers. Obviously the discussion is primarily concerned with the use of tracers in estuaries; however, it includes a wealth of basic information that is applicable to any sediment tracer program.

The importance of adequate numbers of tracer particles with suitable specific activity was stressed by Krone. Not only is it necessary to supply sufficient activity to overcome background radiation but an adequate number of tracer particles must be seen by the detector. Knowledge of the detector sensitivity becomes increasingly important when quantitative data are desired.

In these reports considerable space was devoted to the selection, calibration, and manipulation of an underwater detector to give quantitative data regarding sediment transport. The in situ measurement of radioactive sediment was performed by continuous monitoring of the estuary floor for tracer abundance. A scintillation detection system was selected, because it would provide the most sensitive and useful response to the tracer. The detector positioned on a sled with lightly loaded runners was found to withstand the vibration, shock, and pressures imposed by the environmental conditions. It was pointed out that the sled was found to be stable and to position the detector consistently with respect to the sediment surface. Vertical adjustment of the detector was possible, permitting detector response to be established for different geometrical locations of the detector to the Bay floor. The sled contained suitable towing harness that was connected to the tow boat, permitting continuous monitoring of the tracer for paths along the estuary floor.

It was determined that the newly deposited tracer was usually confined to a thin layer at the surface of the Bay floor. The detector sled would penetrate any unconsolidated surface material. Once the sled encountered a consolidated surface, it could be supported under the prescribed towing conditions. After considerable inspection and evaluation of the Bay floor, it was proved desirable without exception to position the detector end even with the sled runners for activity measurements in the Bay area.

Detector response is obviously sensitive to the distribution of the tracer material around it. Radiation is attenuated by both water and sediment. These factors make the positioning of the detector, and particularly the consistency of its position with respect to the tracer, the most important factor in any tracer investigation. Maintaining a consistent position of the detector to the Bay sediment placed demanding requirements on consistent towing conditions. If the detector was lifted from the bottom by excessive towing speed, a sharp decline in activity was immediately obvious.

A detector sensitivity of 6,000 counts per minutes per microcurie per square foot was determined in the laboratory for the operating conditions to be used in the Bay measurements. This value was obtained for gold-198 with the discrimination circuitry accepting energies between 0.31 and 0.45 mev, and with the detector positioned with its lower end even with the sled runners.

Towing the underwater detector along selected paths in the labeled area of the Bay permitted activity measurements of the Bay floor. Activity profiles were obtained for the various traverses after correction for the background (measured before addition of the tracer) and decay of the radioisotope. Contours of activity were subsequently drawn. The area between adjacent contours was measured with a planimeter; this area multiplied by the average activity levels between adjacent contours provided activity (in counts per minute) per square foot. Summation of such data divided by the previously determined detector sensitivity (in counts per minute per microcurie per square foot) gave the total number of microcuries deposited. In this manner it was possible to make quantitative estimates of the tracer movement.

The silt transport investigations provided information on the cause of shoaling and the direction of transport. Knowledge of the movements of sediment permitted location of the sources of shoaling material, assisted in evaluating dredging practices and provided guidance in determining remedial measures for the San Francisco Bay Area.

Surface labeling has been used with other radioisotopes in a variety of tracer experiments. The sorption of chromium-51 on sand, $t\frac{1}{2} = 28$ days (0.32 Mev), was used by Davidson (*4*) to study the sand movements in Sweden. It was concluded that this isotope was not satisfactory because of its low gamma emissions and poor penetrating ability.

BaLa-140 (barium-140, $t\frac{1}{2} = 12.8$ days–lanthanum-140, $t\frac{1}{2} = 40$ hours) and lanthanum-140 have been used by Kidson and coworkers (*11, 13*) to label natural flint material for drift experiments at Oxfordness, England. The activity was sorbed on the surface of the pebbles. Lanthanum-140 was particularly suitable for short-lived experiments over part of a tidal cycle. The article discussed the ease of handling these materials and the fact that individual contacts of such pebbles can be recorded. It was also emphasized that material resistant to surface abrasion is required.

Scandium-46, $t\frac{1}{2} = 85$ days (0.89 and 1.12 Mev), has been bonded to the surface of quartz grains by McHenry and McDowell (*19*). A selected size fraction of sand was separated by sieving; the heavy mineral fraction was removed by bromoform and the iron oxide coating subsequently removed by Jeffries' treatment (*10*). Sand prepared in this manner was found to readily sorb scandium-46 from a solution of scandium-46 chloride. Slow heating with infrared light followed by heating to 800° to 1000° C. was sufficient to bond the isotope to the quartz. No measurable change in specific gravity of the quartz was noted for the "tagged" sand. Considerable variation in the amount of activity sorbed by different sand grains was observed when single grains were counted, with the use of a differential scintillation spectrometer. Furthermore, when subjected to leaching and abrasion tests, the removal of activity varied greatly from particle to particle.

Artificial Sediments

Several tracer studies have been performed by the use of artificial sediments. This process involves the incorporation of the desired radioisotope, usually a gamma emitter, in some carrier material, e.g., glass.

Glass containing the suitable isotope has been prepared, ground, and sieved to match the density and grain size distribution of the natural sediment. Activation of the glass by slow neutron irradiation (η, γ) produces a simulated sediment for tracer application.

Radioactive glasses have also been prepared by incorporating the radioisotope with the normal ingredients of glass, melting of these constituents, quenching, grinding, and sieving to obtain the desired size distribution. This technique offers considerable hazard to the personnel preparing the material.

The use of radioactive glass in tracer studies has been particularly informative. A discussion of the studies in part is deemed essential in terms of their application to our sediment problems.

Best known of the various experiments is the pioneer work conducted in 1954–55 on tracing the silt movement in the Thames estuary (*21*). This investigation was conducted to determine if landward transport of silt occurs in the estuary. Some confusion exists regarding the size of the natural sediment in the estuary. Apparently the bulk of the material consisted of aggregates of clay minerals together with appreciable amounts of organic matter. The sizes of the stable aggregates (assuming their stability in the natural environment) are of significance in the preparation and use of simulated sediment particles.

The tracer used in these investigations consisted of soda glass ground and sieved to the grain size distribution obtained by sediment tests on the natural sediment. The median size was 45 microns. The labeled glass was mixed with mud before introduction to the channel bottom. Scandium-46 was the selected isotope because of its energetic gamma emissions, ease of activation in a slow neutron flux, suitable half life (85 days), and the solubility of its oxide in glass.

The large-scale experiment on sediment movement was conducted in 1955. In this test 845 grams of scandium glass ground to a me-

dian size of 45 microns (10^{10} particles) were activated to a level of 29 curies. The material was mixed with mud prior to deposition in the estuary.

The tracer was detected by using three large Geiger tubes designed for underwater surveys. Tracer material moved over distances of 15 miles. Mud in suspension near the bed is deposited and consolidated in the shoal areas. Perhaps the most rewarding information from these studies was noted in the tidal basin at Tilbury Docks (12 miles above the injection point). Siltation in this basin necessitates considerable dredging. Activity levels gradually increased in this area to three times background readings during the first 2 weeks after placement of the tracer. These observations indicated that silt had moved toward the head of the estuary as a consequence of the net landward movement of water close to the bed. Density differences in the river and sea water accounted for the net landward movement of water close to the estuary floor. They concluded that dredged material should not be put back into another part of the estuary but that it should be pumped ashore.

Putnam and Smith (*20*) used scandium-46 glass (prepared in the manner discussed above) to study the movement of sand on the ocean floor. These tests were conducted 2,000 feet offshore in shallow water. Four months after placement of the tracer, after a period of heavy wave action, the tracer was found distributed over an area approximately 3.2×10^5 square feet. Two curies of tracer contained in 2×10^7 particles were used in this study. No quantitative information was given on the rate of transport.

The transport of large "pebbles" (approximately 2 inches in diameter) was investigated by Steers and Smith (*22*). Sandstone pebbles and additional concrete pebbles were obtained to simulate 1,200 naturally occurring pebbles. Holes were drilled in the pebbles and the tracer was added to the interior. The holes were then filled with cement. Barium-140–lanthanum-140 (Ba La-140) was utilized in this study. Monitoring of the pebbles on the sea floor was accomplished by a Geiger-Mueller (GM) detector. Pebbles were located singly by covering the area carefully. This study not only shows that large pebbles are moved on the sea floor but clearly demonstates that individual labeling of gravel or pebbles is feasible. This technique might be applied successfully to the transport of gravel and pebbles larger than 6 or 8 mm. in diameter.

Several aspects of sediment transport by tracer techniques are reported in an extensive article by Arlman and others (*1*). The authors stressed the importance of sufficient number of tracer particles and particle activity to give statistically meaningful results under the conditions of the test, i.e., for the detector sensitivity and the area over which the tracer is dispersed, both horizontally and vertically.

The primary objective was to investigate the movement of sands in the North Sea. Preliminary investigations of sediment transport were conducted in a laboratory flume. Glass beads containing sodium-24, $t1/2 = 15.06$ hours (1.37 and 2.75 Mev), were used in this study. Sand was removed from the bed of the flume, mixed with the tracer, and returned to give the same bed configuration. It was desired to operate the flume under equilibrium flow conditions with water discharge, sediment discharge, and sediment input (including tracer) carefully monitored. After the test was started, however, it appeared that the input sediment was not carried off sufficiently from the deposit area. This was apparently the result of a dense packing of the original mixing of the tracer and sand. The equilibrium of the bottom was disturbed, and it was decided to continue the test without replenishment of the deposit area and with decreasing transport of the bed material.

Activity measurements were made from a fixed position on a measuring bridge by the use of a scintillation detector. This detector was shielded with lead to restrict the "field of view" to 300 cm.^{2} on the sand bed. In its fixed position the detector received only photons from a 30-cm. strip of the bed as longitudinal traverses were made. The distribution of activity over other parts of the bed were made on unstirred samples from the bed. Core samples were also removed from the bed and analyzed in a GM counter to obtain information on the vertical distribution of the tracer. This GM counter was found to be inadequate for tracer measurements. Sample preparation was involved. Geometry and back scatter problems made this detector less desirable.

The transport velocities of the glass tracer beads (specific gravity 2.95) and original sand were estimated to be 0.09 and 0.16 meter per second, respectively. The glass beads used in this test did not match the grain size distribution of the sand. The authors concluded, however, that sediment transport could be measured qualitatively and quantitatively by means of radioactive tracers.

According to Kidson and Carr (*12*), measurement of the sand movement in the North Sea was conducted by Arlman and coworkers using scandium-46. It was found impossible to bond the isotope directly to the sand surface; instead, the isotope was bonded to a zeolitic mineral having a specific gravity of 2.72 and 2.76. The zeolite is soft and easily abraided;

grain size would have been reduced from that typical of the natural sand. This was apparently remedied, however, by firing the zeolite to harden it.

Littoral drift measurement of sand with a glass tracer labeled with zinc-65, $t\frac{1}{2} = 245$ days (1.12 Mev), was made in Japan by Inose and others (*9*). Zinc-65 was chosen because of its (1) strong gamma emissions, (2) long half-life, (3) low cost, and (4) ease of incorporation in glass. The isotope had to be shipped from the United States and be incorporated in glass before use; thus, the 245-day half-life was not undesirably long. No irradiation facilities were available in Japan, making it necessary to fix the radioisotope in the glass directly.

The radioactive melt, carefully prepared to give the proper specific gravity 2.63, was quenched in water, crushed, and sieved to obtain the desired size distribution. The addition of the radioisotope before grinding is not desirable unless special facilities are available for protection of the personnel.

Thirty millicuries in 1.5 liters of ground glass (approximately 10^9 to 10^{10} particles) were used in the investigation. Measurements were made with a scintillometer using a 1 by 1 by 0.5 inch anthracene crystal. The tracer was placed in 2.8 meters of water. Four radial surveys from the injection point were made during the month after deposition of the tracer. Activity measurements were plotted as maps of equal activity rate contours. The direction of net transport was readily obtained from contours plotted for the different surveys.

Measurement of the Tracer

Interaction of Radiation With Matter

Sediment transport investigations using labeled sediment requires appropriate instrumentation capable of detecting and measuring the intensity of the radiation. Isotopes emitting only alpha or beta particles would not be suitable for in situ measurements of sediment transport. These particles are readily absorbed in matter. For illustration, examples of the penetrating abilities of hard and soft beta radiations are given in table 1.

TABLE 1.—*Penetrating abilities of hard and soft beta radiations*

Isotope	Maximum energy	Penetrating ability		Radiation range
		In air	In water	
	Mev	*Inches*	*Inches*	*M_g./cm.²*
Phosphorus-32	1.70	280	0.2	800
Carbon-14	.155	10	.01	28
Yittrium-90	2.18	350	.45	1,100

These data indicate the impracticability of using even the hard betas for in situ measurements of sediment transport. Phosphorus-32, $t\frac{1}{2} = 14$ days (1.7 Mev), or yittrium-90, $t\frac{1}{2} = 64.2$ hours (2.18 Mev), could be used in the manner employed by Inman and Chamberlain. The use of gamma-emitting isotopes, which has been stressed by previous workers, is necessary where measurements are to be conducted with a minimum of sampling.

When gamma radiation interacts with matter, there are essentially three competing possibilities; namely, the photoelectric effect, Compton interaction, and pair production. These processes are dependent upon the energy of the radiation and the nature of the absorber. It will suffice to state that in the photoelectric process essentially all the energy of the gamma photon is transferred to an electron that is ejected. This is a true absorption effect. Likewise, in pair production the gamma ray energy goes into the formation of a pair of electrons and imparts kinetic energy to them. In the Compton interaction, however, only a part of the energy of the incident photon is transferred to the electron, since a scattered photon of lower energy is also produced. This lower energy photon may suffer several Compton collisions, i.e., undergo multiple scattering, depending on the thickness of the absorber. If it does not escape, the scattered photon will eventually be absorbed as a result of the photoelectric effect.

In passing through matter, gamma ray photons are absorbed so that the intensity falls off in an exponential manner. If a collimated beam of monoenergetic gamma rays of intensity I_0 passes through a thickness $\times$ absorber, the intensity I_x of the emergent beam is given by the familiar equation.

$$I_x = I_0 e^{-\mu x} \qquad (1)$$

where μ is the linear absorption coefficient of the absorber for the given radiation. It should be noted that the absorption coefficient is a measure of the probability of occurrence of an interaction process. The total absorption coefficient μ_0 is the arithmetic sum of the various components of interaction discussed above. Laboratory calculations frequently are made on the thickness of the material that reduces the radiation intensity by one-half. This "half thickness" or "half value thickness" ($X_{\frac{1}{2}}$) is related to the absorption coefficient by

$$X_{\frac{1}{2}} = \frac{0.693}{\mu_0} \qquad (2)$$

These values are valid only for absorption obeying a simple exponential function, i.e., for a narrow collimated (parallel) beam of monoenergetic radiation. For this so-called "good" geometry, it is postulated that scattered radiation is completely removed from the radiation

beam. In other words, the scattered radiation does not reach the detector.

Absorption and geometric attenuation of radiation becomes important in tracer applications where the activity measurements are usually performed under "broad beam" or "poor" geometry conditions. In this situation, the scattered photons are not removed and the intensity at the detector is greater than that estimated by the exponential equation. The difference between the results of measurements in "narrow" and "broad beam" cases is described by means of a buildup factor. This factor refers to some measurable property of the radiation flux (e.g., intensity) and is defined as the ratio of this quantity when the effects of all photons are included to that obtained when only the uncollided photons are considered. Although the interaction of gamma radiation with matter is well defined, the calculation of buildup factors depending on the multiple scattered radiation is still a difficult problem. The correction of measured values for buildup can, nevertheless, be made in many cases.

Detector System

Both Geiger-Mueller (GM) and scintillation (organic and inorganic) detectors have been utilized for activity measurements of the labeled sediment. A complete treatise on the various detection systems available is beyond the scope of this paper. However, a brief discussion of GM and scintillation detectors, relevant to their application to sediment tracing, is deemed necessary.

Geiger-Mueller detectors are ion collection devices (consisting usually of an enclosed metal cylinder with a fine center or concentric wire electrode) containing an ionizing and quenching gas. Ionization is produced when radiation passes through the gas-filled chamber. A high potential is placed across the electrodes to allow collection, acceleration, and subsequent production of secondary ions on collision with the gas atoms. An avalanche of ions results that produces a current through the electrode system. The voltage across the electrodes is sufficient to produce gas amplification in the detector of 10^7. Under these conditions the discharge covers the entire length of the central electrode and all the output pulses are of the same size or height. The height of the pulses produced are consequently not proportional to the energy of the incident radiation. These pulses are amplified and counted or integrated with time, to measure the intensity of the incident radiation.

The efficiency of the GM detectors for gamma photons is 0.5 to 2 percent. High energy photons often penetrate the ion collection chamber without producing ionization. Furthermore, these detectors are characterized by a long resolving time, i.e., the minimum time interval required between registered events. Typical resolving times for GM detectors are in the range of 200 to 600 microseconds. Such systems have considerable loss of counts when receiving events greater than 10,000 counts per minute. Corrections can be made, however, to the registered events once the resolving time is determined for the counting system.

GM detectors are simple, rugged, and can be produced in large sizes. They require only a stable power supply and rate meter for field use. The disadvantages of this detector system are, however, the inability to distinguish or discriminate between photon energies, low efficiency for gamma radiation, and limited useful radiation intensity range.

Scintillation detectors utilize a principle different from the ionization-collection technique employed in the GM detector. Gamma radiation transfers energy to the electrons of matter with which it interacts. This energy transfer may be total or partial, depending on the type of interaction. The excited electrons in turn lose energy either by ionization or excitement of the atoms through which they are passing. This excitation of the atoms can be used for detection of the incident gamma radiation. Substances, i.e., phosphors, are available that can absorb such excitation energy and reemit (fluoresce) the energy in the form of light photons. A sodium iodide (thallium activated) crystal is the most commonly used phosphor for detecting gamma radiation. The important feature of this type of detection is that the number of light photons produced is proportional to the energy of the electron (i.e., to the absorbed energy). Light photons produced in the phosphor are converted to electrical energy by a multiplier phototube. These pulses undergo considerable amplification and are finally recorded by suitable scaling equipment.

Incorporation of a pulse height analyzer in this system permits rejection of unwanted emission energies. Ideally, two discriminators — contained in the pulse height analyzer — can be employed to accept only those pulses corresponding to the characteristic gamma photon energies of the particular radioisotope. This system permits (1) the rejection of already existing activity, i.e., background, having emission energies different from the tracer, (2) rejection of scattered radiation from attenuation of the photons on passing through matter, (3) use of different tracers simultaneously, i.e., radioisotopes having well-defined energy spectrums whose primary photon energies can be separated, (4) the accurate measurement of high levels of activity without loss of counts

from "dead" time of the system, and (5), within limits, the calibration of the detector with regard to spatial distribution of the tracer.

A complete scintillation counting system is made up of the following components:

(1) Detector (scintillation probe) — sodium iodide (thallium activated) crystal, multiplier phototube, and preamplifier;
(2) Pulse amplifier;
(3) Pulse height analyzer — upper and lower discriminators;
(4) High voltage supply;
(5) Scaler and timer; or;
(6) Ratemeter;
(7) Strip chart recorder.

These systems require linear pulse amplification and very stable voltage supplies. They are obviously lacking in ruggedness and require careful handling during field surveys. They do offer the ultimate in instrumentation for gamma ray detection in any tracer program.

The sole purpose of instrumentation is to determine the amount of unknown radioactivity for a particular system. To make full use of the scintillation system, it is possible to calibrate the instrument to determine the counting rate under the photopeak energy of a given radioisotope. This can be illustrated by referring to the gamma ray spectrum of scandium-46 in figure 1. The two photopeaks, 0.88 and 1.12 Mev, can be used to advantage in activity surveys. With the appropriate analyzer settings, e.g., 1.02–1.22 (2-volt gate) Mev for scandium-46, the counting rate would be essentially independent of scattered radiation in the source or in the vicinity of the crystal.

Measurements of the activity can then be conducted by one or a combination of the following procedures: (1) The absolute measurement of gamma ray activity by determining the probability of a maximum interaction in the given size of crystal and calculating the efficiency of detection for that energy, (2) a relative measurement may be made by counting in the photopeak region and calibrating the instrument with a sample of known activity intensity, and/or (3) under certain experimental conditions a relative measurement may be made by counting in the photopeak for the unknown and reference samples. In the latter case the actual activity of the reference sample need not be known, the interest is placed on the change in the reference before and after (or during) the experimentation. In any situation, constant attention must be given to maintaining the same source geometry. Ideally, it is desirable to apply all these methods of counting standardization so that one may serve as a check on the other. In other words, it is advisable to use a standard source of desired radioisotope and determine the resolution, sensitivity, and efficiency for the counting system under differing geometrical arrangements of detector to source.

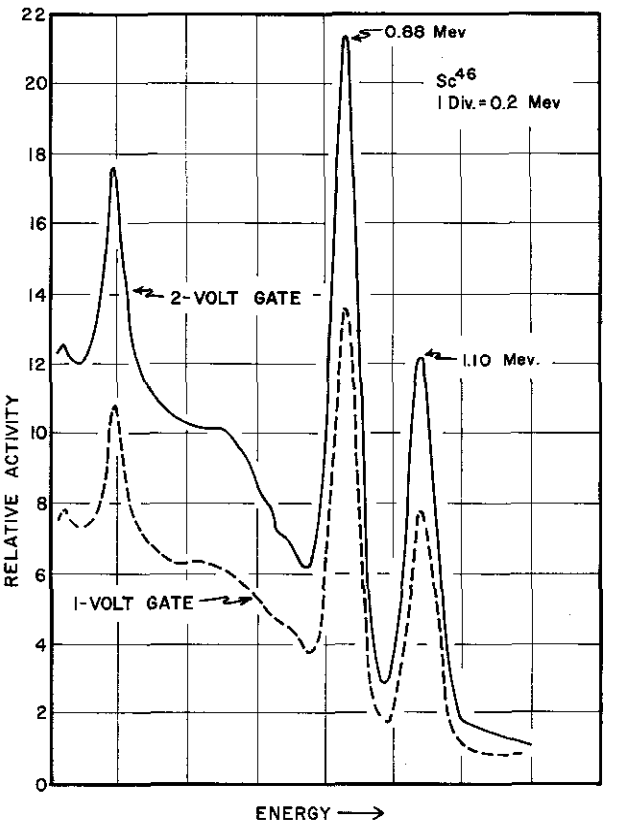


FIGURE 1. — The gamma ray energy spectrum of scandium-46. (Differential scintillation spectrometer: 1- and 2-volt gates.)

Sediment Transport in a Laboratory Flume

Radioactive tracers are being used by the Sedimentation Laboratory to investigate sediment transport by both overland and channel flow conditions. Controlled investigations utilizing tracers will be conducted in a 200-foot outdoor flume. These studies will provide needed information for the design, analysis, and interpretation of current and future field investigations.

Before initiating studies in the 200-foot flume, it was decided to conduct a preliminary investigation of sediment transport in one of the small flumes located in the Hydraulics Laboratory. A differential scintillation spectrometer has recently been modified for flume and field investigations of tracer movement. A preliminary study of sediment transport in a small flume under low flow conditions would (1) provide a check on the operation of the detecting and

counting system, i.e, the spectrometer system; (2) provide information on the procedures to use in determining tracer distributions, both vertically and horizontally; (3) acquaint the personnel with the problems associated with the geometry of the detector and source; and (4) illustrate the overall difficulties associated with a quantitative analysis of tracer distribution.

Methods and Materials

The flume selected for this study consisted of a rectangular channel 50 feet long and 14 inches wide equipped with a variable speed axial flow pump at the downstream end and a return pipe for recirculating water and sediment to the upper end of the channel. The flume was temporarily modified to permit operation as a "put and catch" channel, i.e., water could be added at the upper end and water and sediment discharges could be measured at the downstream end.

An instrument rack mounted on rails along the upper edge of the flume provided a mount for point gage and tracer detection equipment. The carrier with point gage provided a reference for positioning the detector. Longitudinal traverses of the flume could be made by moving the instrument rack along the rails. Thus, the detector, i.e., a scintillation probe, could be positioned vertically or horizontally at any point above the channel bed.

The scintillation probe (3 inches in diameter and 14 inches in length) contained a 2- by 2-inch sodium iodide (thallium activated) crystal, multiplier phototube, and preamplifier. The probe was connected through 100 feet of cable to the differential scintillation spectrometer. The cable was necessary to (1) supply voltage to the multiplier phototube, (2) supply voltage to the preamplifier, and (3) receive the incoming pulses from the preamplifier. The discriminators (contained in the pulse height analyzer) were calibrated to accept only pulses corresponding to photons having an energy range of 1.02–1.22 Mev. This energy corresponds to the second photopeak in the scandium-46 energy spectrum. The pulses, after amplification and discrimination, were supplied to a ratemeter for integration with time or to a scaler-timer unit. Provision was also made to supply the ratemeter output to a strip chart recorder for permanent recording of the tracer signal.

The upper 10 feet of the channel were used to dissipate the energy of the incoming water and hence to provide a smooth flow condition to the channel bed. The flume was operated at zero slope. Sand having a mean diameter of 0.48 mm. was used as the bed material. The sand bed was smoothed under conditions of low flow to give a slope of 0.0025. Water depths ranged from 0.017 to 0.035 foot.

Ten grams (10^5 grains) of radioactive sand (0.350 to 0.297 mm. in diameter) were prepared by the following method:

1. Heavy minerals (specific gravity, 2.89) were removed by separation in bromoform.
2. Iron oxides were removed from the surface of the quartz grains by Jeffries' treatment, i.e., reduction with magnesium in oxalic acid.
3. Further separations in bromoform were made to obtain sand having a specific gravity of 2.60 to 2.65.
4. The sand was activated by the sorption technique. The sand was permitted to sorb scandium-46 chloride with subsequent heating to 840° C. to convert the chloride to the more stable oxide.
5. The activated sand was washed in water to remove any loosely bonded radioisotope.

Previous studies have indicated that considerable variation in grain activity is obtained with this system of labeling. Furthermore, it has been shown that the loss of activity by abrasion in water and sand varies from one grain to another. Estimates of activity lost under laboratory abrasion tests range from 2 to 5 percent. Obviously it is difficult to estimate the losses that may occur under field or flume conditions.

The labeled sand was mixed with 40 grams of nonradioactive sand and subsequently placed in a 2- by 14-inch pan for analysis. The detector, placed at a distance of 0.12 foot above the sand, was used to obtain measurements of the radiation. Attenuation measurements of the radiation were also made with varying depths of water above the sample.

The labeled sand was subsequently placed on the surface of the channel bed 15 feet from the upstream end of the flume. Placement was made in a narrow band extending the width of the flume. Activity measurements of the injection strip were in agreement with the measurements conducted in the analysis pan. Measurements made on the upstream part of the channel and 33 feet downstream from the injection strip indicated a background radiation of two counts per second for the specific spectrometer operation.

Water discharge measurements were periodically checked and found to vary from 0.0146 to 0.0150 cubic foot per second during the 5 days of operation. Sediment transport was extremely slow during the experiment. The sediment discharge for the first 30 hours of flume operation was 7 grams per minute. Trapping problems arose during the second day; measurements of the sediment discharge were terminated for the remainder of the experiment. During the experiment, considerable change in the bed configuration occurred. Small sand dunes,

¼-inch in height and 2 to 4 feet in length, were present during the second day of flow. This condition slowly changed to a meandering thalweg that remained throughout the experiment. The thalweg extended along one flume wall from station 0 to station 7. It crossed the channel at this point to the other flume wall and continued downstream.

It was originally planned that the probe would be surrounded by a lead shield to restrict the area of the crystal receiving incoming radiation. The area of the channel bed "viewed" by the detector could be well defined by extending the lead shield below the crystal surface. Two inches of lead are required to attenuate 99 percent of the gamma photons having an energy of 1.10 Mev. Shielding of this magnitude presents two immediate problems: (1) The physical manipulation of the probe, and (2) attenuation losses of the radiation in the additional 2 inches of air interposed between the crystal and the channel bed. The latter condition would make it necessary to increase the activity level in the bed. Restriction of the channel bed area "viewed" by the detector is advantageous; it, however, does necessitate the use of larger amounts of activity to obtain sufficient flux for the crystal. It was desired to keep the activity levels to a minimum; hence the lead shield was not used in this trial run.

A ratemeter, having a range in time constants from ½ to 80 seconds, was used in various parts of the study. This instrument contains both a panel-type meter and a recorder jack for reading the output. It has the advantage of indicating and recording the counting rate continuously. It cannot, however, attain the accuracy of the counter-timer (scaler-timer) combination for situations in which the measuring time is limited.

Results and Discussion

Horizontal and Vertical Distribution of Activity

Measurements were made with the probe in a fixed position above the bed and at 1-foot intervals downstream from the injection strip. The ratemeter is, of course, less desirable than the scaler-timer combination for these measurements. Sufficient time must elapse for equilibrium to be established in the tank circuit of the ratemeter before the errors involved are entirely statistical. Activity surveys of the channel bed were made after 5, 23, 30, 46, 70, and 125 hours of flow. The measured activities at 1-foot intervals were related to the total activity placed in the injection strip to obtain the fraction of the labeled sand at each cross section of the channel (figs. 2 and 3). These measurements were made with an unshielded probe at a constant distance from the channel bed. No corrections have been applied to the data for the

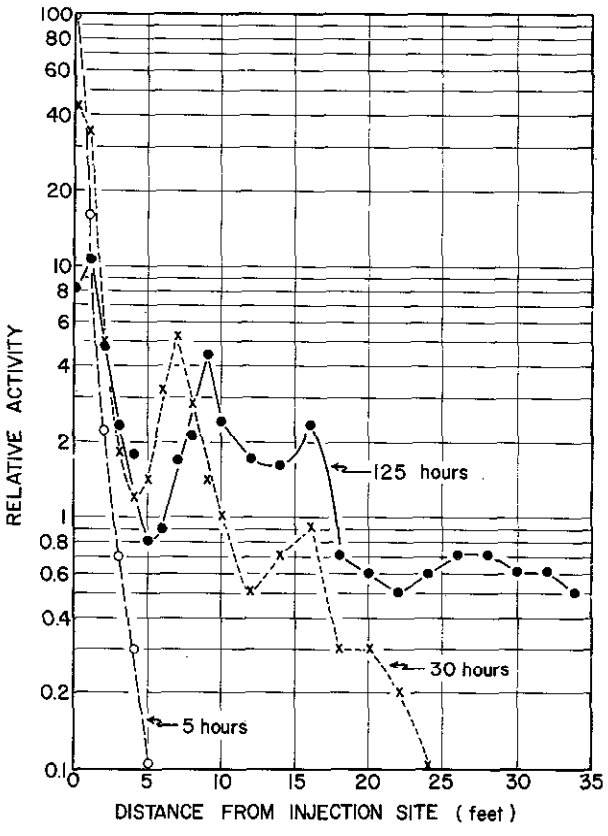


FIGURE 2. — Distribution of scandium-46 labeled sand in the channel bed after 5, 30, and 125 hours of flow.

statistical errors of the independent measurements. Such considerations will be necessary for quantitative analyses in which the summation of a series of independent measurements are compared to the originally deposited tracer.

Summations of the activity surveys for 5 and 23 hours show a positive error of roughly 17 percent. This is to be expected during the early stages of the experiment when the activity gradient is steep. A rough approximation can be applied to determine the contributions to the detector from adjacent areas. Such approximations indicate that an error of roughly 10 percent exists between the original measurement of the deposited tracer and the summed values of the individual measurements.

The data in figures 2 and 3 represent a semiquantitative analysis of the tracer distributions for various flow time intervals. If quantitative data were available it would be possible to estimate the transport velocity of the labeled sand from the weighted distributions of tracer for any given activity survey. The calculation of sediment-transport velocity and sediment discharge will be discussed further in a later part of this section.

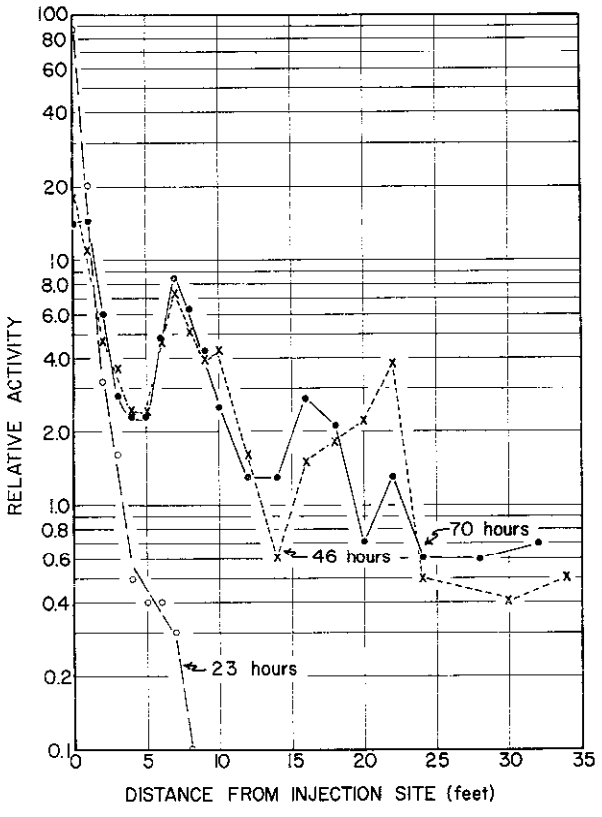


FIGURE 3. — Distribution of scandium-46 labeled sand in the channel bed after 23, 46, and 70 hours of flow.

Sediment transport velocities can be obtained by monitoring the injection strip for equilibrium flow conditions. In this experiment where aggradation and degradation are occurring, such measurements are questionable. Decreases in activity in the injection strip, as noted by the detector, may be the result of (1) the actual movement of tracer from the area, and/or (2) the attenuation of gamma photons from the tracer by overlying sediment. The detector is unable to distinguish between a few photons resulting from a large quantity of buried tracer and those arising from a small amount of tracer on the surface of the bed. The vertical distribution of tracer is then one of the more important factors in any tracer analysis. Qualitatively, there is evidence of tracer burial by nonradioactive sand in the 30- and 46-hour activity surveys. Observations of the changing bed configuration during the flow also lead to this conclusion.

Considerable variation of activity within each cross section was observed during the flow. These measurements were made by traversing definite cross sections of the channel. A more detailed survey of the variation of activity was made after termination of the flow. Variations within the injection strip and the first five stations downstream are illustrated graphically in figure 4. The values show qualitatively (1) the presence of high activity levels in the upstream end of the channel, and (2) variations of three- to fourfold for activities within any one cross section. The values represent the total radiation as received by the detector without any correction for attenuation of photons from buried tracer. The lower activities present in the 8.75- to 12.25-inch part of the channel bed reflect the removal of tracer in the thalweg. The higher activity levels at station 7 (fig. 2) illustrate the cross-over point of the thalweg. The cross-over point of the thalweg progressed downstream at the rate of 0.02 ft./hr. as determined from the 30- and 125-hour activity surveys.

The analysis of any tracer distribution is necessarily complicated by nonuniform burial of the tracer. The vertical distribution of tracer was qualitatively determined at various cross sectional intervals. This was accomplished by removing thin layers of sand from the bed with subsequent measurements of the activity levels

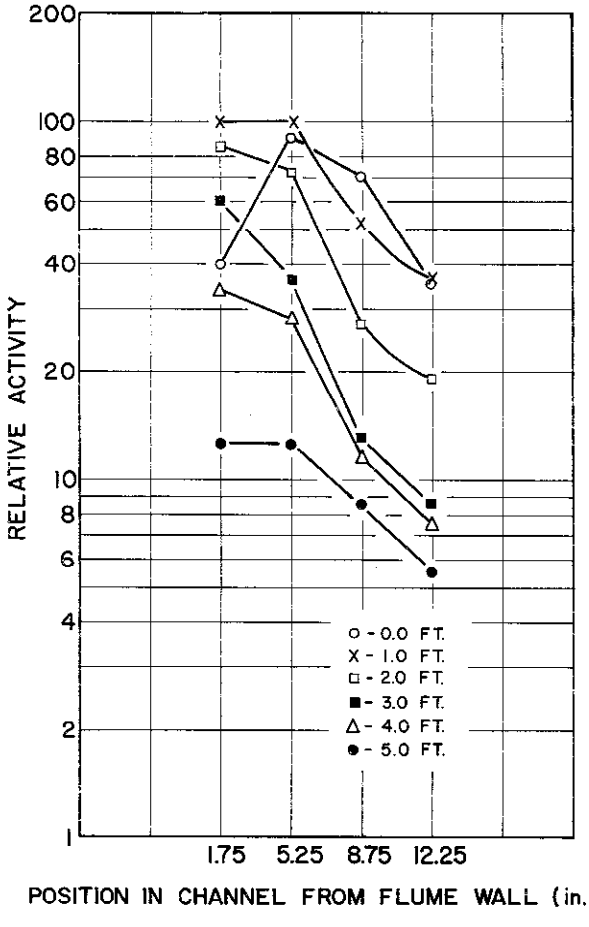


FIGURE 4. — Distribution of scandium-46 labeled sand within defined sections of the channel bed.

remaining in the bed. The depth of sediment removed each time was determined by point gage measurements of the channel bed. Surveys of stations 0, 1, and 2 are given in figures 5, 6, and 7, respectively. Only two positions within

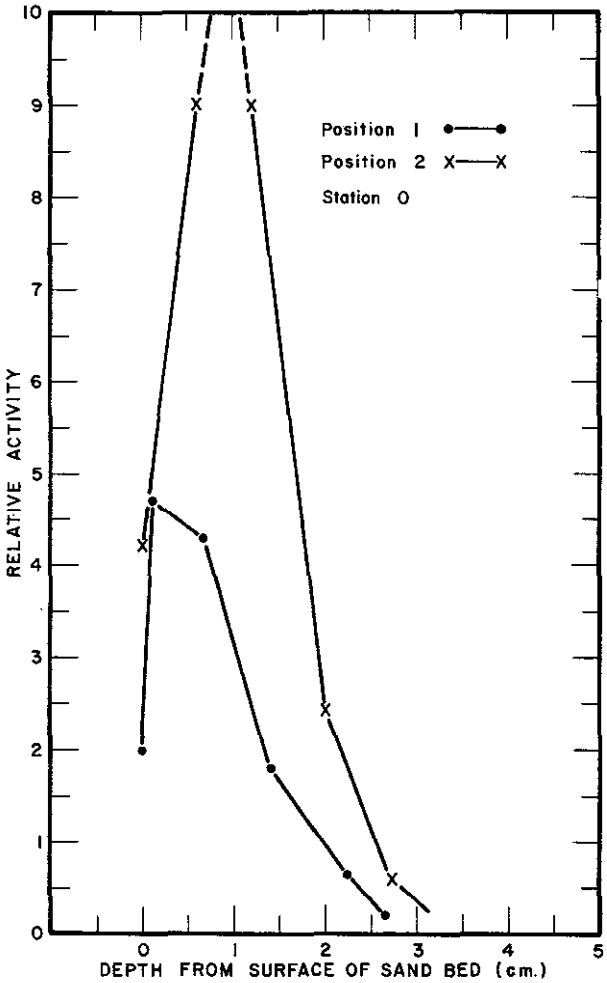


FIGURE 5. — Vertical distribution of scandium-46 labeled sand in the channel bed after 125 hours of flume operation (station 0).

each cross section are graphically illustrated. The activity levels at zero depth refer to those determined on the natural channel bed prior to the removal of sand. The initial increase in activity (as seen by the detector), when successive vertical layers of sand were removed, is indicative of aggradation in this part of the channel. The photons from the tracer were attenuated by the overlying nonradioactive sand. Tracer sand was covered by approximately 1 cm. of sand for this part of the bed.

The decrease in activity with successive removal of bed material is to be expected. These data clearly indicate that tracer sand was buried to a depth of 1.0 to 1.2 inches for the flow condition imposed on the bed. Vertical examination of stations 3, 4, 5, and 6 indicated no overlying nonradioactive sand, but values for buried tracer were similar to those mentioned above. Further examination of the bed at various cross sections indicated that labeled sand had been buried to a depth of 1.0 to 1.2 inches throughout the channel bed.

Measurement and Correction of Attenuation

Vertical distribution of tracer places a serious restriction on the investigation of sediment transport by tracer application. Knowledge of the attenuation of the gamma photons in passing through the sediment is essential for any quantitative estimate of sediment transport. In this experiment the attenuation of the 1.12 Mev photons by sand (bulk density 1.7) from scandium-46 was measured by a plane source (cross

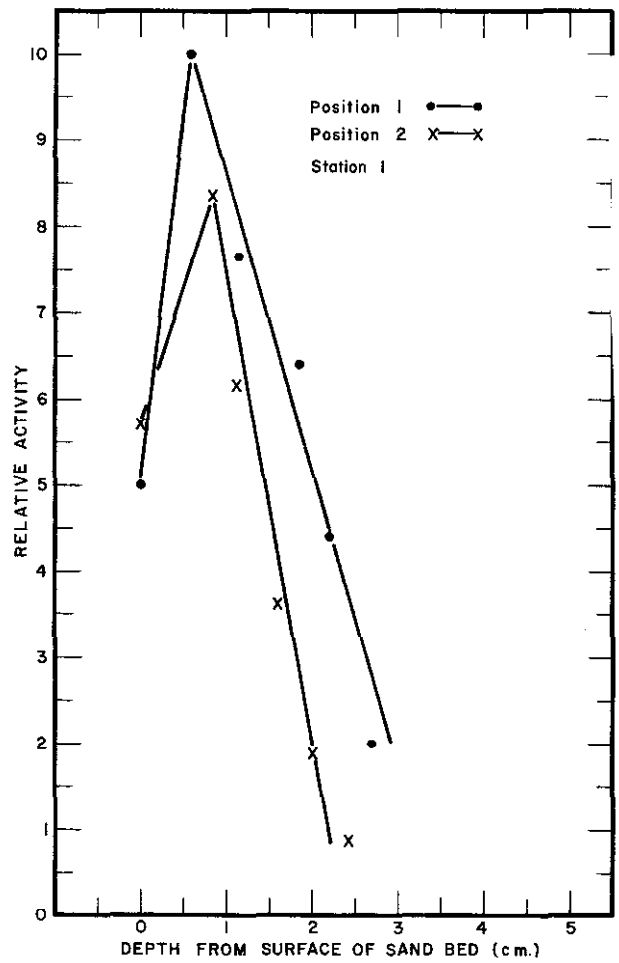


FIGURE 6. — Vertical distribution of scandium-46 labeled sand in the channel bed after 125 hours of flume operation (station 1).

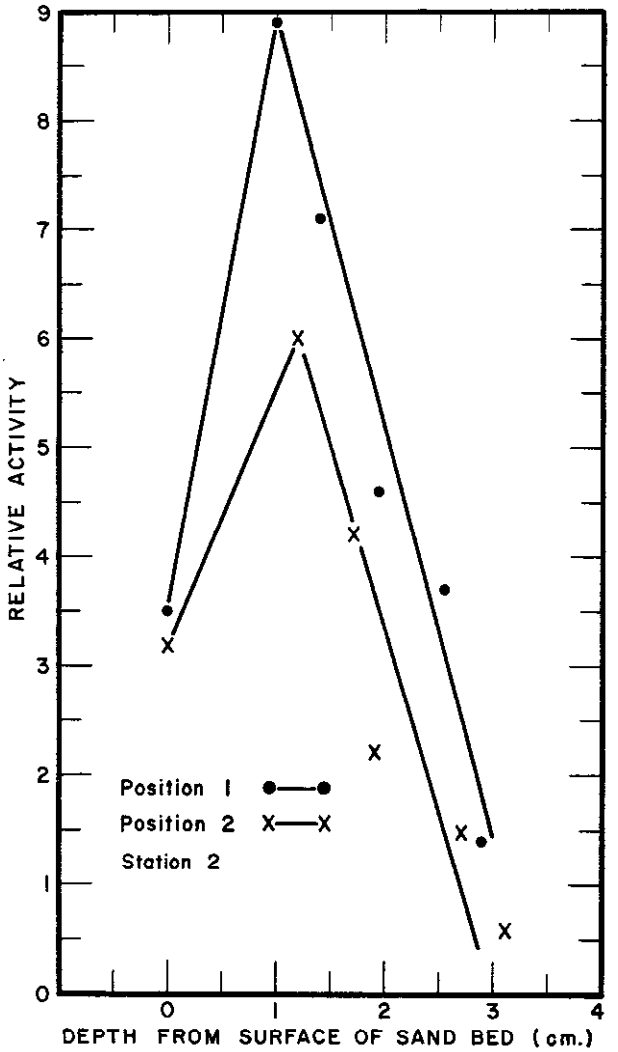


FIGURE 7.—Vertical distribution of scandium-46 labeled sand after 125 hours of flume operation (station 2).

sectional area—8 cm.^{2}) of labeled sand. Measurements were made by interposing various thicknesses of sand between the source and detector. These determinations were made under "poor geometry" conditions. The results are presented in figure 8. The half-thickness value, i.e., the thickness of sand required to attenuate one-half of the incident photons, was found to be 2.5 cm. as measured to the surface of the probe. Extrapolation of this value to the center of the phosphor would give a value of 5 cm. under these particular operating conditions. The tenth-value, i.e., the amount of sand required to attenuate 90 percent of the photons, is found to be 9.5 cm. when extrapolated to the center of the detecting crystal.

The total linear absorption coefficient for sand can be calculated from equation 2:

$$\mu_o X_{1/2} = 0.693$$

$$\mu_o = \frac{0.693}{5} \qquad (3)$$

$$\mu_o = 0.138/\text{cm.}$$

Linear and mass absorption coefficients are related in the following manner:

$$\mu = m\rho \qquad (4)$$

where μ and m refer to the linear and mass coefficients, respectively, and ρ refers to the density of the absorber. The linear (or mass) absorption coefficient of water is 0.065/cm. (or cm.^{2}/gm.) for 1.1 Mev gamma rays. This value can be taken as the mass absorption coefficient of sand. Equation 4 can, therefore, be used to estimate the linear absorption coefficient of sand, bulk density 1.7 g./cm.^{3} The value of $\mu =$ (0.065) (1.7) becomes 0.11/cm. which is in reasonable agreement with the experimentally determined value of 0.14/cm.

Knowledge of the "average thickness" of burial combined with the linear absorption coefficient of the sediment for the particular gamma ray energy would permit a correction to be applied (equation 1) for attenuation losses of buried tracer. Such a correction assumes a uniform vertical distribution of tracer as illustrated in figure 9, *a*. If the tracer is overlain by sediment as shown in figure 9, *b*, a correction for attenuation losses for both the zone of uniform burial and the overlying sediment could be applied. The latter may be calculated from the following relationship as derived by Glasstone (*6*).

$$f = \frac{\lambda}{2H}\left[\tfrac{1}{2} - F_2\frac{(H)}{\lambda}\right] \qquad (5)$$

where f = fraction of radiation escaping from a self-absorbing slab source;
λ = the relaxation length of the shielding for the given radiation = $1/\mu$;
μ = linear absorption coefficient;
H = thickness of the source;
$F_2(q) \equiv q^2E_3(q)$;
$E_3(q) \equiv \int_q^\infty \frac{\exp(-p)}{p^3}\,dp$;
$q = H/\lambda$;
$p = R/\lambda$;
R = distance between source and detector.

Calculation of Sediment Discharge

Equations 1 and 5 can readily be applied to the activity survey of the channel bed after 125 hours of flow. Roughly, 55 percent of the activity present at stations 0, 1, and 2 was received by the detector. Forty-two percent of the total activity initially placed in the channel (station

0) remained in stations 0, 1, and 2 after 125 hours of flow; the remaining 58 percent of the activity was distributed throughout the channel reach, i.e., 34 feet. An average sediment transport velocity (V_s) of 0.0688 ft./hr. is obtained from the weighted longitudinal distribution of tracer (corrected for vertical distribution) for the 125-hour survey. An estimate of the sediment discharge (Q_s) for the 0.35- to 0.297-mm. sand can be made from the following relationship:

$$Q_s = W\, d\, V_s\, f_s\, a_s \qquad (6)$$

where Q_s=sediment discharge, pounds per hour;

W=width of channel=1.16 ft.;

d=average depth to which sediment movement occurred=0.0917 ft.;

V_s=average sediment transport velocity=0.0688 ft./hr.;

f_s=fraction by weight of 0.35 to 0.297 mm. sand found in the transported sand=0.08;

a_s=bulk density of the sand = 1.06 lb./cu. ft.

A sediment discharge value of 0.062 lb./hr. is obtained by calculation. The measured discharge rate (0.35- to 0.297-mm. fraction) was found to be 0.074 lb./hr. for the first 30 hours of flow.

Future Investigations

The wooden flume (1 by 1 by 200 feet) will be operated on a "put and catch" system for both water and sediment. Early investigations will be restricted to low flow conditions with some analyses of sediment transport attempted during the flow, but with the major emphasis placed on the horizontal and vertical distribution of tracer after termination of flow. Both equilibrium and nonequilibrium flow conditions will be investigated. Emphasis will be placed on the quantitative analysis of the total tracer. The differential scintillation spectrometer described herein with scaler-timer combination will be used for the analyses of the tracer distribution. Several specific conditions and techniques will be considered. They may be briefly outlined in the following manner:

(1) The use of short half-life isotopes will alleviate the problem of storage of contaminated sand. Several possible isotopes are available with a wide range of gamma photon energies. The lower energy gamma photons will be adequate for these investigations, because emphasis will be placed, at least in the beginning, on the analyses of dry sand beds (i.e., after termination of flow).

(2) If available, radioactive glasses will be used as tracers for the bulk sand. Such glasses

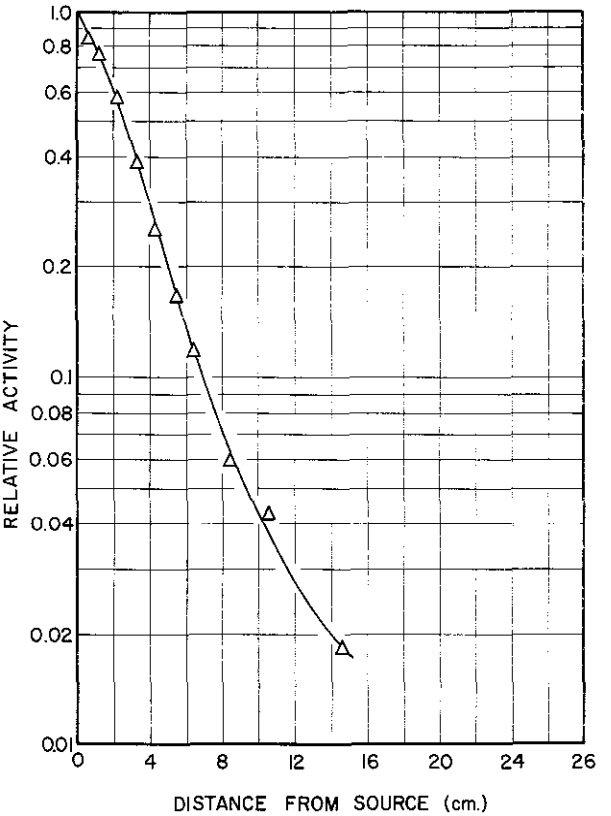


FIGURE 8. — The attenuation of scandium-46 photons by sand (bulk density 1.70 g./cm.^{3}; differential spectrometer; 1.12 Mev peak, 1-volt gate).

are currently available (with a variety of isotopes) in certain restricted density and size ranges. The preparation of special melts (specific gravity 2.65) with the desired isotope has been under investigation for several months. Radioactive glass produced from a single melt (batch) would possess the following desirable properties:

(a) It would be mass labeled, i.e., the specific activity per gram of glass would be constant throughout the batch. The photons received by the detector would be proportional to the mass of sediment transported.

(b) The melt can be crushed, ground, and sieved to produce a desired grain size distribution.

(c) The physical and chemical stability of such glass greatly reduces the possibility of tracer loss by abrasion and dissolution.

(d) A standard melt would provide material for the standardization of the detection system. In other words, glass containing a known specific activity per gram could be obtained for measurement of the sensitivity and efficiency of

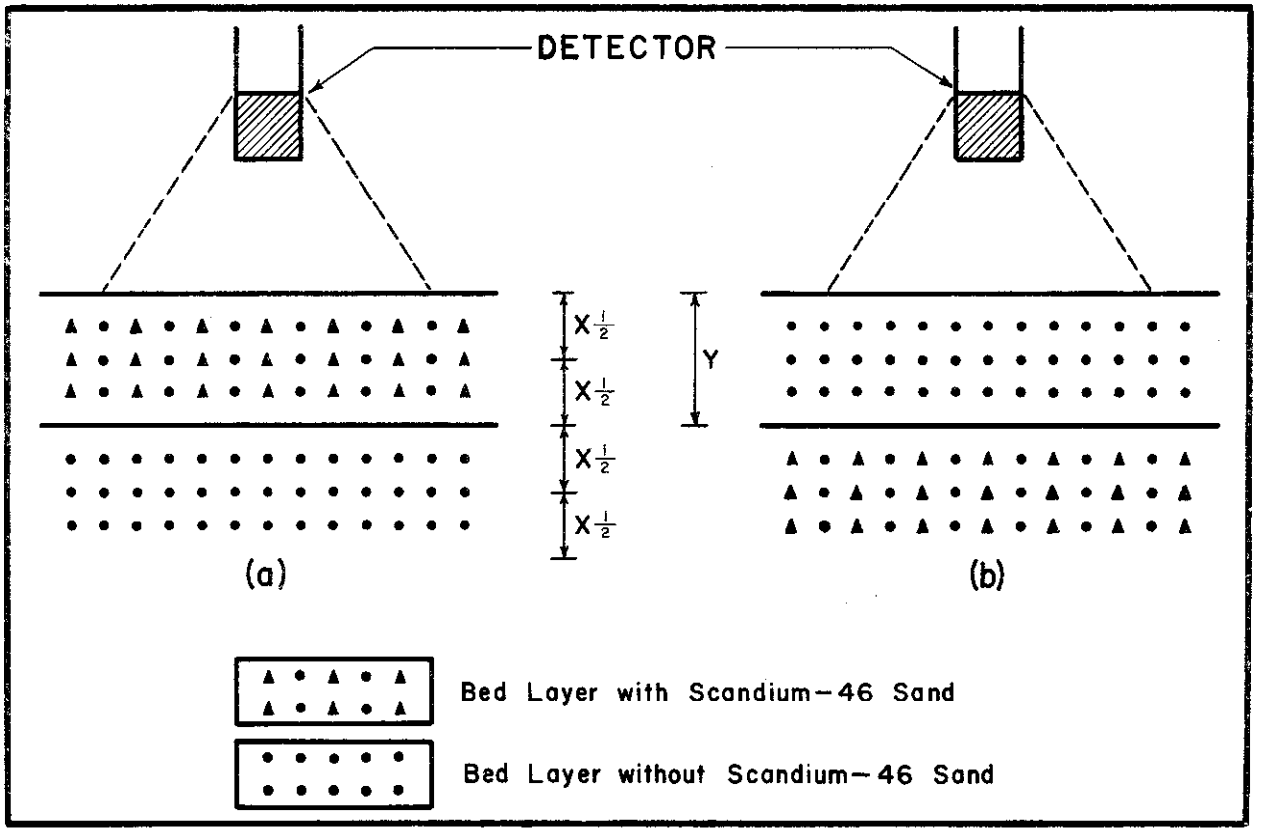


FIGURE 9. — Ideal vertical distribution of labeled sand: *a*, Uniform burial without overlying sediment; *b*, uniform burial overlain by nonradioactive sediment.

the instrument. Such material would be invaluable for investigating various geometries and spatial distributions of tracer.

(3) Lead shielding will be employed to define the geometry of the detector and source, i.e., tracer in the channel bed. It may be necessary to replace the present 2-inch shield with a smaller one which is more easily manipulated by point gage equipment. The thickness of the shield will depend upon the energy of the particular isotope.

(4) Transport studies of selected grain sizes and total size distributions matching the bulk sand will be considered.

(5) Analyses of the channel bed for both horizontal and vertical distribution of tracer will be conducted. Total analyses will be attempted. In other words, a summation of the activity levels from various cross sections of the bed (as seen by the detector from a well-defined geometry above the bed) will be compared to the initial amount of tracer placed in the channel. The attenuation of photons from buried tracer will cause the summed values as detected above to be smaller than the initial amount of deposited tracer. Careful measurements of the vertical distributions will then be conducted by removing thin layers of sand from the bed with subsequent measurement of the activity present in each layer and measuring the activity remaining in the bed.

In some situations it may be possible to correct partially for the attenuation losses of the buried tracer. Corrections will be made, assuming uniform vertical distributions of tracer, by use of equations 1 and 5. Situations involving nonuniform vertical distributions of tracer combined with interstratified layers of nonradioactive sediment that are most likely to be under these conditions are speculative. Nevertheless, it will be informative to make such approximations, using a model distribution, and to compare the corrected values of previously measured activities with the total amount of tracer initially placed in the channel.

(6) In the majority of experiments, the tracer will be mixed uniformly with sand from a definite cross section in the channel bed and replaced to simulate the original bed configuration. The tracer applied in this manner will

appear to the detector as a volume-distributed sample with self absorption, i.e., represented by equation 5.

(7) The hydraulic parameters will be measured so that tracer distributions will be meaningful. Comparisons of the calculated and measured sediment discharges will be informative.

(8) The average transport velocity of the sediment can be calculated from the weighted distribution of the tracer; subsequent calculations of the sediment discharge can then be made by using equation 6.

The ultimate objective of the tracer program is to investigate sediment transport in the field. These measurements in both overland flow, i.e., gross erosion, and in channels will require quantitative analyses. Certainly there is little need for qualitative evaluations of sediment transport in agricultural watersheds, e.g., the directional movement of sediment.

The spectrometer described herein, mounted in a vehicle with auxiliary power supply, will be used for these field investigations. The scintillation probe will be connected through 500 feet of cable to the spectrometer. This will facilitate measurements of activity in areas nonaccessible by the vehicle. Vertical core samples of the particular area, "fixed" in gelatin or in polyester resins to keep them intact, can be returned to the laboratory for further analyses. This will provide some information on the vertical distribution of the activity.

Analysis of the vertical distribution of activity will be necessary not only for estimating the amount of buried tracer but also in defining the average depth of particle movement, d, that is the zone through which sediment transport has occurred. The analysis and interpretation of the "average" or "effective" depth of particle movement and tracer distributions will be extremely complex for nonequilibrium events. The analysis is simplified considerably for an equilibrium flow, e.g., where the bed configuration consists entirely of dunes of a certain mean amplitude, with no net accretion or scour occurring in the bed.

Problems associated with the determination of d can be more clearly illustrated by considering an application of radioactive tracers of specific interest to the Agricultural Research Service, i.e., the measurement of bedload discharge in ephemeral sand bed streams. For example, assume that a definite cross section (injection strip) of the channel bed is uniformly labeled with the tracer. Furthermore, assume that (1) the tracer is placed to a depth sufficient to embrace the deepest dune troughs present during the flow, and (2) the tracer sand is mass labeled, i.e., that the total size distribution of sand found in the channel bed is labeled so that the activity detected at any point downstream after the flow has subsided will be proportional to the total mass of tracer and not to any specific size fraction.

During any flow event, considerable change in bed configuration might be expected. Dunes of different amplitude and duration plus local scour (and fill) with subsequent removal (and burial) of tracer from the injection strip will occur. A certain fraction of the labeled sand will be found distributed downstream from tne injection strip after termination of flow. Vertical core samples will be taken from the injection strip at frequent intervals extending the width of the channel.

Assume that the vertical distribution of the tracer can be quantitatively measured for all the core samples. Each core sample could be fixed in a polyester resin for subsequent analysis in the laboratory under well-defined analytical conditions. These cores can be segmented at various intervals, and the activity present in each segment compared to a like segment in the nondisturbed, uniformly distributed portion of the core sample. The vertical distribution of activity in each core will represent the total or overall series of events that occurred during the flow. Changes in bed configuration will be reflected in the activity distributions of the selected core samples. For example, certain depths will be noted where abrupt changes occur in the activity levels. These depths may denote that dunes of that mean amplitude were present during part of the flow event. The duration of flow involving dunes of that mean amplitude, however, is also reflected by that depth (*3*).

Obviously, any one type of sediment transport may be partially or totally masked by other types, resulting in an overall remaining distribution of activity that cannot be readily separated into its various components. The crux of the experiment then is found in the interpretation of the vertical activity distribution within the injection strip. Hydrographic records and changes in bed configuration determined by soundings, e.g., utilizing the Dual Channel Stream Monitor, would assist in evaluating and interpreting the final activity distributions. In any event, the estimation of d will be difficult; in many situations it will be highly speculative.

Summary

In this investigation the radioisotope scandium-46 was utilized to measure the sediment transport velocity and sediment discharge of 0.35- to 0.297-mm. sand in a small laboratory flume. Because a large part of this study was devoted to a review of previous sediment trans-

port investigations utilizing radioactive tracers, a summary of the literature review is given in addition to the specific experiment with scandium-46.

Review of Literature

Labeling of sediments for transport investigations has been accomplished on both natural and artificial sediments. Natural sediments, or their primary constituents, have been irradiated in slow neutron fluxes or permitted to sorb the desired radioisotope. Artificial tracers, e.g., glasses, have been prepared by fixing the desired element in the glass, crushing, and sieving to obtain the required size distribution followed by irradiation to produce the tracer material. Incorporation of the radioisotope prior to grinding has also been performed, but this requires special handling facilities and poses a greater health hazard to the personnel involved.

The choice of a suitable radioisotope for the tracer depends upon a number of factors. It is the consensus of previous investigators (primarily concerned with the movement of sediments in estuaries and on the ocean floor) that the isotope should emit strong gamma radiations with energy levels near 1.0 Mev. This point has been stressed, because it is the only radiation capable of penetrating any measurable depths of sediment and water.

The isotope must possess a half life appropriate to the duration of the investigation. Selection will also depend on the method of labeling to be used. If the element is to be fixed in glass with subsequent activation in a slow neutron flux, then consideration must be given to (1) its ease of incorporation in the glass and (2) its ease of activation, i.e., its cross sectional capture of slow neutrons in the reactor. On the other hand, if the material is to be sorbed by the natural sediment, it is necessary to consider the permanency of fixation in the test environment. In any case, its hazard to the public must be carefully considered.

Sediment transport investigations have been largely concerned with the redistribution of tracer material placed on or near the bottom of seas and estuaries. Emphasis has been placed on the direction of sediment transport, the rate of dispersion of sediment, and the path of sediment movement as defined by wave action or tidal currents. The results of such investigations have been largely qualitative in nature. The only exception has been the work of the Hydraulics Research Station in Wallingford, England, under conditions of equilibrium flow in a large laboratory flume.

The measurement of radioactive tracers is discussed in terms of the different detector systems available. The principal advantage of the differential scintillation spectrometer compared to the Geiger-Mueller system is its ability to accept only those pulses corresponding to the characteristic gamma photon energies of the particular radioisotope. In addition, the spectrometer permits (1) the rejection of already existing activity, i.e., background, having emission energies different from the tracer, (2) rejection of scattered radiation from attenuation of the photons in passing through matter, (3) use of different tracers simultaneously, i.e., radioisotopes having well-defined energy spectrums whose primary photon energies can be separated by pulse height discrimination, (4) the accurate measurement of high levels of activity without loss of counts from "dead" time of the system, and (5) within limits the calibration of the detector with regard to spatial distribution of the tracer.

Laboratory Flume Experiments

A preliminary investigation of sediment transport was conducted in a small laboratory flume. Scandium-46, a gamma emitter with a half life of 85 days, was employed. Natural sand (0.35 to 0.297 mm. in diameter) was labeled by the sorption technique. Ten grams (roughly 10^5 particles) of labeled sand was placed in a narrow band on the surface of the channel bed (50 feet long by 1.16 feet wide). The channel bed was subjected to a constant water discharge of 0.0148 c.f.s. for 5 days. No sediment was added during the experiment. In situ measurements of the tracer movement, made with a differential scintillation spectrometer, provided a semiquantitative estimate of the tracer distribution for various flow-time intervals. Vertical and lateral distributions of the tracer were qualitatively determined at various cross sections of the channel. Tracer sand was buried to a depth of 1.2 inches for the flow condition imposed upon the bed.

The attenuation of the gamma photons in passing through the sand were measured by interposing various thicknesses of sand between a plane source of scandium-46 and the detector (scintillation probe). The total linear absorption coefficient (μ_0) was calculated to be 0.138/cm. Corrections for attenuation of the buried tracer were made for the 125-hour activity distribution survey. An average sediment transport velocity of 0.0688 ft./hr. was calculated for the 0.350- to 0.297-mm. fraction of sand. The sediment discharge for this fraction was calculated to be 0.062 lb./hr. Measured sediment discharge of 0.074 lb./hr. was determined for the first 30 hours of flow.

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APPLICATION OF RADIOACTIVE TRACERS IN THE STUDY OF SEDIMENT MOVEMENT

[Paper No. 61]

By D. W. HUBBELL and W. W. SAYRE, *engineers, U.S. Geological Survey*

Abstract

Radioactive tracer techniques were employed in order to investigate the dispersion and transport of bed material in a test reach of the North Loup River near Purdum, Nebr. Sand particles, labeled with iridium-192, were used as tracers to enable observation of the natural dispersion and transport processes.

The amount of radioactivity and the number of tracer particles required for the experiment were determined by considering the sensitivity of the radiation-detection system, the characteristics of the test reach, and radiological safety factors.

In the experiment, the tracer particles were released from a line source that extended across the bed of the stream. As the tracer particles were transported and dispersed downstream, their longitudinal and lateral distributions in the bed were observed by periodic surveys with a sled-mounted scintillation detector, and their vertical distribution in the bed was observed by monitoring core samples. Information obtained from a laboratory calibration of the radiation-detection system under simulated field conditions was used to reduce the field data to a set of tracer-particle concentration-distribution curves.

The results of the field study indicate a potential for the wide application of radioactive tracer in sediment studies.

Introduction

One relatively new measuring technique that is being applied more and more frequently in studies of sediment transport is the radioactive tracer technique. This technique is essentially different from more conventional methods in that the displacement of individual particles or groups of particles is measured rather than such characteristics as the flux of sediment particles past a cross section, the net volumetric change in a reach, or the displacement of channel boundaries. As a result, the radioactive tracer technique is particularly useful in studies of the dispersion of sediment, the rate of travel of sediment particles, and in other studies concerned with the fundamental mechanics of sediment movement.

Although radioactive tracers can be extremely useful, a number of factors must be considered in their application. In general, the accuracy of measurements increases as the level of radioactivity increases. However, the higher the level of radioactivity, the greater is the possibility for harmful contamination and radiation exposure. As a result, in order to achieve a satisfactory balance between the level of accuracy and the level of radioactivity, all aspects of an experiment must be considered in detail. The fundamental factors for consideration include (1) the kinds and required accuracy of measurements to be made, (2) the radionuclide to be used as a tracer and the means for labeling the sediment particles, (3) the amount of the tracer to be used, (4) the method of introduction of the tracer into the flow system, (5) the method of detection of the radioactivity, and (6) the reduction of the data.

This paper discusses briefly some of these factors as they were considered in the design and implementation of a field experiment on the North Loup River, Nebr., that was conducted by the U.S. Geological Survey in November 1960.

Experimental Design

The primary object of the field experiment was to study the dispersion and transport of bed-material particles in a natural stream. Hence, it was decided to release radioactive tracer particles as a line source and then to monitor the longitudinal distribution of the particles at various times after release. The site on the North Loup River near Purdum, Nebr., was selected because of favorable experimental conditions. In particular, the North Loup River, which is in the sand hill region of north-central Nebraska, maintains a relatively constant water discharge of about 250 c.f.s. for prolonged periods of time; draws from, rather than contributes to, the ground water; has a bed normally composed of large dunes approximately 1.5 ft. high and 15 ft. long; and has bed material having a median diameter of about 0.29 mm. The specific reach used for the experiment is a man-made cutoff about 1,800 ft. long and 50 ft. wide that is fairly straight except for two minor bends, has stable banks, and is isolated from any community.

It was concluded that optimum results would be obtained by using a single, narrow-size range of tracer particles having a median fall diameter that was slightly coarser than that of the bed material. Such a size range was selected to insure that the tracer particles would not be in suspension any significant part of the time, but that the particles in the size range containing the greatest proportion of the bed material

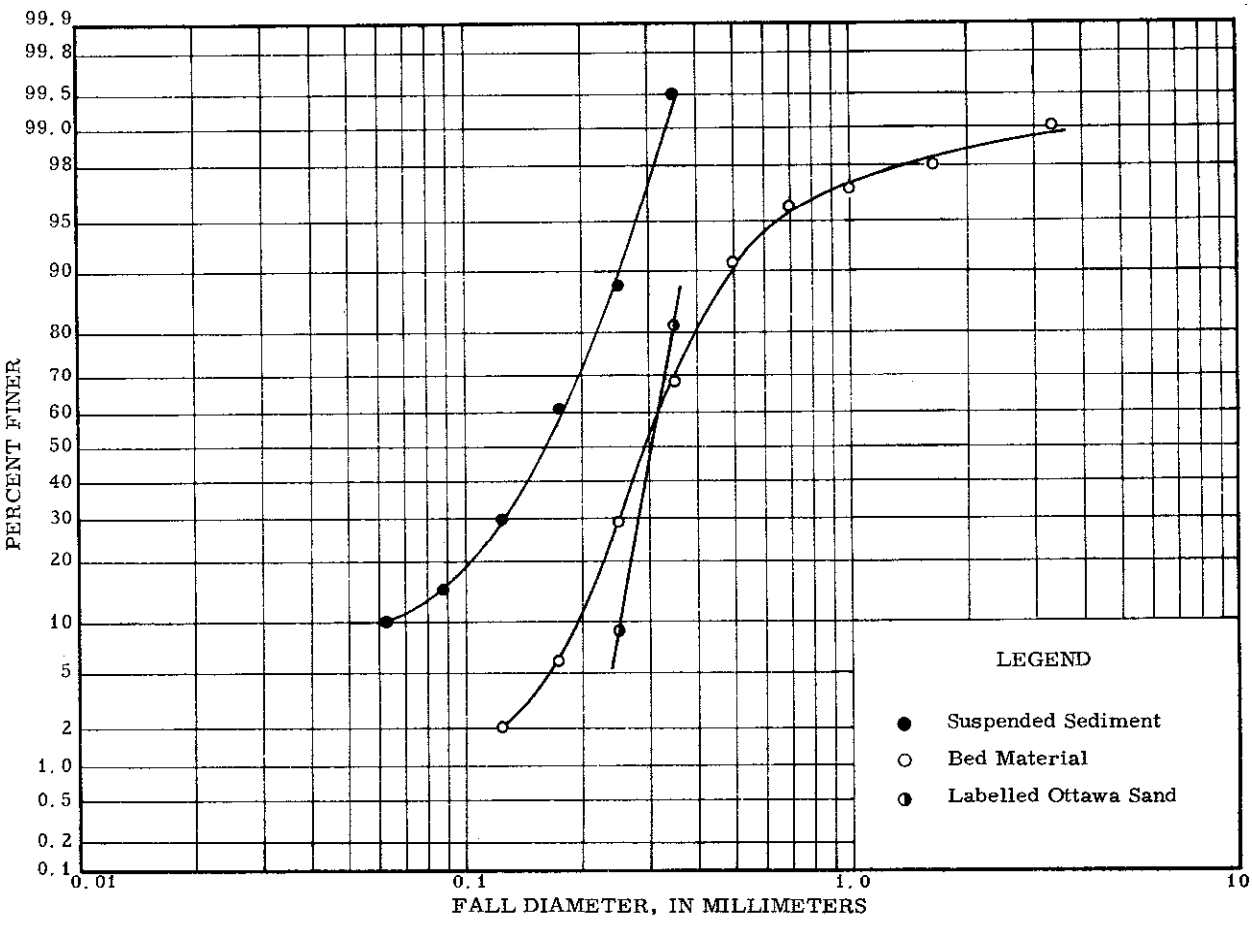


FIGURE 1. — Size distributions of measured suspended sediment, labeled Ottawa sand, and bed material.

would be represented. The selected tracer particles had a median particle size of 0.305 mm. and a distribution as illustrated in figure 1.

A comparison of the distribution of the tracer particles with those of the suspended sediment and the bed material, which are also presented in figure 1, shows that most of the tracer particles could have been in suspension at one time or another but that suspension probably occurred only infrequently.

Because the tracer particles would move primarily as bedload, in situ measurements seemed to be the most feasible means of defining the longitudinal dispersion of the particles. Such measurements virtually necessitated the use of a gamma-emitting radionuclide, because the penetration range of alpha and beta radiation is limited to only a few centimeters in water. A scintillation detection system was selected to monitor the gamma radiation because of its versatility and its inherently high sensitivity. The system consisted of a scintillation detector, pulse-height analyzer, count ratemeter, strip-chart recorder, and scaler. With this system, the number of gamma rays (photons) that interact with the crystal in the detector can be counted and displayed as a count rate or a count accumulation. Inasmuch as the number of gamma rays emitted from a source is directly proportional to the amount of radioactivity within statistical limitations and provided all other factors are the same, the count rate also is proportional to the amount of activity. Hence, if the amount of activity on each particle is proportional to the particle weight, measured count rates indicate the weight of tracer particles near the detector, provided, of course, suitable calibrations have been made.

For the experiment, the scintillation detector was housed in a watertight aluminum casing and mounted on a sled (fig. 2) that was dragged along the streambed. The detector was attached to the sled by a four-bar parallelogram linkage and supported on the bed by a wooden dish. This system allowed the detector to remain in continuous contact with the streambed during

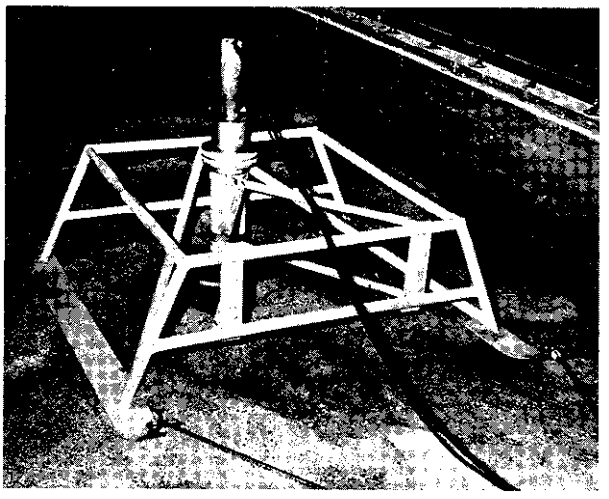


FIGURE 2. — Sled and scintillation detector. Note the parallelogram linkage that allows the detector to move independently of the sled and the wooden dish-shaped piece that supports the detector on the bed.

transport and to move vertically independently of the sled.

Iridium-192 was selected as the tracer because it (1) emits gamma rays having energies that are readily detectable underwater (0.14 to 0.90 Mev); (2) has a half-life of 74 days, hence, it would decay slowly enough to be readily detectable during the entire experiment, but rapidly enough to preclude long-term contamination of the stream; and (3) could be procured commercially already plated onto sand particles. The plating process consisted of wetting the sand in a solution containing the iridium-192 and then baking the sand at 700° F. for several hours. Tests showed that with continuous agitation of the particles immersed in water approximately 25 percent of the activity was abraded from the particles in 24 hours. Although it is impossible to relate the amount of abrasion the particles received in the tests to that likely to occur in a natural stream, 24 hours of agitation no doubt is equivalent to a relatively long time. Krone [1] also plated the radionuclide onto the sediment by an adsorption process. However, other methods of producing tracer particles have been used. For instance, Inose and others [2] incorporated the radionuclide into glass particles that simulated the sediment, and Lean and Crickmore [3] irradiated natural sand in a nuclear reactor. A summary of the various labeling techniques that have been used is given by Feely and others.[4] One important factor in connection with sand plated with a radionuclide is that the measured activity is proportional to the surface area of the particle rather than to its weight. If a wide range of particle sizes is labeled, this factor must be considered in converting a measured count rate to a weight of tracer particles; however, if the particles are relatively uniform in size, the count rate is also approximately proportional to particle weight.

The amount of radioactivity required for the experiment was determined by considering the natural background radiation, the volume of sand throughout which the particles would be dispersed, the decay rate of the activity, the adsorption characteristics of the sand and water, the efficiency of the detection system, and the geometrical orientation of the detector to the tracer particles. The effects of the latter three items can be characterized by the "sensitivity" of the detection system, which is the count rate per unit of activity per unit volume under specific experimental conditions. The sensitivity of a detection system to a uniformly distributed source such as sand tracer particles can be estimated by defining a count-rate attenuation function through measurements of the count rate from a weak point source buried at different locations and distances relative to the detector and then integrating the attenuation function over the volume through which the tracer particles will be distributed.

Once the sensitivity is determined, the amount of radioactivity (M) required for an experiment can be computed from

$$M = \frac{(R_o - R_b)\ V\ e^{0.693t/T}}{S}$$

in which

$R_o - R_b$ is the minimum net counting rate over background that is required during the experiment for statistical significance.

V is the estimated volume through which the tracer particles will be dispersed at the end of the experiment.

S is the sensitivity of the detection system for the conditions of the experiment.

$e^{0.693t/T}$ is a correction factor for radioactive decay.

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where

t is the duration of the experiment.
T is the half-life of the radionuclide.

In the design of this experiment, a net minimum counting rate equal to one-half of the background rate was considered acceptable for the condition of a uniform distribution of tracer particles throughout the test reach. By assuming that the experiment would last for one month, it was determined that 40 millicuries of iridium-192 would be required.

When radioactivity is labeled on sediment particles, each particle is a separate source and the count rate measured by the detection system depends not only on the amount of radioactivity, but also on the distribution of the tracer particles. As a result, sufficient particles must be used in order to minimize the possibility of significantly nonuniform distributions within the bed. If the distribution of tracer particles throughout the bed is random, as the concentration of tracer particles increases, the relative distribution of the tracer particles tends to become more even and random variations in the number of tracer particles per unit volume of bed material tend to have less effect on the count rate.

The random variations in the number of particles in a given volume can be characterized by the coefficient of variation or the relative standard deviation. If the variation in the number of tracer particles in a given volume is assumed to follow the Poisson distribution, the coefficient is $100/\sqrt{N}$, where N is the mean number of tracer particles in the given volume of bed material. For experimental design, a coefficient of about ± 6 percent seems to be adequate when the volume associated with N is defined as that volume of the bed from which 50 percent of the measured gamma rays (counts) emanate when the sand bed contains a uniformly distributed source of infinite extent and N is taken as the required number of tracer particles within the volume at the end of the experiment when the tracer particles are distributed over the test reach.

The coefficient of variation provides only an index to the expected variation in count rate attributable to random variation in the number of tracer particles. However, if a relatively long ratemeter time constant is used and the detector is moved along the bed, the fluctuations in count rate due to local variations in the distribution of tracer particles are damped appreciably. Sensitivity measurements for the iridium-192 and the detection system used in the experiment indicated that 50 percent of the counts emanate from within 4.4 in. of the center of the 2-in. by 2-in. detector crystal, which is equivalent to a volume of 116 cu. in. The coefficient of variation with this volume for the total reach volume of $1{,}800 \times 50 \times 1.5$ cu. ft. and the 40 lb. of sand (approximately 4.6×10^8 particles) used in the experiment was 6.6 percent.

In order to establish the exact relationship between count rates and the weight of tracer particles, the detection system was calibrated under conditions that simulated the actual experimental environment. The calibration was made in a 4-ft. diameter by 4-ft. high tank in which known weights of tracer particles were mixed with known volumes of natural bed material similar to that in the North Loup River. Count rates were measured for different depths of uniformly distributed particles and for different ratios of the weight of tracer particles to the volume of natural sand (concentrations). In order to reduce the background count rates, a depth of 9 in. of water was maintained over the sand bed throughout the calibrations.

The results of the calibrations, which were for two different instrument settings, are shown in figure 3. This figure shows that the adjusted count rate, which is a count rate adjusted for radioactive decay, varies with depth as well as with concentration for all depths less than

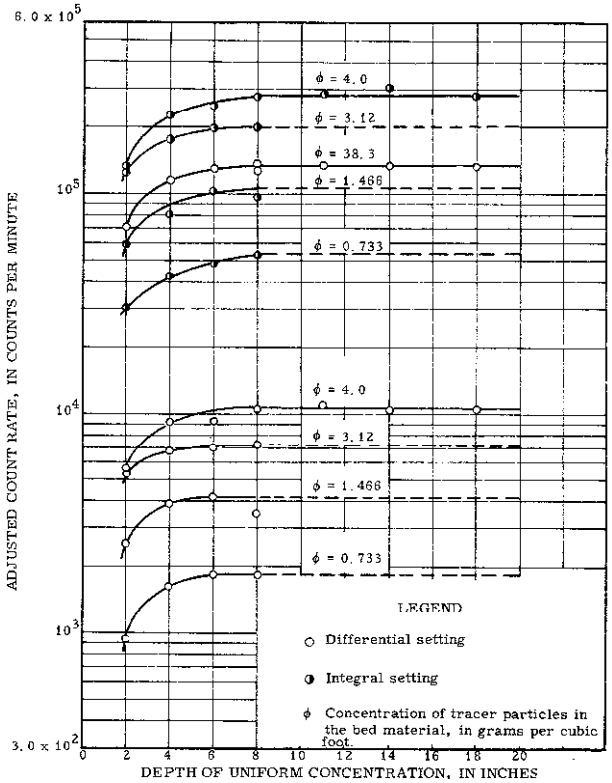


FIGURE 3. — **Variation in adjusted count rate with the depth to which different concentrations of tracer particles are uniformly mixed.**

about 8 in. For depths greater than about 8 in., the adjusted count rate varies only with concentration. In the experiment it was anticipated and eventually borne out that the particles would be distributed to depths greater than 8 in. As a result, the only part of the calibration curves required for converting the count rates observed in the field to concentrations were those parts for an 8-in. depth.

Data Collection and Results

In order to study longitudinal dispersion, the tracer particles were introduced essentially as a line source by depositing the 40 lb. of tracer particles in 2-lb. lots at 2-ft. intervals across the width of the channel. Each 2-lb. lot was labeled with 2 millicuries of iridium-192. The labeled particles were placed on the streambed by use of an apparatus consisting of an electric can opener and a movable funnel tube that were mounted to the stern of the boat. In the placement (dosing) operation, the funnel tube was lowered until the bell at the bottom of the tube rested on the streambed, then a can containing the tracer particles was opened with the can opener and poured down the funnel tube to the streambed.

About 2 hours after the dosing operation was completed the first longitudinal traverse was made. Just before this traverse, as well as all subsequent ones, the detector sled was placed by hand upstream from the dosing section and the natural background radiation was recorded for about 2 minutes. After background counting, the boat was released and maneuvered downstream. The longitudinal traverses were made with the boat and the detector sled arranged in tandem as is shown in figure 4. The

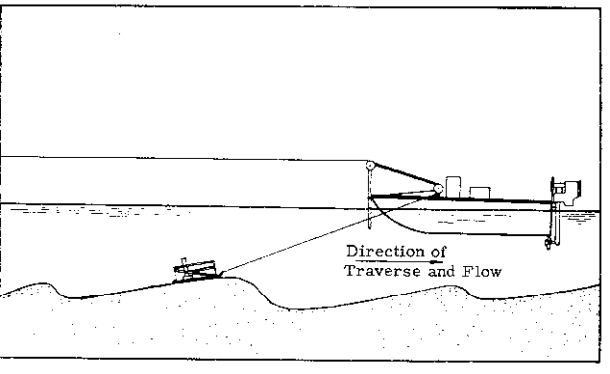


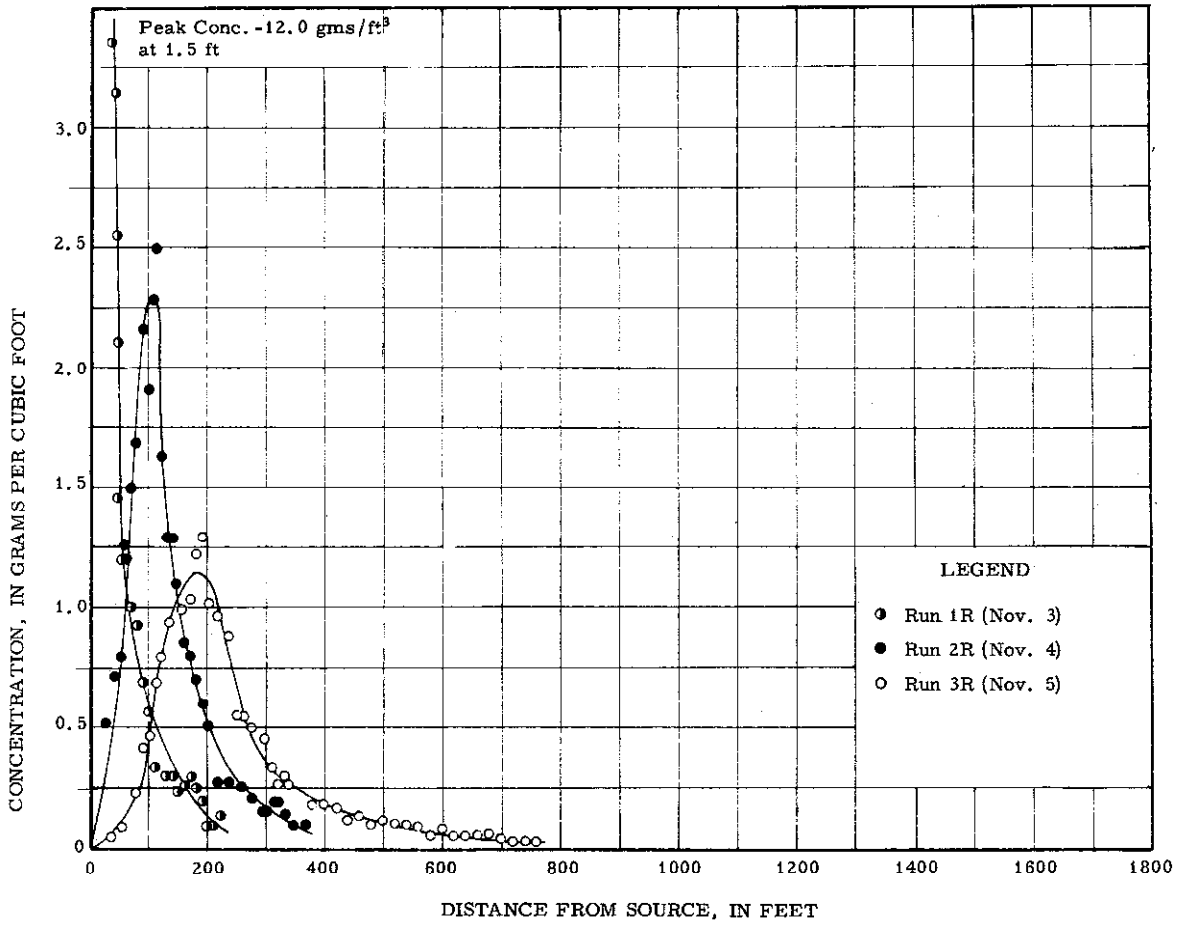
FIGURE 4. — Arrangement of boat and sled for the longitudinal traverses.

boat, which faced upstream and pulled the detector sled, was maneuvered downstream by means of an outboard motor having reverse controls. Mounted on the front end of the boat was a distance-measuring cable and reel that functioned both as a stay line and as the distance marking system. In operation, the cable, which was fixed at the upstream end of each segment of the test reach, was unwound manually so that the boat and sled moved downstream at a uniform controlled rate. When buttons that were located at definite intervals along the cable tripped an event-marking switch, ticks were made on the recorder charts. In this way the recorder charts were provided directly with a distance coordinate.

The tracer particles were tracked daily by making longitudinal traverses with the radiation detection equipment down the left and right sides of the stream along paths that were roughly one-third of the channel width away from the bank. Typical data produced by the longitudinal traverses are shown in figures 5 and 6. The distribution curves in these figures were established by adjusting the observed count rates for radioactive decay and then converting the adjusted count rate to a concentration with the calibration curves. One particularly interesting occurrence is indicated by the distribution curve in figure 6. This curve shows that on about the sixth day after dosing, a relatively large number of tracer particles appeared just downstream from the dosing section along the left side of the stream. Apparently, tracer particles were trapped in a deep trough and remained buried beyond the range of the radiation detection equipment for several days. About the sixth day after dosing, the trapped particles were released by a train of dunes having deep troughs. The particles then began to disperse in a normal manner. As a result of the temporary storage, the distribution curves for the left side of the test reach were less indicative of the general distribution that applies for the average flow condition than those for the right side.

In addition to the longitudinal traverses, several traverses were made to define the lateral distribution of the tracer particles. Figure 7 shows the lateral distribution of the particles 185 ft. downstream from the source 2 days after dosing and 415 ft. downstream from the source 5 days after dosing. The deficiency of particles along the left bank near the source is evident in the distribution for the second day after dosing. The distribution curve for the fifth day after dosing indicates a reasonable uniformity of the tracer particles.

In order to develop accurate concentration-distribution curves from the observed count rates and the calibration curves, it was essential that the depth to which particles were dispersed generally was greater than 8 in. and that no general vertical concentration gradient per-



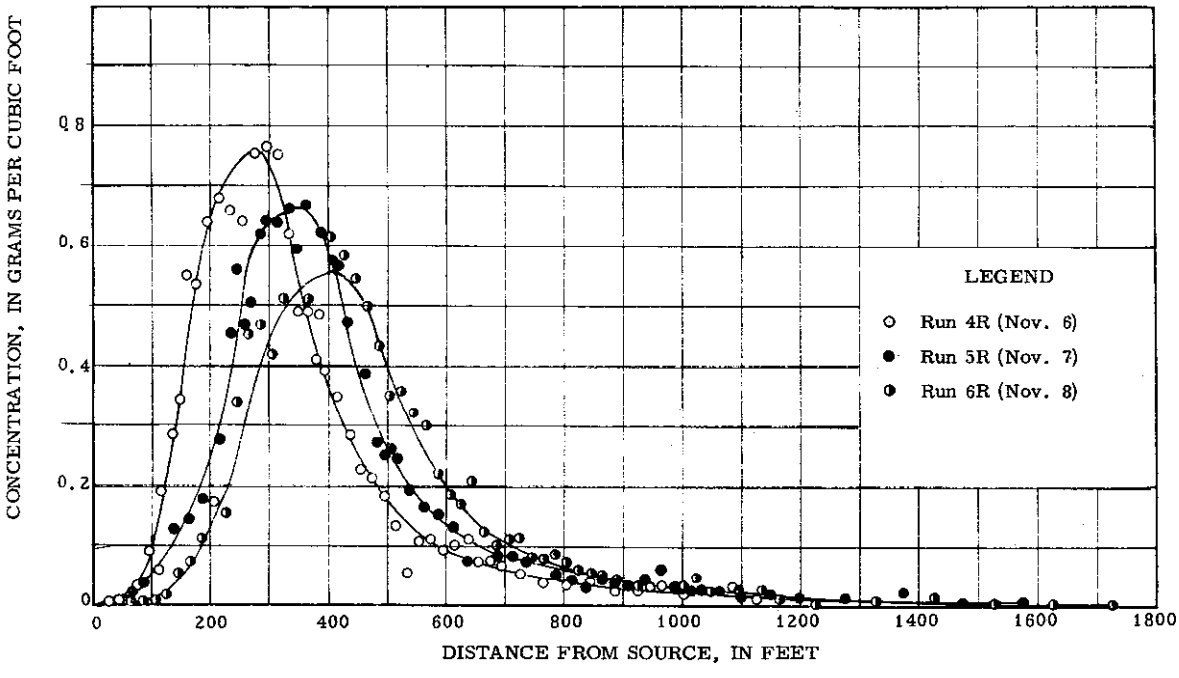


FIGURE 5. — Longitudinal distribution of labeled particles along the right side of the channel, Nov. 3 to 8, 1960.

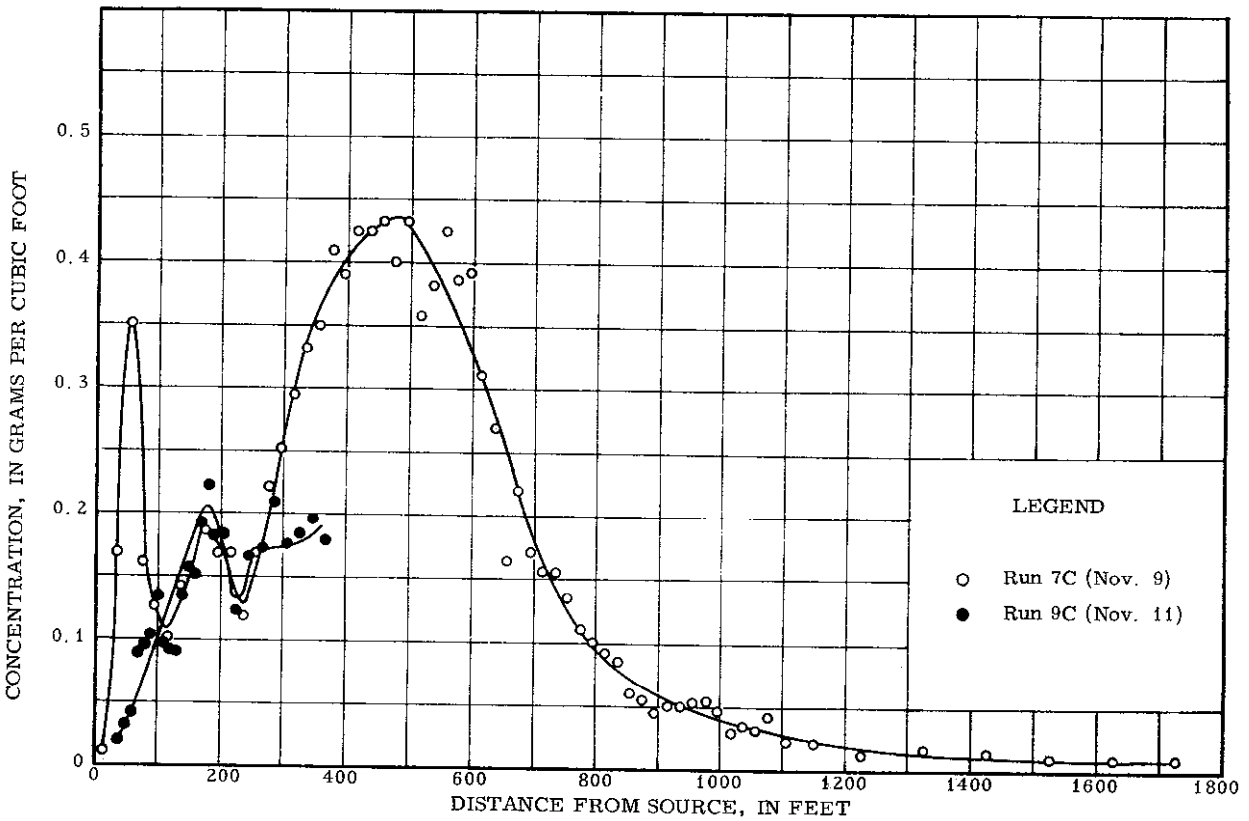


FIGURE 6.—Longitudinal distribution of labeled particles along the center of the channel, Nov. 7 and 9, 1960.

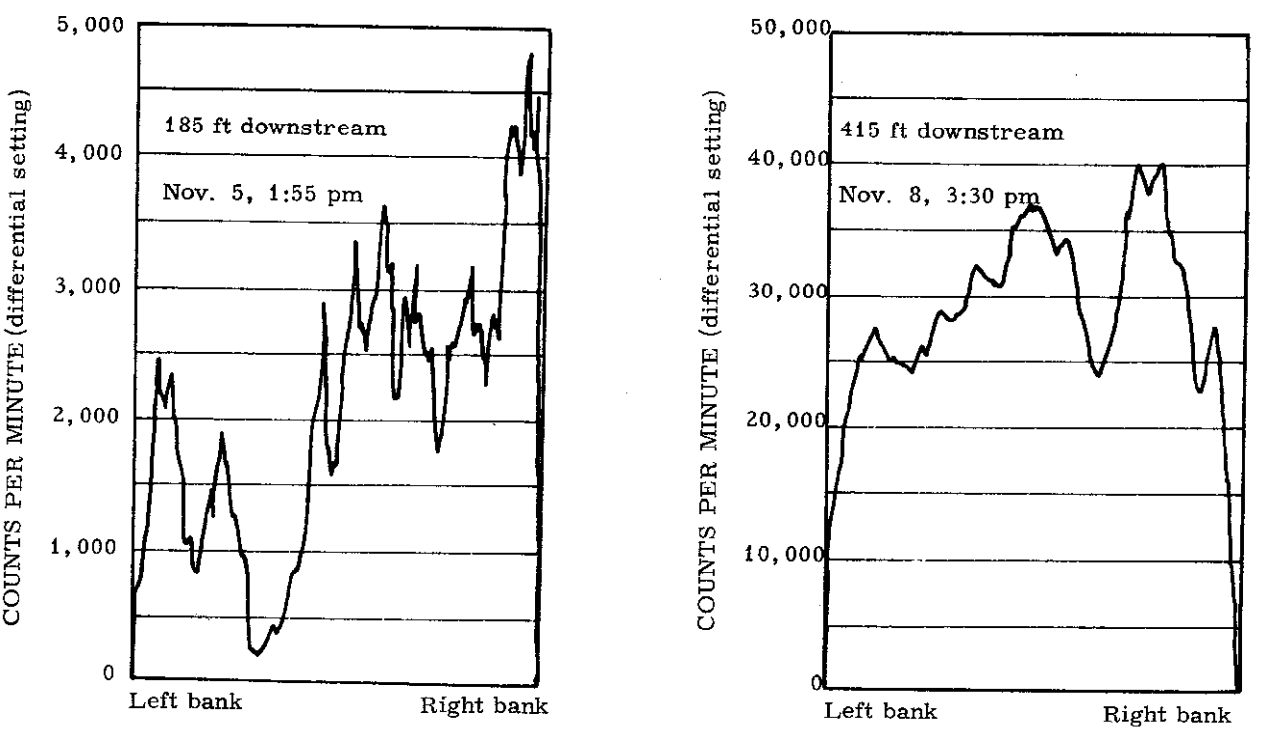


FIGURE 7.—Lateral distribution of labeled particles downstream from the source on Nov. 5 and 8, 1960.

sisted throughout the streambed. Two different methods were used to verify the fact that these two conditions existed. The first method consisted of defining the vertical distribution of tracer particles into the bed. This was done by collecting core samples of the bed material at various lateral and longitudinal positions in the test reach. The core samples were collected with a 1½-in. diameter sampler capable of withdrawing a 3-ft. long core. For analysis, the cores were ejected in 2-in. increments with a hand jack attached to the end of the sampler plunger and each increment was counted separately with a scaler by using the scintillation detector and sled in an inverted position.

Some typical observed vertical distributions are shown in figure 8. A common characteristic of all of the vertical distributions was that the count rate was highly variable with depth into the bed and no semblance of a continuous vertical distribution pattern was definable. Probably

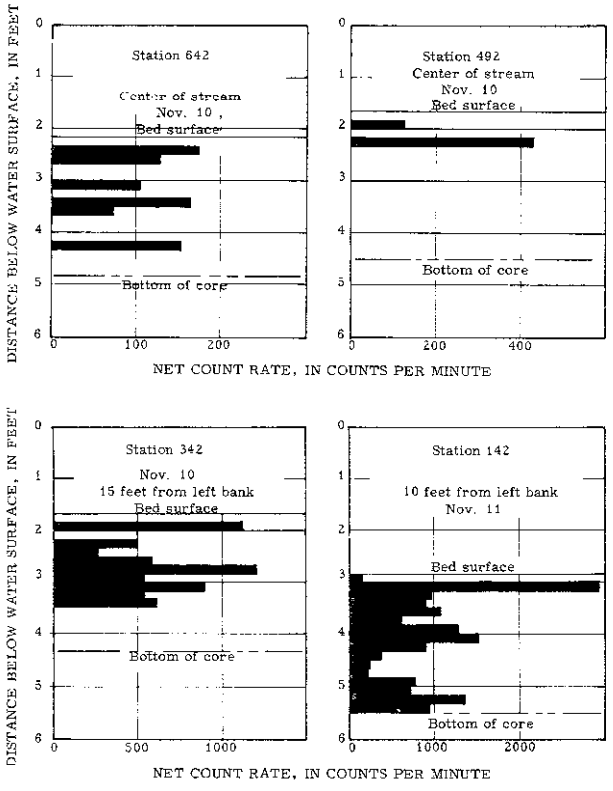


FIGURE 8. — Vertical distribution of labeled particles at selected verticals on Nov. 10 and 11, 1960.

the variations resulted largely from the passage of dunes having different amplitudes, mean elevations, and concentrations of tracer particles. Unfortunately, as a result of these irregularities of the movements of the dunes, neither the distance below the water surface, distance below the bed surface, nor depth of activity could be used to normalize the vertical distribution of count rate. Hence, the distributions from the separate verticals could not be combined to provide an average gradient. However, the distributions from the separate verticals were combined to show that the average depth of the zone through which the sediment particles moved was about 1.45 ft. Because the average depth of the zone of movement was considerably greater than 8 in. and no discernible vertical distribution pattern appeared to exist, it seems reasonable to assume that the observed longitudinal and lateral distributions generally were representative of the actual distribution of the tracer particles.

The validity of converting the observed count rates to concentration of tracer particles with the calibration curves for an 8-in. depth was further verified by comparing the distribution of observed count rate with records of the bed configuration. The comparisons showed that no correlation existed between the depth of the zone of particle movement, as characterized by a profile of the bed configuration over which the detector passed, and the observed count rate. The apparatus for measuring bed configuration was a dual-channel ultrasonic depth sounder,[5] which was mounted on the bow of the boat with one transducer facing upward and another facing downward so that both the water- and bed-surface profiles were recorded. A typical record of the bed configuration is shown in figure 9.[6]

Conclusions and Application of Results

The results of the field study and subsequent laboratory flume studies indicate that the concentration of labeled sediment particles at any point in the streambed at any time can be determined conveniently, accurately, and safely with radioactive tracer techniques. This conclusion is important, because it implies the feasibility of determining, by experiment, distributions of labeled particles in a stream channel with respect to space and/or time for a wide variety of conditions. For example, by selective labeling, certain aspects of the behavior and distribution of particles having definite sizes, shapes, specific gravities, and other characteristics can be determined. Furthermore, by using a scintillation detector in conjunction with a pulse-height analyzer, different radionuclides can be distinguished from one another so that

[5] KARAKI, S. S., GRAY, E. E., and COLLINS, J. DUAL CHANNEL STREAM MONITOR. Amer. Soc. Civil Engin. Proc. 87, No. HY6: 1-16, illus. 1961.

[6] HUBBELL, D. W., and HAUSHILD, W. L. DISCUSSION OF "DUAL CHANNEL STREAM MONITOR." Amer. Soc. Civil Engin. Proc. 88, No. HY4: 287-291, illus. 1962.

several selected labels can be used simultaneously to observe the behavior of different types of particles in the same experiment. For example, the authors currently are conducting laboratory flume experiments in which three different radionuclides are used to trace, simultaneously, the movement of three different sizes of particles in a bed material of naturally graded sand. Measurement of concentration distributions of labeled particles will provide new insights into the phenomenon of sediment transport and will supply the kind of data necessary for analyzing sediment transport as a random phenomenon that can be characterized by probability laws and treated as a stochastic process.

In addition, observed longitudinal distributions of tracer particles released from a uniformly distributed source provide a means for computing sediment discharge directly with a continuity-type equation. The computation is based on the idea that (1) the time rate of movement of the mean of the distribution is equivalent to the average particle velocity and (2) the extent of the tracer particle distribution within the bed defines the effective cross-sectional area through which the particles move.

The examples given above indicate only a few of the many possible applications of radioactive

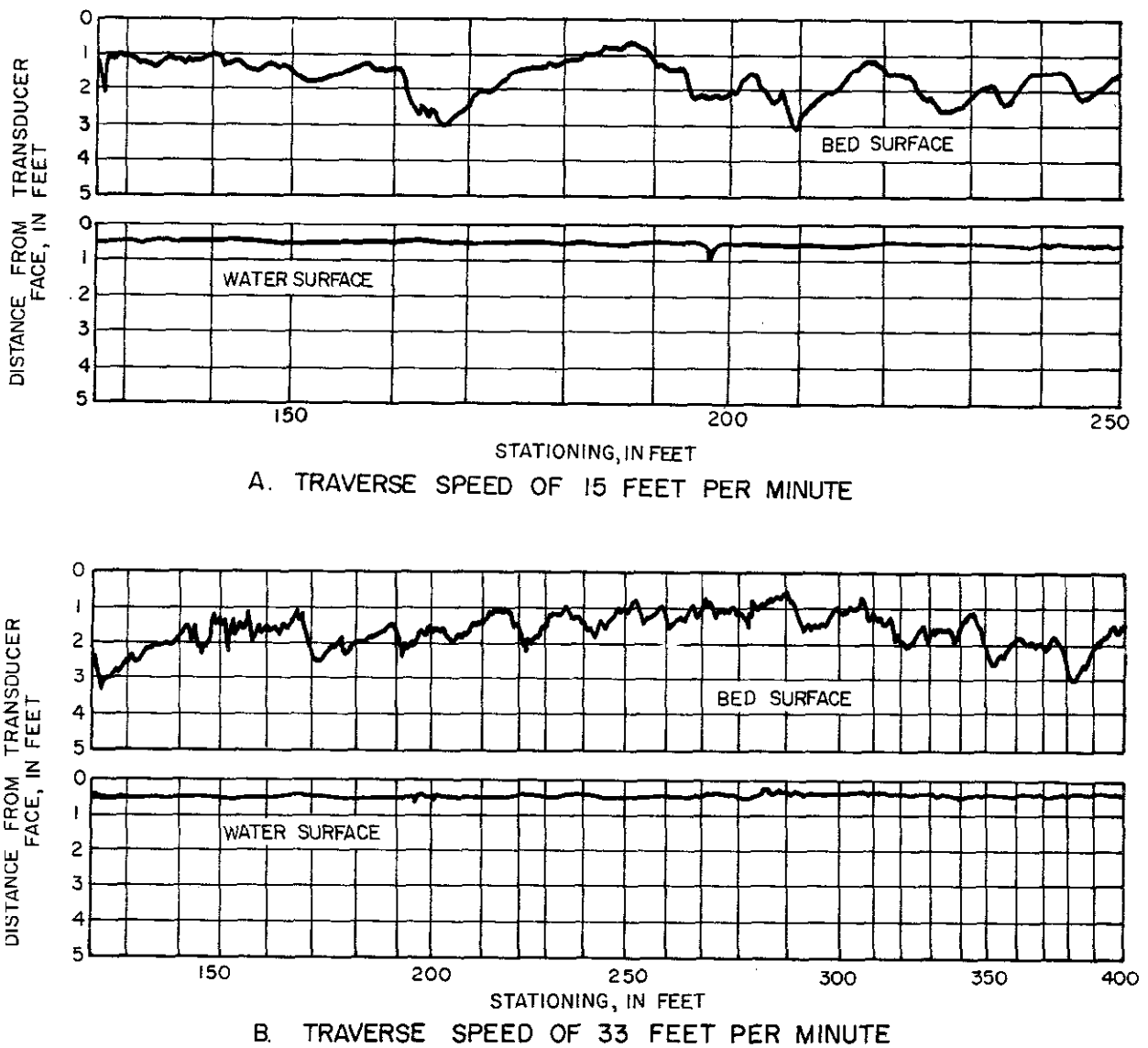


FIGURE 9.—Water- and bed-surface profiles defined by the dual-channel stream monitor. (From Hubbell and Haushild, 1962.)

tracers in sediment transport research. Probably only a small part of potential value of radioisotope techniques in sediment research has been realized. It is the authors' hope that this paper will serve to assist others with the development of new applications.

USE OF RADIOISOTOPES IN SEDIMENT TRANSPORT STUDIES

[Paper No. 62]

By R. S. CUMMINS, JR., *engineer*, and L. F. INGRAM, *research physicist, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss.*

Synopsis

In 1956 the U.S. Army Engineer District, San Francisco, in conjunction with the University of California, undertook a sediment tracing project using radioactive tracers in San Francisco Bay. This initial investigation indicated the tremendous possibilities of the technique for sediment studies and assisted with the development of special tracing equipment and methods of labeling sediments. In 1960 the U.S. Army Engineer Waterways Experiment Station selected a group of trained personnel to continue such tests for other Corps of Engineers offices as the occasion arose. Tests have since been conducted at Galveston Harbor, Tex., and in Cape Fear River, south of Wilmington, N. C. The tracer material selected for use in each instance was gold[198], which is a short half-life, low energy gamma emitter. The labeling methods employed, however, were different in that the Galveston test material was a manufactured particulate tracer with metallic gold fused in glass. The Cape Fear test material was existing sediment with the radioisotope adsorbed on it. The labeling procedure in the Cape Fear test involved the introduction of the isotope into a slurry of natural sediment. The slurry was agitated, which allowed the tracer to be adsorbed on the particles. The mixture was then released in a uniform pattern at the predetermined placement site. No problems other than those inherent in the handling of radioactive material and contamination of tracing equipment were encountered.

Surveying operations, both background and postplacement, were conducted, using either transits or sextants for position fixing at time intervals to permit correlation of location and activity. A continuous recording of bottom activity was made during the life of the tracer.

The results obtained were plotted on charts that showed either development patterns of movement between daily surveys or deposition areas for transported sediment. Since the labeled material may be lost, either by being covered or by being widely dispersed, quantitative measurements are extremely difficult. Limitations are imposed by (1) rapid rate of movement required because of the short-lived isotope; (2) difficulty of surveying in areas containing wreckage or debris; and (3) inability to measure labeled material in suspension or in flocs.

Introduction

This paper is not intended to be a treatise on the subject of radioisotopes, but rather a brief discussion of the Corps of Engineers efforts with the use of radioisotopes for sediment tracing. In 1956 the U.S. Army Engineer District, San Francisco, entered into a contract with the University of California to develop techniques and equipment suitable for use in connection with a comprehensive investigation of the sedimentation and shoaling processes in San Francisco Bay. From this effort came a method of labeling sediment, the development of an underwater detector to trace sediment movement and other field equipment, handling procedures, and field techniques.

In 1960 the U.S. Army Engineer Waterways Experiment Station selected and trained a group to continue such tests for the Corps of Engineers as the occasion arose. Because of the specialized training and equipment involved, it was more practical to utilize a single group for tracer purposes than to train and equip groups in each District Office that might have use for tracer techniques.

Since establishment of the test group at the Vicksburg Waterways Experiment Station, tests have been conducted at Galveston Harbor, Tex., and in Cape Fear River, N. C. Additional tests are scheduled early in February at Galveston Harbor. In each instance the problem was to trace the movement of bottom sediment. Although the techniques employed were somewhat similar, the method of labeling the tracer material was different in each case.

The tests in Galveston Harbor were conducted in an effort to determine the path taken by resuspended sand that comes from a generally known source area. Galveston Harbor (fig. 1), which serves not only Galveston but also Houston and Texas City, is protected by stone jetties extending several thousand feet beyond the island itself. It was believed that material outside the North Jetty moved either through the jetty or around the end and into the ship channel, necessitating extensive dredging to

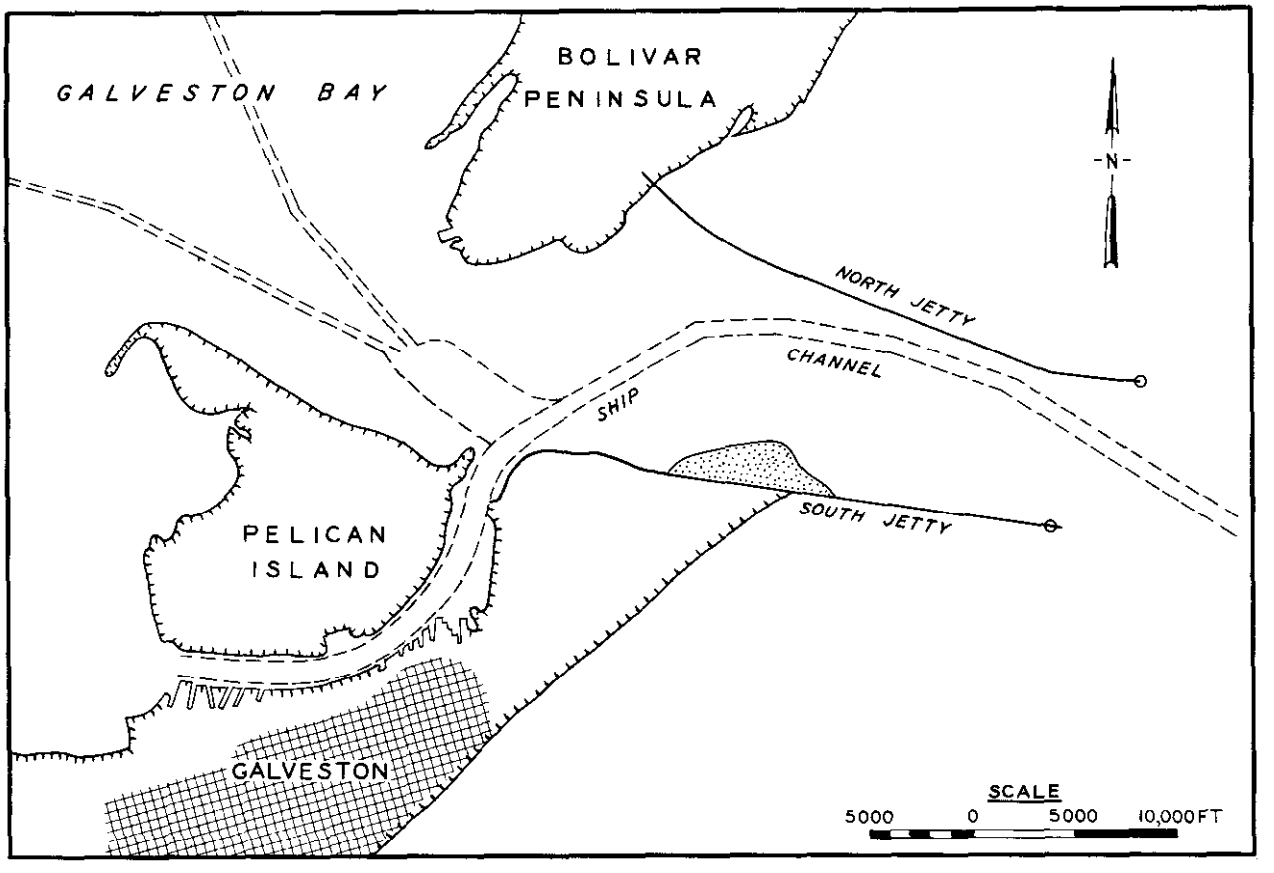


FIGURE 1. — Galveston Harbor and locations of stone jetties.

maintain channel depths. Therefore, the purpose of the North Jetty tests at Galveston, made during June 1961 and June 1962, was to trace the movement of material outside North Jetty to the shoaled areas within the ship channel. Similar tests are scheduled for the South Jetty. Three deposits of radioactive material were made in water depths of 3, 6, and 9 ft. during June 1961, and six deposits were made in deeper water during June 1962.

Tests were made in the Cape Fear River at the Sunny Point Army Terminal to determine the necessity of diking dredge spoil areas (fig. 2). The Army Terminal is located about 20 miles south of Wilmington, N.C. The entire wharf facility was dredged out of a shallow area and the spoil pumped across the river channel into a disposal area. Dock facilities consist of three wharves with 800-foot-wide basins connected by 300-foot-wide channels and served by three entrance channels. Shoaling in the wharf areas is extremely rapid and averages about 0.8 ft. of depth per month near the north wharf and about 0.6 ft. per month near the center wharf. Suspected source areas include not only the spoil areas in the vicinity but also those upstream from the terminal.

Prior to the actual conduct of tracer surveys, preliminary investigations are made of such items as velocity and current patterns in the vicinity of the problem area, bottom sediment samples, and dredging surveys. Such information is necessary to locate deposit areas, areas where surveying might be difficult or dependent on tide conditions, and areas containing wreckage that might hamper surveying operations.

License Requirements

Once it is decided that tracer techniques are feasible in a particular problem, the tracer material must be selected and a license obtained from the Atomic Energy Commission for procurement of the radioactive material and its use in a specified location. A separate license must be procured for each type of radioactive label and test location. The license request is filed through the Safety Division, Office of Chief of Engineers, and the office of Environmental Hygiene Agency of the U.S. Army Surgeon General. Standard forms are available for the pur-

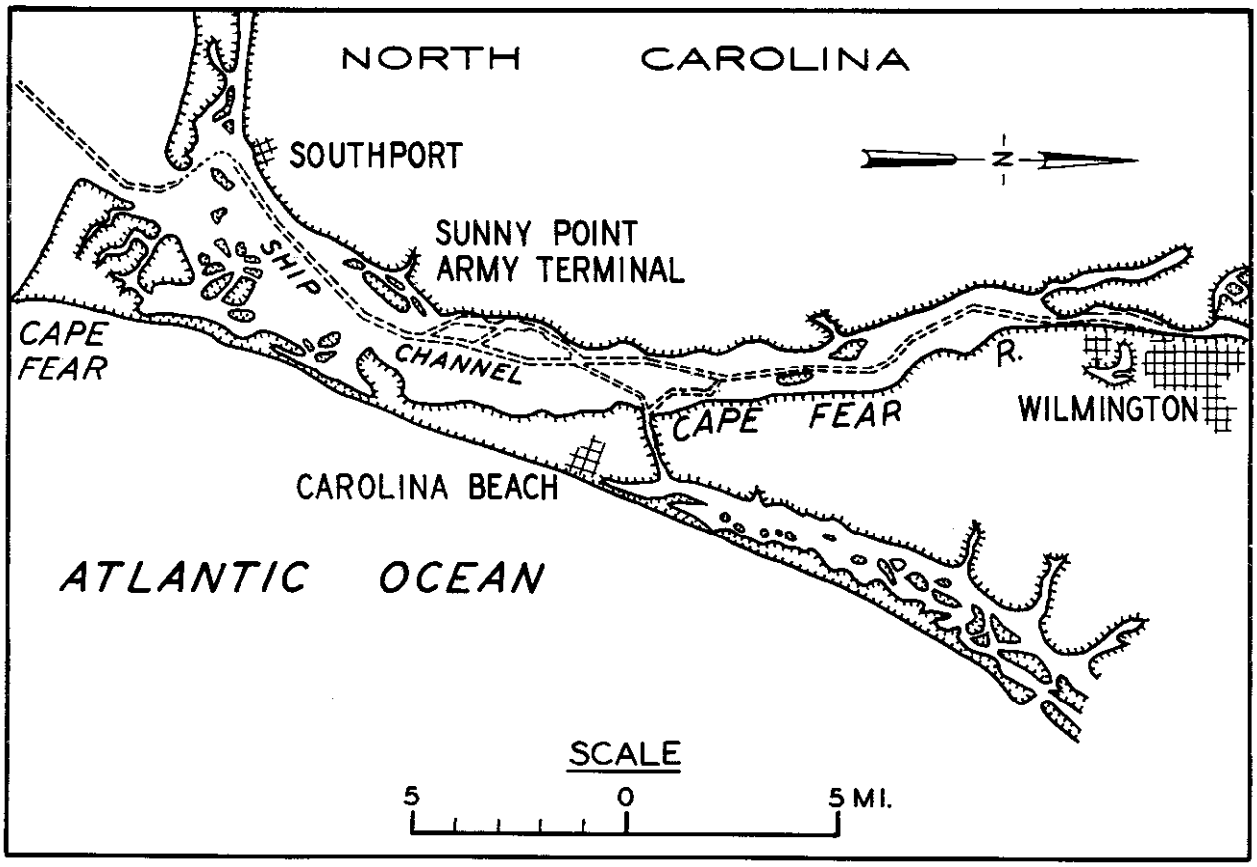


FIGURE 2.— Sunny Point Army Terminal, Cape Fear River.

pose. On these forms detailed steps of the entire operation must be defined. Information must be provided on the quantities of isotopes desired; handling, storage, and transportation; and safety precautions to eliminate possible health hazards to users and contamination of adjoining areas. To satisfy prelicense requirements, a public meeting with city, county, and State officials, representatives of the health department, and other interested agencies that might be affected must be held to explain the tests and obtain the approval of those concerned. Several public meetings were held in connection with the tests at Galveston and in Cape Fear River.

After the approving agencies are completely satisfied that there are no possible health hazards to the users or residents of the affected area, a specific license is granted for a period of 1 year. This license authorizes possession of a specified amount of a particular isotope for use as described in the license. Procurement of a license requires considerable lead time, but with experience in filing for such license, the time probably can be reduced. Increased experience of the licensee also helps decrease the lead time.

Selection of Isotope

The selection of the isotope to be used as the tracer is of considerable importance. Controlling factors are (1) duration of a single test or required half-life of the isotope to be used; (2) energy level of and type of emission desired from the isotope; and (3) the type of labeling procedure that will yield accurate data with regard to particle size. Consideration must be given to the fact that radioisotopes having significant half-lives are frequently elements required in the metabolic processes of marine organisms or animals. Also isotopes of higher energy create special handling problems and are not readily available in forms easily applied to various types of sediment.

If the entire sample of radioactive material is released into a flowing stream, the concentration of radioactive material cannot exceed the maximum permissible concentrations stated in the National Bureau of Standards Handbook No. 69.[1] For example, the maximum allowable concentration of gold ^{198} is given as 5×10^{-4} mi-

[1] U.S. NATIONAL BUREAU OF STANDARDS. MAXIMUM PERMISSIBLE BODY BURDENS AND MAXIMUM PERMISSIBLE CONCENTRATIONS OF RADIONUCLIDES IN AIR AND IN WATER FOR OCCUPATIONAL EXPOSURE. Handbook No. 69, 95 pp. 1963.

crocuries per cubic centimeter, or about 1.9 microcuries per gallon. The amount of tracer material required, however, must be sufficient to yield significant counting rates when the material is uniformly distributed over the problem area. The sensitivity of the detecting device governs the concentration required to yield a mathematically significant increase over the natural background count rate. An accepted significant increase under laboratory conditions, using a statistical nine-tenths error determination, is an increase of more than $1.645\sqrt{R}$, where R is the observed counting rate. (This value is the limit of error for a 0.10 level of significance.) In field tests, where background rates vary by as much as a factor of 10, it is not unrealistic to use the figure, $10\sqrt{R}$, as a significant value. The quantity of material to be labeled may be calculated by taking into consideration grain-size distribution, probability of deposition or movement, and activity obtainable per particle for the smaller particles.

The tracer material selected was gold^{198}, which is a low energy, short-life, gamma emitter. The energy of this isotope is spectral, that is, of nearly Gaussian distribution, at the photopeak of 0.41 Mev (million electron volts). The basis for selection of this particular isotope and tests of labeling sediment are discussed by Dr. Krone [2] in the first annual report.

Sediment Labeling

Two different labeling procedures have been successfully used at Galveston and in Cape Fear River. The bottom samples from Galveston Harbor were sand ranging in grain size from 10 to 140 microns, with a median size of about 40 microns. A manufactured glass containing 0.3 to 0.5 percent gold by weight was ground and graded to the proper size distribution to duplicate bottom sediments. The uniformity of samples taken from the source area permitted the use of such a method of tracer preparation. The manufactured glass was of such chemical composition that there were no long-lived isotopes present. It was necessary to add potassium in the glass to reduce the melting point to a practical value. Since potassium is a short-lived, high-energy isotope, the irradiated samples of glass were retained at Oak Ridge National Laboratories for 24 hours to permit decay of activity to maximum permissible shipping limits.

The bottom sediments in the Cape Fear River area were largely sandy silts, except for the fraction dredged from the Sunny Point Army Terminal, which was fine silt. This silty material was suitable for labeling by the adsorption method. The adsorption method of labeling is accomplished by adding a solution of gold^{198} dissolved in aqua regia to a slurry containing the natural sediments. The mixture is thoroughly blended and allowed to remain in residence for a period determined by concentration. The gold metal is adsorbed onto the soil particles, thus labeling them. This method was developed and used by the University of California in San Francisco Bay.[3]

Handling procedures resulting from the different methods of labeling create special problems and necessitate the development of suitable handling equipment. The particular tracer used at Galveston was ground, graded, and sealed in plastic bottles prior to irradiation at Oak Ridge, Tenn. The irradiated material was shipped to the test site in special sealed containers. The problem of placement was one of opening the containers and releasing the sediment at the proper depth in the selected area. This was accomplished by using a long aluminum tube attached to a boat, with a puncturing and flushing device mounted on the lower end of the tube (fig. 3). This device consisted of a three-bladed

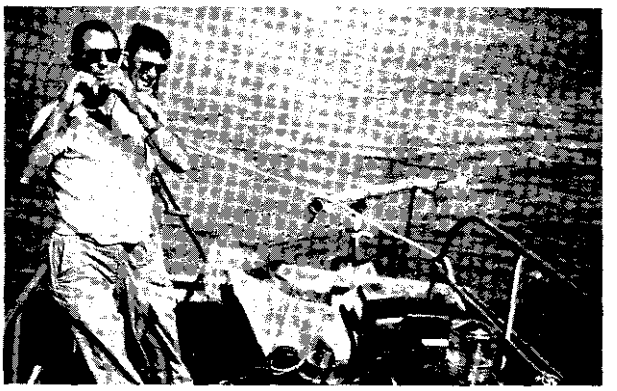


FIGURE 3. — Tracer release apparatus attached to survey boat.

spear that was fastened to a hollow shaft through which water was pumped into the punctured container (fig. 4). Annular holes in the bottom plate behind the spear allowed the mixture to flow from the container onto the harbor bottom. The plastic bottles were taken from the lead-

[2] KRONE, R. B., EINSTEIN, H. A., KAUFMAN, W. J., and SNYDER, N. W. SILT TRANSPORT STUDIES UTILIZING RADIOISOTOPES. Calif. Univ. (Berkeley) Hydraul. Engin. Lab. and Sanitary Engin. Res. Lab. Prog. Rpt. 1. 1957.

[3] KRONE, R. B., EINSTEIN, H. A., KAUFMAN, W. J., and SNYDER, N. W. SILT TRANSPORT STUDIES UTILIZING RADIOISOTOPES. SECOND ANNUAL PROGRESS REPORT. Calif. Univ. (Berkeley) Hydraul. Lab. and Sanitary Engin. Res. Lab. 1959.

KRONE, R. B., EINSTEIN, H. A., KAUFMAN, W. J., and SNYDER, N. W. SILT TRANSPORT STUDIES UTILIZING RADIOISOTOPES. THIRD ANNUAL PROGRESS REPORT. Calif. Univ. (Berkeley) Hydraul. Lab. and Sanitary Engin. Res. Lab. 1960.

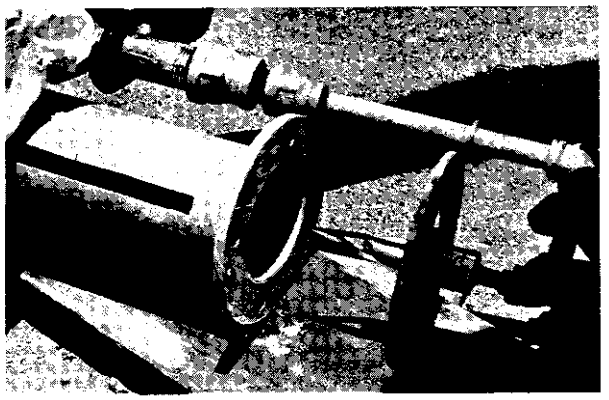


FIGURE 4. — Device for releasing sediment from container.

shielded shipping containers with 4-foot-long laboratory handling tongs, placed in the tube, forced down on the spear with a plunger, flushed, removed, and replaced in the shipping containers. In later tests during June 1962, aluminum containers were substituted for the plastic bottles as the plastic became brittle during the irradiation process.

The labeling of natural sediment, such as the silts and clays encountered in the Cape Fear River tests, required the use of a labeling vessel equipped with a mixing device, a method of introducing the tracer, and an ejection device, all of which had to be operated remotely. A cylindrical tank with truncated cone bottom was constructed (fig. 5). An electric motor drove a

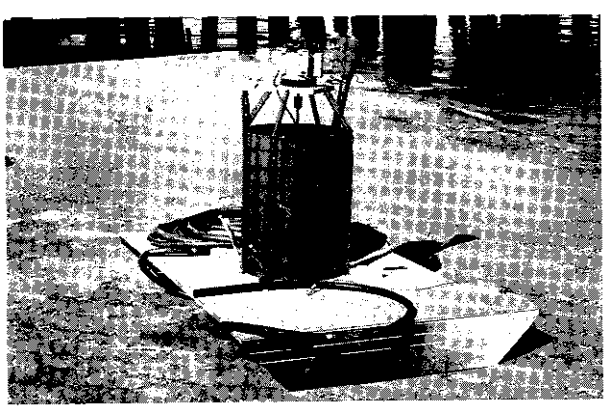


FIGURE 5. — Tank for mixing tracer and sediment.

mixing propeller, located at the bottom of the tank. A bottle-crusher tube, which terminated just above the propeller, was perforated to allow the slurry to circulate through the lower end of the tube. A hydraulic ejector was located outside the tank with the suction line entering from the top of the tank and extending to the bottom along the side. The discharge line consisted of a hose weighted at the end to place the sediment mixture on the bottom.

Instrumentation

Instrumentation requirements for tracing operations are rather complex and necessitate the use of a boat equipped with a stable alternating-current generator. The system consists of a sled-mounted radiation detector (fig. 6)

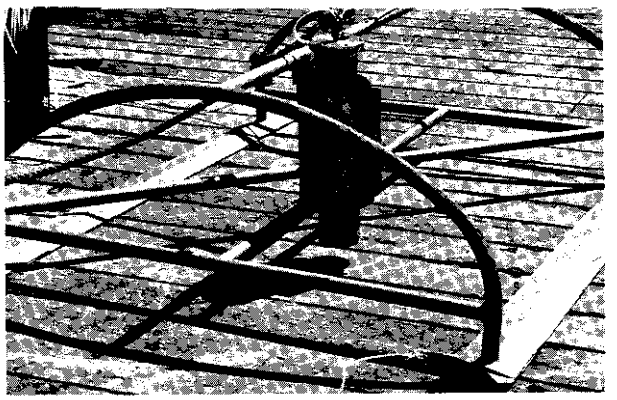


FIGURE 6. — Radiation detector mounted on sled.

that is towed along the bottom behind the survey boat. The detector is connected by a coaxial cable to the electronic equipment located on the boat.

The detector uses a scintillation crystal (thalium-activated sodium iodide) mounted in an aluminum cylinder and coupled to a photomultiplier tube with a three-stage transistorized preamplifier. The light pulses generated in the scintillator are converted to electrical pulses by the photomultiplier tube, and the preamplifier then increases the pulse power for transmission through the cable to an amplifier-analyzer located on the boat. The scintillation detector is more sensitive than a Geiger-Muller (G-M) system, and pulses produced by the scintillation detectors are proportional in size to radiation energy, whereas those from a G-M system are not. The pulses are amplified and fed to a discriminator circuit that counts or rejects the pulses, depending on the pulse height. Since the pulse height is proportional to the energy of the incident particle, energies above or below the desired level are rejected. This feature is desirable in that naturally occurring radioactivity or fallout does not interfere with the tracer activity measurements. As the energy of radiation from most isotopes is spectral, a band of energies is usually observed in which the greater portion of incident particles or rays from the isotope in question can be counted.

A strip chart recorder records continuously the pulse rate (which is proportional to bottom activity) in the area being covered. These recordings are marked at time intervals that can be correlated with location for plotting.

Survey Procedures

The survey boat at any time was located through the use of either sextants or surveyor's transits. Both methods were used with position fixes being made at predetermined time intervals or called intervals. Simultaneously with position fixes, time marks were placed on the activity recording so that the correlation could be made. Direct interpolation was made between location points on the charts for plotting variations in activity. Of the two systems used, the sextant method was more effective, since the observers were on the boat and plotting of survey paths was done as the operation proceeded. This system of constant location lends itself to better coverage of the affected area, as partially surveyed areas can be covered to define completely limits of material deposition.

Since the accuracy of the information obtained is no better than the accuracy of location measurements, constant checks and rechecks of location can be made by crossing and recrossing courses already run and visual checks between objects located on the charts can be made. The largest scale maps available were used for navigation and data reduction to reduce possible error.

The background radiation survey before deposition of the radioactive material is very essential, not only because it establishes a datum plane for later observations but also because the mathematical significance of activity measurements is a function of the background levels. For these reasons one must be certain that the area is thoroughly surveyed and that all areas of high or low activity are completely defined. Since silts, clays, and organic materials possess more natural radioactivity than sands or gravels, it is possible to construct (through careful data analysis) a deposition chart that defines the various material locations including hard clay areas. By drawing activity level contours, a marked resemblance can be seen of soundings showing deposited material. Thus background surveys can be used as guides to probable deposition areas for the after-placement surveys.

Placement of labeled sediment with optimum tide or flow conditions to obtain maximum movement is a situation that is difficult to obtain. In tests conducted by the Waterways Experiment Station, the assumed extreme conditions have been used as criteria for times of placement. At Galveston Bay, for example, the inshore deposits of material along the North Jetty were made at low slack water whereas the outer deposits were made at high slack water. This was done to prevent movement of the tracer while the material was in suspension. The times of placement of material in the Cape Fear River, however, were selected to encompass both extremes; that is, deposits were made with rising and falling tide conditions. This was done because the bulk of the sediment would remain in suspension for a sufficient time to allow movement regardless of the time of deposit.

The postplacement surveys were conducted in the following manner: After the material was placed, the survey operation was undertaken immediately. No set pattern of coverage or route was established. The areas of high activity were defined by sufficient tracing. Problem areas or areas of special interest were covered as time permitted. In the case of slow movement of the tracer, daily surveys reveal development patterns or gross tracer movement. Severe weather conditions or rapid movement in suspension will leave no evidence of a "trail" or indicated direction of movement. In these cases general coverage of the area will show pockets of material or sometimes random distribution of particles over large areas. After dispersion has progressed to the point where activity falls below the significant levels or the development patterns begin to recede, surveying may be discontinued.

Safety precautions as outlined in the license application were followed during the entire surveying operations. In particular, all beaches and shores in the immediate vicinity were monitored to insure no contamination. The boat (from which all operations were conducted) and the placement equipment were checked to insure freedom from harmful radiation. Significantly, the largest dose rate observed during the Galveston test came from the luminous dial of the boat's clock.

Data and Results

Background surveys were plotted in the form of contour maps. Each daily postplacement survey was reduced in a similar manner, the background subtracted to yield net count rate, and this value corrected for the decay of the radioisotope. The decay process of any radioisotope is an exponential function that follows the general equation $I=I_o e^{-\lambda t}$ where I is the present intensity of radiation, I_o is the initial intensity, e is the base of natural logarithms, λ is the decay constant for a particular isotope, and t is the elapsed time. For gold ^{198} (half-life 2.7 days) the activity correction factor for four elapsed days would be 2.8 by which those significant net readings would be multiplied. After the decay factor was applied to the count rates, these values were plotted on the charts in the proper locations. At this point, either activity contours

may be drawn to show daily movement patterns or areas of significant deposit may be shown also by contours or by cross-hatching. In cases where sufficient coverage was not obtained to define boundaries of the area affected, points indicating activity level were plotted on the survey path to show that activity was observed.

The initial observations made at Galveston showed general movement inshore from the inshore locations, with some rapid movement from severe weather encountered during the survey operation. Activity contours showed the dispersal of material with a small fraction of material passing through a small boat pass in the jetty (fig. 7). Since results did not show conclusively whether the bottom material went through or around the jetty into the ship channel, six additional deposits were made in deeper water near the end of the jetty. These tests showed considerable movement of tracer material around the end of the jetty, with random depositions in the ship channel and in the area between the two jetties. Movement of material was dependent upon weather conditions in the area. That is, it appeared that wave action caused the tracer material to be in a semi-suspended state wherein it was moved readily by the current. In one test, tracer material was observed 5,500 ft. from the drop location within 4 hours after placement. When waves were small, movement was slow. For long-term operations at Galveston, another license for using an isotope of long half-life (such as scandium [46] with a half-life of 85 days) would have been necessary. This, however, would permit determinations of transport processes and allow more comprehensive studies to be made.

Observations made in Cape Fear River showed deposition of material in the problem area within a short time regardless of the tide conditions at the time of placement. In fact, movement was so rapid that delineation of the actual paths of tracer material was not possible. However, the purpose of the test was served in that tracer material deposited in the area was detected in the wharf basins (fig. 8). Hence, it can be concluded that the dredged spoil area should be diked. One test, made with tracer

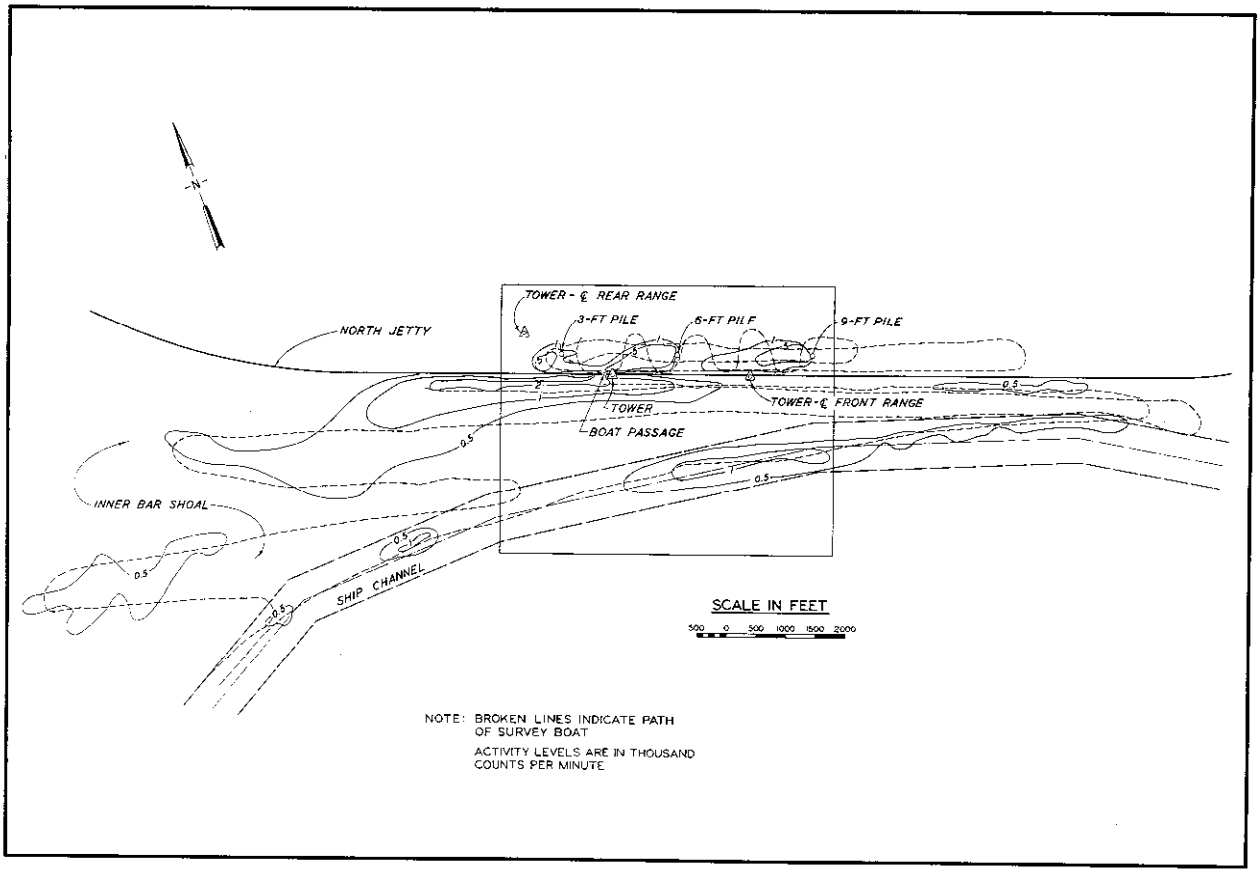


FIGURE 7. — Radioactivity contours on 10 June 1961; Galveston tracer tests.

material deposited on a shoaled area in the river well upstream from Sunny Point Army Terminal, revealed considerable shoaling of the dredged area by material transported by the river. Therefore, the shoaling problem at Sunny Point may only partially be alleviated by diking the spoil area.

Closing Remarks

The use of radioactive material for tracing sediments is only one tool to be employed in the overall analysis of a particular problem. It has limitations, but under certain situations can provide worthwhile results. Generally the information obtained from tracer tests is of a qualitative nature and pertains to relative rates of shoaling or concentrations of material in certain areas. Quantitative measurements are extremely difficult at present because of the nature of sediment movement and the tracer material itself. For example, as the detector passes over the bed there is no method of determining the difference between a single large particle of tracer or several small particles. Neither is it possible with the adsorption method of labeling to determine the exact amount of label that is adsorbed to the sediment particles. If any of the labeled sediment is shielded by being covered with inert bottom material or moves in suspension above the face of the detector, the observed counting rate is decreased. The inherent safety hazards resulting from use of long-lived isotopes prevent tests of long duration.

In spite of limitations, tracing techniques offer the only method for continuously tracing sediments in deep water without direct sampling. In addition, large areas can be studied within a relatively short time. Further study and development of techniques and equipment may reduce or eliminate many of the limitations currently faced.

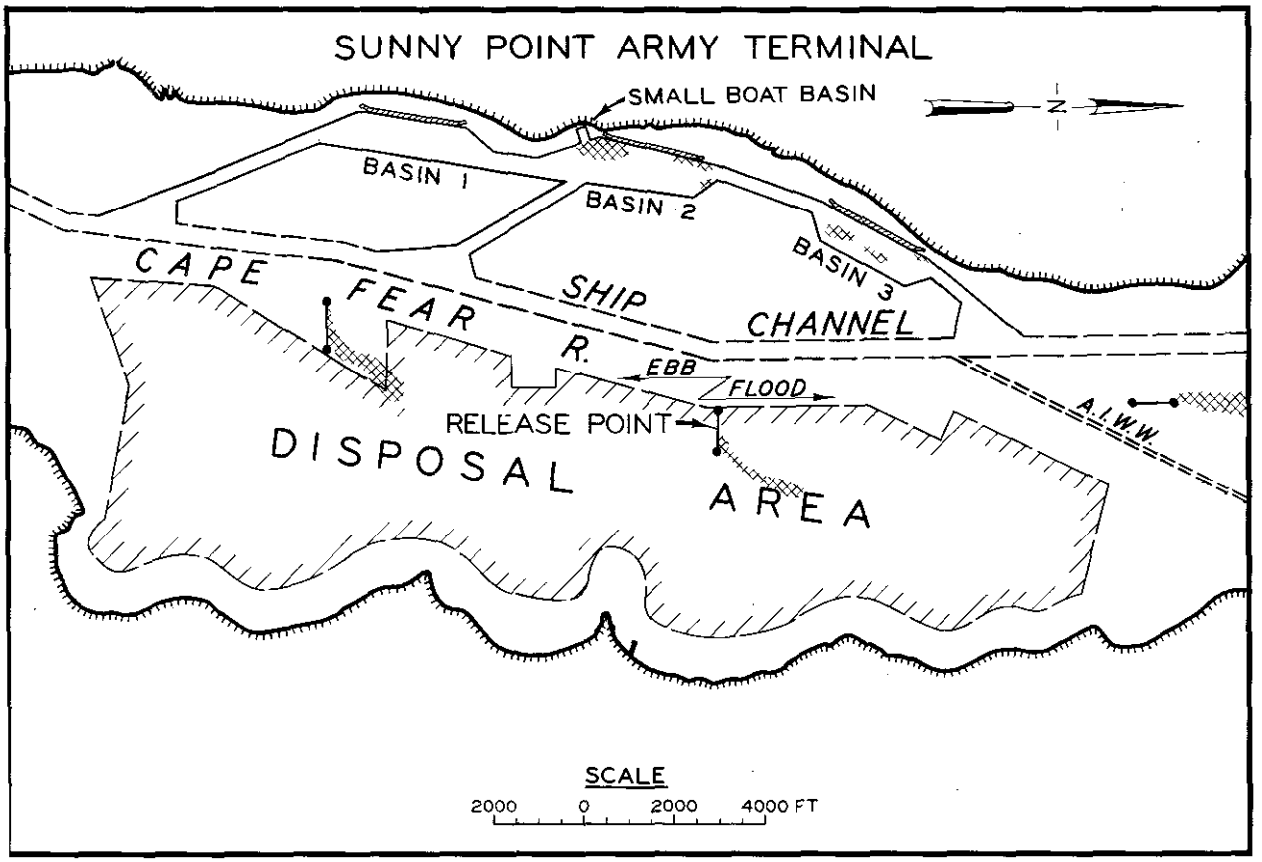


FIGURE 8. — Sunny Point Army Terminal tracer release points and deposition areas.

USE OF THE SWEDISH FOIL SAMPLER FOR TAKING UNDISTURBED CORES OF RIVER BOTTOM SEDIMENTS[1]

[Paper No. 63]

By R. J. PICKERING, *geologist, Water Resources Division, U.S. Geological Survey, Oak Ridge, Tenn.*

The release during the past 20 years of very small but measurable amounts of radionuclides into the Clinch River in eastern Tennessee by the Oak Ridge National Laboratory has created a unique opportunity for determining the distribution and behavior of fission products in a natural stream. A joint study of the fate of radionuclides released to the Clinch River has been carried on by the Water Resources Division of the U.S. Geological Survey and Oak Ridge National Laboratory since February, 1960, as part of the general program of the Clinch River Study Steering Committee.[2] Undisturbed core samples of river bottom sediments were required for phases of the joint study pertaining to the vertical distribution of fission products in the sediments.

Preliminary coring tests with several sampling tools showed that the relatively simple tube-type samplers were unsatisfactory for use in the Clinch River. With such devices there was preferential coring of certain sediment layers and excessive compaction of the sediments in the core sample. Driving of thin-walled tubes having inside diameters of 3 to 4 inches into the river sediments caused less compaction of samples than the driving of tubes with diameters of 1 to 2 inches,[3] but better control of compaction was necessary.

Further tests and inquiry resulted in the selection of the Swedish Foil Sampler, which takes cores that are slightly less than 3 inches in diameter, for sampling bottom sediments in the Clinch River. The Foil Sampler is a piston-type sampler in which thin axial metal strips (foils) are used to decrease friction between the sediment core and the sample tube while coring is proceeding. Cores as much as 14 feet in length were obtained in water as much as 45 feet deep with the Foil Sampler mounted on a barrel float equipped with a drilling tower. More than 100 undisturbed cores were collected at 14 cross sections on the Clinch River and at four cross sections on two tributary streams.

The loading of the sampler is illustrated in the series of photographs shown in figures 1 to 4. Steel "foils," fed from coils in the sampler magazine, are attached to the piston assembly (fig. 1). The piston assembly is then pushed up into the sample tube a short distance, pulling the foils down around the lip of the tube and up along its interior surface (fig. 2). After the

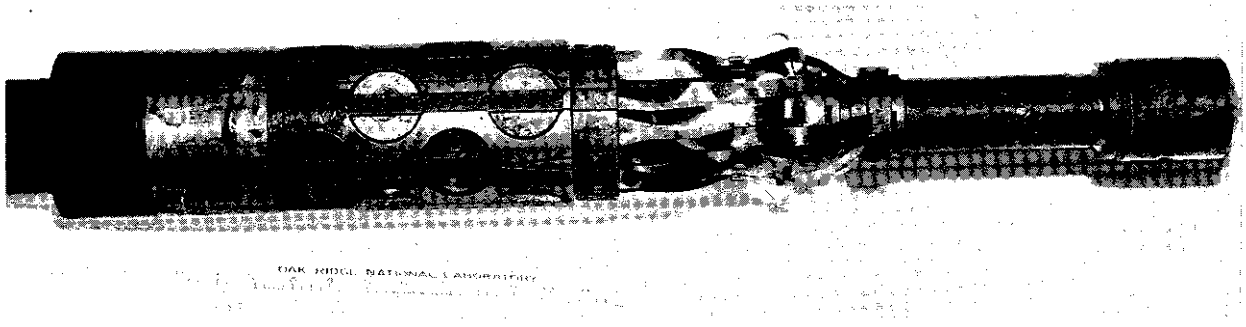


FIGURE 1. — Attachment of foils to piston assembly, Swedish Foil Sampler, showing sample magazine, foil fasteners and piston guide.

magazine cover, cutting shoe housing, and cutting shoe (fig. 3) have been attached and the piston guide has been locked into the base of the piston assembly, the Foil Sampler is ready for lowering (fig. 4).

The distance from the water surface to the sediment surface must be measured accurately with a weighted line before coring can proceed. The sampler is then lowered to within a few inches of the sediment surface, and the piston

[1] Work done at the Oak Ridge National Laboratory, operated by Union Carbide Corp. for the U.S. Atomic Energy Commission.

[2] The Clinch River Study Steering Committee is composed of representatives of the Oak Ridge National Laboratory and six State and Federal agencies that have an interest in the Clinch-Tennessee River system. It is responsible for the investigation of the distribution, behavior, and effects of radioactive materials in the river system.

[3] P. H. Carrigan, personal communication.

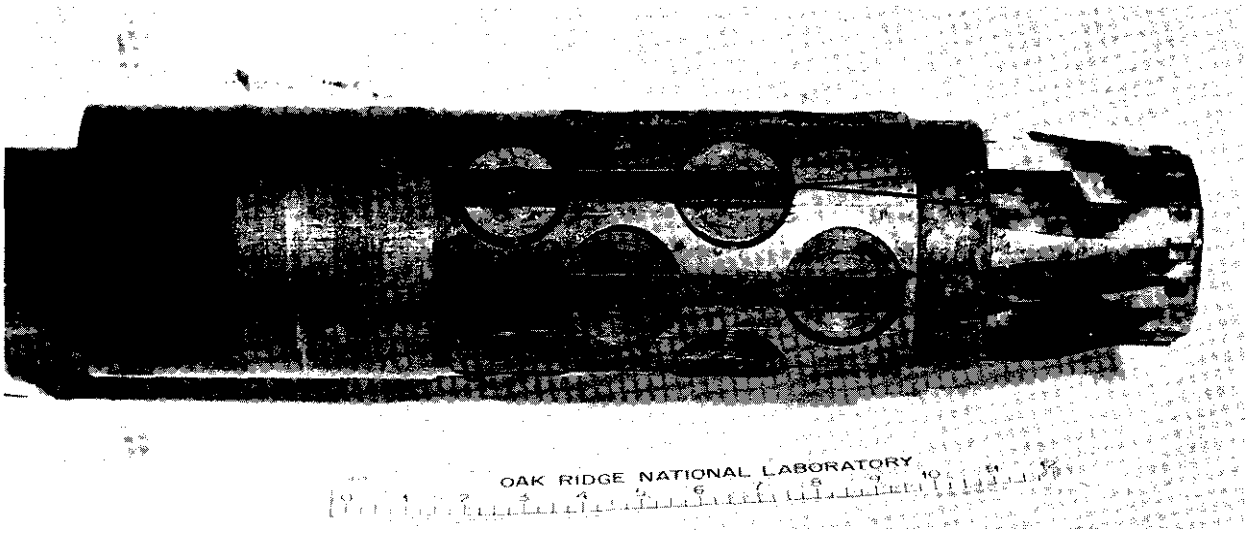


FIGURE 2. — Close-up of sampler magazine, Swedish Foil Sampler.

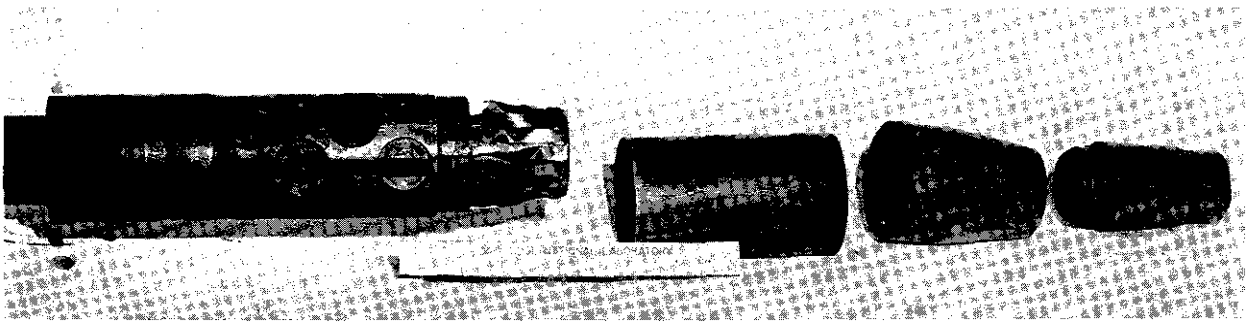


FIGURE 3. — Exploded view of sampler head, Swedish Foil Sampler. Left to right: sampler magazine, magazine cover, cutting shoe housing, cutting shoe.

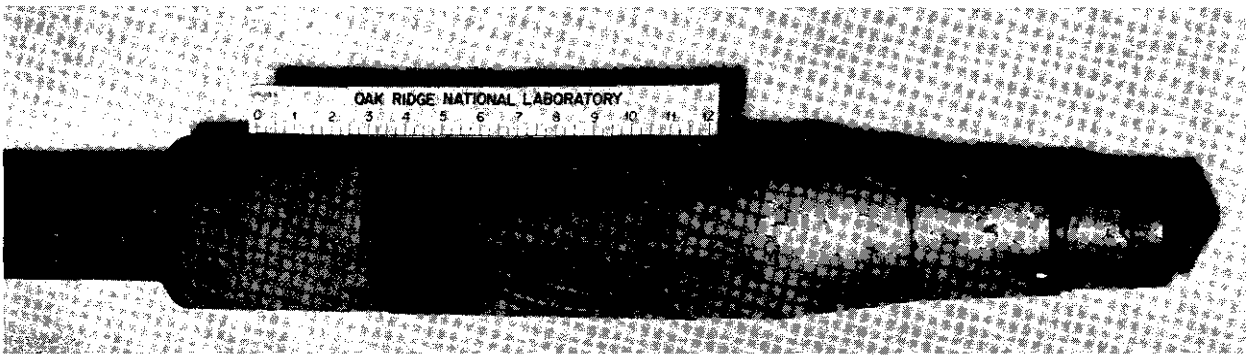


FIGURE 4. — Sampler head ready for sampling run, Swedish Foil Sampler.

chain is fixed to a hook at the top of the drilling tower. As the sampler is pushed into the sediment, the foils unroll and form a sheath that encloses the core and prevents its contact with the interior of the sample tube. The core does not move relative to the foil strips, and the only friction in the sample tube is the metal-to-metal friction between the foils and the interior of the tube. Compaction and blocking of the core in the tube due to friction between the sediment and the tube is thus prevented. During the insertion of the sample tube into the sediment, the piston and the core of sediment do not move from their original positions.

The primary function of the piston in a piston sampler is to prevent loss of core while the sampler is being raised to the surface. Because of the rigid sample tube and the tight-fitting piston immediately above the core, the hydrostatic force due to the column of water above the base of the sample tube can act on the core only in an upward direction, thus holding it in the tube. If the core begins to slide out of the tube, a partial vacuum is created between the top of the core and the piston, which is locked in position while the sample tube is being raised, thus increasing the differential upward push of hydrostatic pressure on the core and inhibiting its further movement. Adhesion of the core to the metal foils of the Swedish Foil Sampler also tends to hold the core in the tube.

Cores collected from the Clinch River were transferred from the metal sample tube of the Foil Sampler to a slightly larger plastic storage tube in the manner illustrated in figure 5. Because the plastic tube could be slipped over the sample tube, it was possible to transfer the core, still encased in its foils, without sliding it through the entire length of plastic tube. After the cores were transferred, they were refrigerated in order to suppress biological and chemical action.

Under certain sampling conditions, satisfactory cores could not be obtained with the unmodified sampler. Recovery of river sediment was excellent at points near the bank where the river sediment overlies soil, but in the deeper parts of the river channel where soft sediment lies directly on bedrock, the lower 6 to 18 inches of the sample were commonly lost. At some sampling sites, the driller was able to minimize core loss by disconnecting the piston chain while the sample tube was still in the sediment and then raising the sampler as quickly and as gently as possible, but this procedure was not always adequate. To decrease further core loss, a sampling shoe containing a basket-type core retainer was designed for the sampler and, upon request, was fabricated by Sprague and Henwood, Inc., the drilling contractor. The



FIGURE 5. — Transferring core from sample tube to plastic storage tube.

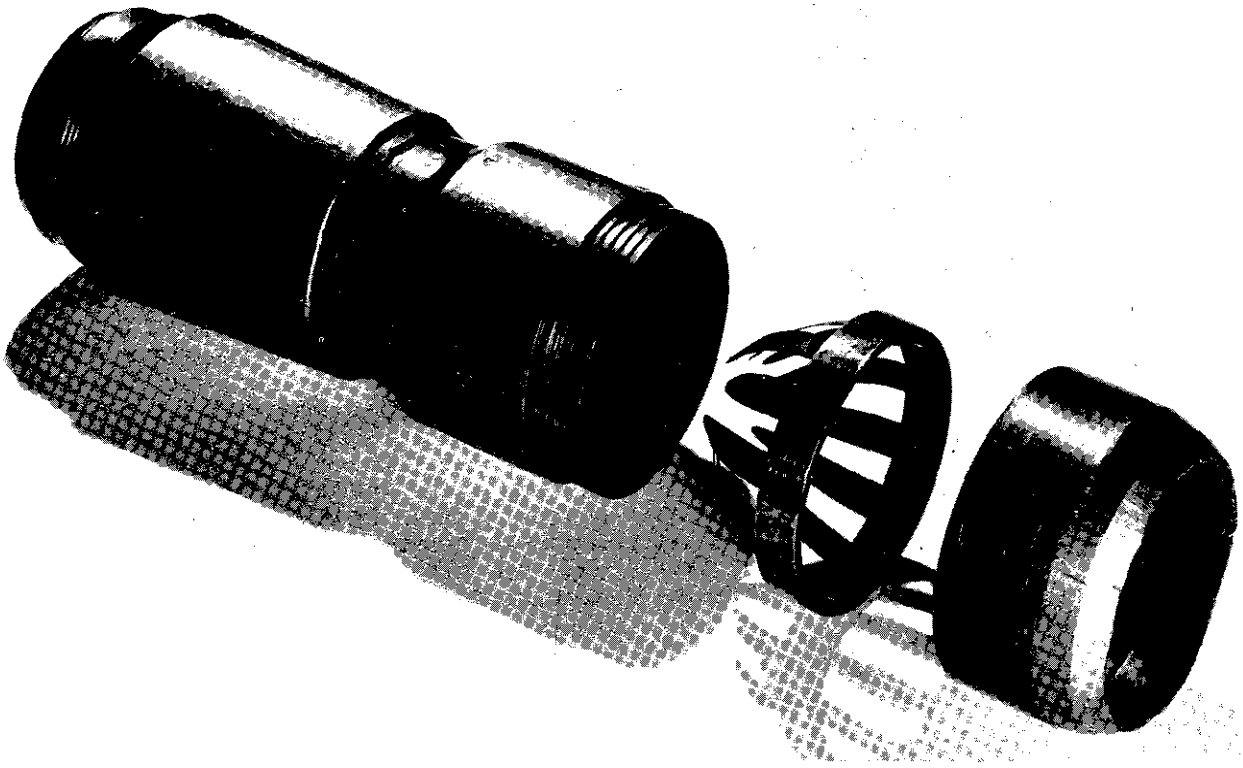


FIGURE 6. — Basket shoe assembly, Swedish Foil Sampler.

"basket," which consisted of curved, closely spaced, flexible spring steel fingers attached to a steel ring, was inserted near the base of the cutting shoe (fig. 6). Above the basket was fixed a thin plastic sleeve that was slit at the top so it would collapse over the basket when the sediment began to slide out of the shoe and thus partially, or at times completely, seal the opening at the base of the sample tube. Table 1

TABLE 1.—*Bottom sediment coring results, Clinch River, at mile 10.0*

Hole No.	Depth of water	Sediment penetrated	Percentage of core recovered	
			Without basket shoe	With basket shoe
	Ft. In.	*Ft. In.*		
1	8 9	6 9	75	.....
2	19 8	8 7	90	.....
3	22 5	3 8	59	.....
3	23 2	3 8	.....	80
4	16 6	8 4	96	.....
5	7 5	4 6	78	.....
6	8 7	2 4	0	.....
6	9 3	2 0	.....	92
7	8 4	8 9	79	.....
8	17 8	6 3	92	.....
9	27 9	2 9	0	.....
9	28 10	4 1	.....	78
0	4 9	5 9	93	.....

provides an example of the excellent results that were obtained with the basket shoe. One hundred percent recovery of a 10-inch-long core, composed entirely of sediment with a consistency like that of thick gravy, was obtained at one sampling site. The nearly watertight seal formed by the collapse of the plastic sleeve over the basket made such excellent recovery possible. The basket shown in figure 6, which has more widely spaced fingers made of heavier spring steel than the basket used in the fine sediment, was designed for recovering sediments that have a high sand content.

Particle-size distribution in the sediment is fairly constant throughout the portion of the Clinch River that contains radioactive sediment. The sediment is composed of approximately 15 to 20 percent clay, 25 to 30 percent sand, and 50 to 60 percent silt.

Core recovery at sampling points where radioactivity is believed to be present throughout the full thickness of the river sediment generally ranged from 80 to 100 percent (see table 1). Poor core recovery was accepted only where the sediment consists of nearly pure sand or of a mat of leaves and twigs overlying soft mud, but few such localities were encountered.

DISCUSSION — SYMPOSIUM 2

Sediment in Streams

Moderator: Thomas Maddock, Jr., USGS

Panel: H. A. Einstein, University of California
V. A. Vanoni, University of California
E. L. Pemberton, BR

MR. MADDOCK:

I've selected a panel to help on this problem and I think I have a real good one. The first one is Dr. Hans Albert Einstein, who, we might say, is Mr. Sediment Movement himself. I don't think anybody can work with the movement of sediment or solve any sediment problem without having been exposed to his work. The next one is Dr. Vito A. Vanoni, who I think is equally well known all over the country. Certainly my own association with him has extended over 26 or 27 years. We also have one of the younger men in the sediment field, Ernest Pemberton, who is assistant head of the Sedimentation Section in the Bureau of Reclamation in Denver.

The question I want to ask the panelists to discuss is this: The single thread holding together our model studies and our work in the field in trying to design stable channels, whatever it may be, is the relation between roughness and sediment load. We find that to get a stable channel these two things have to be compatible in some way. To start out the discussion on this question, I would like to ask each panelist, beginning with Dr. Einstein, to spend some time trying to bring out some of the elements that are to be found in this kind of a problem and what we have to do about them.

DR. EINSTEIN:

I hope I understand the question properly as to mean that we ought to answer whether both the friction factor and the sediment transport are related or unrelated. Is that about the meaning? In the design of any kind of a channel that carries sediment, there is such a tremendous combination and variety of various cases where once one of the variables and once another one of the variables is given and some of them are unknown, that it is almost impossible, in general, to answer this question. What I would like to say, and what I have been saying all along, was that there exists at least two relationships that govern all these flows and these are two simultaneous relationships such that, given a whole set of conditions, two of the variables can be determined and predicted for each case. Now these two may be the friction and the sediment load or, in other words, what I'm trying to say is that if all the other variables are given, then these two can be predicted and for that reason are naturally interrelated. I would say that the two are definitely interrelated in the sense the question was asked.

I would like to make one more remark and that is, looking around this room, I am somewhat reminded of the last meeting in Denver of this corresponding group, and I wouldn't want the moment to pass without having an occasion to express my satisfaction that today the design and field engineer contributes at least as much to the knowledge in our field as the laboratory does. At that time field and design engineers came to the meeting in order to learn what the relationships are, what they could calculate that they had been guessing at before. Today we're being taught which of our doings are usable and which are not. I think this progress is tremendous and I would like to congratulate you for that and let's give everybody who is involved in the field work and has contributed so much a good hand right now.

MR. MADDOCK:

Thank you very much Dr. Einstein. I would like to ask Dr. Vanoni the same question. I think from some of the discussions we've had that he may express this a little bit differently. This is a matter of semantics, and I think that if you make a question broad enough everybody will interpret it differently, and you will get a different answer. I hope this is the case as I ask Dr. Vanoni to talk about the same thing.

DR. VANONI:

Thank you, Tom. The question of the interrelations between the friction factor of an alluvial stream and the transport rate of sediment is one that has been discussed at length in the literature during the last 15 years. I think it is clear from laboratory studies that there is a relationship. For instance, we know that when a bed is covered with dunes, it is hydraulically rather rough, and the velocity and sediment transport rate over it tend to be relatively small. Now if we increase the velocity by increasing the discharge or slope, the dunes will tend to be modified so as to reduce the friction factor and increase the sediment transport rate. This is something we've learned from the laboratory.

Some river engineers knew about this relation quite a few years ago. I recall the work of Buckley, on the Nile River in 1922, in which he

plotted river stage, hydraulic radius, mean velocity, friction factor, and suspended load discharge against time for a whole year. It was clear from this that when the suspended load discharge increased, the water discharge also increased and that there was a tendency for the friction factor to decrease. This relation was rediscovered by many of us, including myself. It came as a discovery when we should have known it to start with.

The problem of predicting the friction factor is very closely linked with that of predicting the bed form. We know from laboratory studies that the presence of suspended load tends to decrease the friction factor, but we also know from laboratory experience that this is not the major effect in reducing the friction factor. The big reduction in friction factor results from changing the size and shape of dunes. Now I realize that a flume is not an accurate model of a river but, just the same, I think it is clear from observations that the same kind of changes in roughness occur in rivers as in flumes. So our problem is to predict for what conditions the dunes are going to start to wash away because this is when the biggest change in roughness is going to occur.

You have seen Dr. Simons wrestle with this problem this morning for a rather simple case of a flume with one material, and you can see that he got into some pretty complicated mathematics and relationships. I'm afraid we're not ready yet to predict the roughness relation on a river, because we really can't predict it in a flume. Here is where one of our big problems lies in predicting sediment load of a stream. One might say that this is the problem of working out the hydraulics of alluvial streams.

The trend in recent years to consider the stream as a whole rather than only the sediment discharge represents a move in the right direction. For several years many of us focused our attention on the suspended load or bedload and ignored the hydraulic factors of velocity and friction. I'm sure everybody didn't do this but many of us did. I think we are more realistic now that we are looking at the stream as a whole.

MR. MADDOCK:

Thanks very much, Vito. I'm going to call on Ernie Pemberton. Mr. Pemberton is one of those of many among us who has the problem of prediction, because that's what design is. You make a certain set of assumptions, and then you predict what's going to happen and you hope the verification confirms your decision. Now, inasmuch as Dr. Vanoni has indicated there may be some question as to whether you can do this or not, I want Mr. Pemberton to explain how he does it and some of the problems that he has encountered.

MR. PEMBERTON:

Thank you, Tom. I think from the question and the response today that there still remain many problems in predicting the sediment load of a stream. I consider it a privilege to be one of this distinguished group and will admit that I've had my feet wet with regards to the collection of field data.

In the collection of data, we have tried to analyze some of the variables that take place in sediment transport and to obtain a better knowledge of the existing conditions of rivers. I'd like to bring up one additional point on the same subject to which Dr. Einstein and Dr. Vanoni responded. That is, the bank friction factor as related to sediment transport. We have encountered many problems in the Bureau of Reclamation on sediment transport in canals or in rivers which involve channelization works where bank friction is an important factor. Dr. Einstein pointed this out to us a number of years ago, and I still don't believe we know how to handle the bank friction problem as related to sediment transport.

I would also like to comment on the application of material presented by Dr. Simons and Dr. Kennedy on sediment transport. We in the Bureau of Reclamation are continually working with problems of river channelization for channels that are aggrading or degrading. This involves trying to estimate the total sediment load in aggradation or degradation areas and requires a better knowledge of sediment transport in order to do something about the problem. Also, we need to know the total sediment transport of channels above reservoirs and diversion dams. At the diversion dam, we want to know how much of this sediment will be diverted and carried by the canal. I think the papers presented have given us more useful tools to aid in these problems. I'm not sure whether the solution to some of the problems as presented are applicable to other rivers. As an example, I'm sure many of you are aware of the problems involved in taking the Missouri River data and using them on a little stream out in Nebraska which was very dissimilar with regards to friction factors, the suspended sediment transport and total sediment transport. I hope this has answered the question.

MR. MADDOCK:

Well, frankly, I think you had an impossible question to start with, but I was wondering what you did about it and we did find out something. We found out something about what you were trying to do about it. We have about three more minutes before this meeting is supposed

to be devoted to the announcements. There were a large number of questions that came in that had to do with the details of the papers. I think that each and every one of us had questions about some part of some paper. I know I have questions about almost every one of them. Some things I disagreed with and on other things I would like additional explanation. It just doesn't seem possible that we could really accomplish very much if we took up these individual problems one at a time.

Pemberton brought up one just a moment ago, and that is how well can you apply what you have learned in the model to a natural stream. We can sit around and debate that one all afternoon. There are a number of the wonderful debatable problems and discussible problems in this particular field. Unless somebody has a question that he would like any of the people on this panel to answer (and I think that this is a real opportunity for asking questions you might not have another opportunity to get), I turn the program back to Don Bondurant.