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SYMPOSIUM 4

SEDIMENT TRANSPORT AND DEPOSITION

MODEL STUDY OF THE DRAG COEFFICIENT OF A STREAMBED PARTICLE

By Neil L. Coleman, Geologist, and Wilbert M. Ellis, Hydraulic Engineering Technician, USDA Sedimentation Laboratory, Oxford, Mississippi.

ABSTRACT

A time-mean drag coefficient for a streambed particle has been derived from considerations of flow-generated force moments and particle packing geometry. The drag coefficient is taken as a function of a particle Reynolds number and a velocity profile steepness parameter. These parameters are shown to be in fixed relation to each other because of the nature of the bounded shear flow near a streambed. The relation between these two parameters and the correlation of the drag coefficient with the relation between them has been demonstrated in experiments.

INTRODUCTION

Many streambeds consist of noncohesive particles ranging in size from boulders and cobbles to sand and silt particles. Design calculations for stable channels in natural materials, and for reliable riprap for constructed channel linings are sometimes based on theories about how force is applied to streambed particles by flow. A potentially useful method for exploring the validity of some of these ideas is by experimenting with a scaled-up model of a streambed segment with a particle resting on it. The advantage of such a model is that the flow field around the particle and the application of force to the particle can be observed in far greater detail than that possible in real streams or in conventional flume experiments. Early scaled-up streambed segment models were studied by Coleman (1967) and by Watters and Rao (1971).

This paper describes a model study of the time-mean drag coefficient for a spherical particle resting on a bed of similar particles, in a bounded shear flow characteristic of that found in streams.

SIMILITUDE PRINCIPLES

In Fig. 1 a spherical particle of diameter D rests on a bed of close-packed identical particles. The approach flow velocity profile $U(y)$ has its origin at a distance $D/5$ below the tops of the bed particles (Einstein and El-Samni, 1949). Because of the velocity profile, pressure and shear forces are exerted asymmetrically so that the drag component f_d acts along a line of action some distance, nL_3 , above or below 0. Particle motion tends to begin as rotation about the apparent support point P_2 under application of the moment:

$$M = f_d(L_3 + nL_3) \quad (1)$$

where $(L_3 + nL_3)$ is the length of the moment arm. The value of n is unknown, so it is advantageous (and permissible from elementary mechanics) to replace M with a moment:

$$M_o = M = f_o L_3 \quad (2)$$

where f_o is the apparent drag force component acting along a line of action through O . An apparent particle drag coefficient is then:

$$C_{do} = \frac{8f_o}{\pi D^2 \rho U_o^2} \quad (3)$$

where ρ is the fluid density, and U_o is a velocity acting, in the vector sense, along the same line of action as f_o . In the approach flow profile, U_o is the point velocity $U(y_o)$ at the elevation y_o of the particle center. Packing geometry for spherical particles indicates that:

$$y_o = \frac{2}{\sqrt{6}} - \frac{3}{10} D = 0.516D \quad (4)$$

A general equation (Scotton, 1967) for the velocity profile is:

$$\frac{U}{U_*} = F\left(\frac{U_* y}{\nu}\right) + \Omega\left(\frac{y}{\delta}\right) - G\left(\frac{U_* D}{\nu}\right) \quad (5)$$

where U_* is the bed shear velocity in the plane containing the profile origin, ν is the kinematic fluid velocity, and δ is the boundary layer thickness. The functions F and Ω are, respectively, the basic velocity profile function deduced from dimensional analysis (Schubauer and Tchen, 1961), and the wake component velocity augmentation function (Coles, 1956). The function G is the boundary roughness velocity reduction function (Schlichting, 1960). Introducing equation (4) into equation (5) expresses U_o nondimensionally as:

$$U_o^+ = \frac{U_o}{U_*} = F\left(0.516 \frac{U_* D}{\nu}\right) + \Omega\left(0.516 \frac{D}{\delta}\right) - G\left(\frac{U_* D}{\nu}\right) \quad (6)$$

which is a similitude parameter for the steepness of the approach flow velocity profile and for the asymmetry of the particle-proximate flow field in the presence of the profile. Introducing the ratio D/ν into equation (6) produces a conventional particle Reynolds number:

$$R_o = \frac{U_o D}{\nu} = F\left(0.516 \frac{U_* D}{\nu}\right) \frac{U_* D}{\nu} + \Omega\left(0.516 \frac{D}{\delta}\right) \frac{U_* D}{\nu} - G\left(\frac{U_* D}{\nu}\right) \left(\frac{U_* D}{\nu}\right) \quad (7)$$

which is the universal similitude parameter for the particle-proximate flow field.

Although U_o^+ and R_o are the two independent similitude parameters in the nondimensional statement of the streambed particle drag problem, they are not unrelated to each other. Experience with bounded shear flows (White, 1974) suggests that for a totally viscous flow the relation between the two parameters is:

$$U_o^+ = 0.718 R_o^{1/2} \quad (8)$$

while, for a totally turbulent flow the relation is:

$$U_o^+ = 3.29 + B + \Omega \left(0.516 \frac{D}{\delta} \right) \quad (9)$$

where B is a constant depending in value on the nature of the function G in equation (5). If a smooth transition exists between equations (8) and (9), then the incremental distance, dR_s , along the curve, U_o^+ , is everywhere:

$$dR_s = \sqrt{\left(dU_o^+ \right)^2 + \left(dR_o \right)^2} \quad (10)$$

and C_{do} must be a function of the line integral parameter:

$$R_s = \int_0^{R_o} \left[\left(\frac{dU_o^+}{dR_o} \right)^2 + 1 \right]^{1/2} dR_o \quad (11)$$

where R_s is a new kind of Reynolds number.

EXPERIMENTS

Experiments verified equation (8), defined equation (9), and demonstrated the existence of a smooth transition between the two functions. The experiments provided numerical values of C_{do} , U_o^+ , and R_o for correlation of C_{do} with R_s .

A simple streambed segment was simulated by installing, in a water tunnel, a bed of close-packed 1/2-inch spheres with a single sphere resting on the bed, as shown in Fig. 2. The isolated upper sphere was part of a drag force transducer, similar to that used by Coleman (1967), containing a strain gauge load beam for measuring f_o . A differential pressure transducer was connected to a small hole drilled in the front of the sphere, and referenced to a static pressure tap in the wall of the water tunnel. This system permitted simultaneous measurement of f_o and U_o . A second system, composed of a moveable total head tube, a second differential pressure transducer, and a second static pressure tap, was used to measure the velocity profile $U(y)$. The signals from the force and pressure transducers were recorded with a high-speed milliammeter recorder. Transducers were calibrated by the same methods used by Coleman (1967).

Operation of the water tunnel with Hydroxethylcellulose solutions, varying in kinematic viscosity from 1.29×10^{-2} to 1.04×10^{-5} ft²/s permitted a range of R_o from 0.463 to 13200 to be covered in the experiments. The kinematic viscosity of the working solutions was found by measuring the dynamic viscosity of each solution with a falling ball viscosimeter, and dividing this value by the fluid density obtained by weighing a known volume of the solution.

ANALYSIS AND RESULTS

Time-mean values of C_{do} and R_o were calculated from the f_o and U_o records using equations (3) and (7). The bed shear velocity, U_{*o} , was estimated from the measured profiles $U(y)$ by means of a cross-plot method (Liu, Kline, and Johnston, 1966) specially adapted to take into account the roughness of the model streambed. The profile parameter, U_o^+ , was then computed from U_o and U_{*o} , and plotted against R_o in Fig. 3.

Fig. 3 shows the verification of equation (8) and the evaluation of equation (9) as U_o^+ constant at 7.49. The latter result could be interpreted plausibly to mean that B has the value 4.2 and that the function Ω is negligible close to the bed. The dashed line in Fig. 3 demonstrates that a smooth transition function can be fitted between equations (8) and (9), so the line integral R_s can be defined by graphic differentiation of the curves in Fig. 3, and subsequent integration of the point derivative function so obtained.

Point derivatives for use in calculating R_s from equation (10) were found from

$$\frac{U_o^+}{R_o} \frac{d(\log U_o^+)}{d(\log R_o)} = \frac{dU_o^+}{dR_o} = \frac{U_o^+}{R_o} \tan \theta \quad (12)$$

where $(\tan \theta)$ is the slope at a given point $U_o^+(R_o)$ of the curves fitted to the data in Fig. 3. Although $(\tan \theta)$ from the smooth curves were used in making these calculations, actual measured values of U_o^+ and R_o were entered into equation (11) to preserve the scatter of the original experimental data. Values of the derivative dU_o^+/dR_o occurring in the experiments ranged from 0.485 at $(R_o = 0.463)$ to a maximum of 9.18 at $(R_o = 93.1)$, hence dropping rapidly to zero at $(R_o = 400)$.

The line derivative function dR/dR_o calculated from equation (10) and the point derivative function is shown in Fig. 4. This function is characterized by a gradual transition away from the viscous flow condition until the inflection point of the function ($d^2R/dR_o^2 = 0$) is reached at R_o equal to about 150. A sharp drop then ensues, with dR/dR_o reaching a value of 1 at R_o of about 400. This marks the establishment of complete turbulence in the flow approaching the particle.

Calculation of R_s was accomplished by integration from $(R_o = 0)$ to $(R_o = 0.463)$ using equation (8) to obtain the set point value of 0.689 shown in Fig. 4. Subsequent integration of the function defined by the data was by summation of incremental areas from one data point to the next, the points having first been ordered in R_o . Like the technique used in obtaining dU_o^+/dR_o this integration method preserves the original experimental scatter.

In Fig. 5, C_{do} is plotted against R_s . A tentative trend curve is fitted to the experimental data points. In the region of totally vis-

cous approach flow, the trend evidently approaches a function:

$$C_{do} = \frac{40}{R_s} \quad (13)$$

as a streambed equivalent of the Stokes range for a sphere falling in an infinite quiescent fluid. The gradual deviation away from this function, which is characteristic of the gradual increase of influence of separation and form drag at the expense of viscous drag, is punctuated by a marked perturbation in the trend, which corresponds exactly with the transition of the approach flow from viscous to fully turbulent. After onset of complete approach flow turbulence at ($R_s = 400$), C_{do} tends toward a constant value of about 0.7, within the range of experimental data presented here.

CONCLUSIONS

A fixed relationship exists between U_o and U_* for a particle resting on a streambed. The nature of this relationship agreed with what would be expected from the known properties of bounded shear flows.

The Reynolds number for a streambed particle can be defined in an integral form R_s which provides for similitude of the steepness of the approach flow velocity gradient, as well as for the similitude of the particle-proximate flow field.

The relation between C_{do} and R_s displays a trend toward a Stokes law type function (but with a different numerical constant) in the region of viscous approach flow. The gradual trend away from the viscous flow function as pressure forces begin to predominate is ended by an abrupt perturbation of the data trend which corresponds exactly with the transition from viscous to turbulent approach flow. In fully turbulent approach flow, C_{do} tends to be constant within the range of data obtained in this study.

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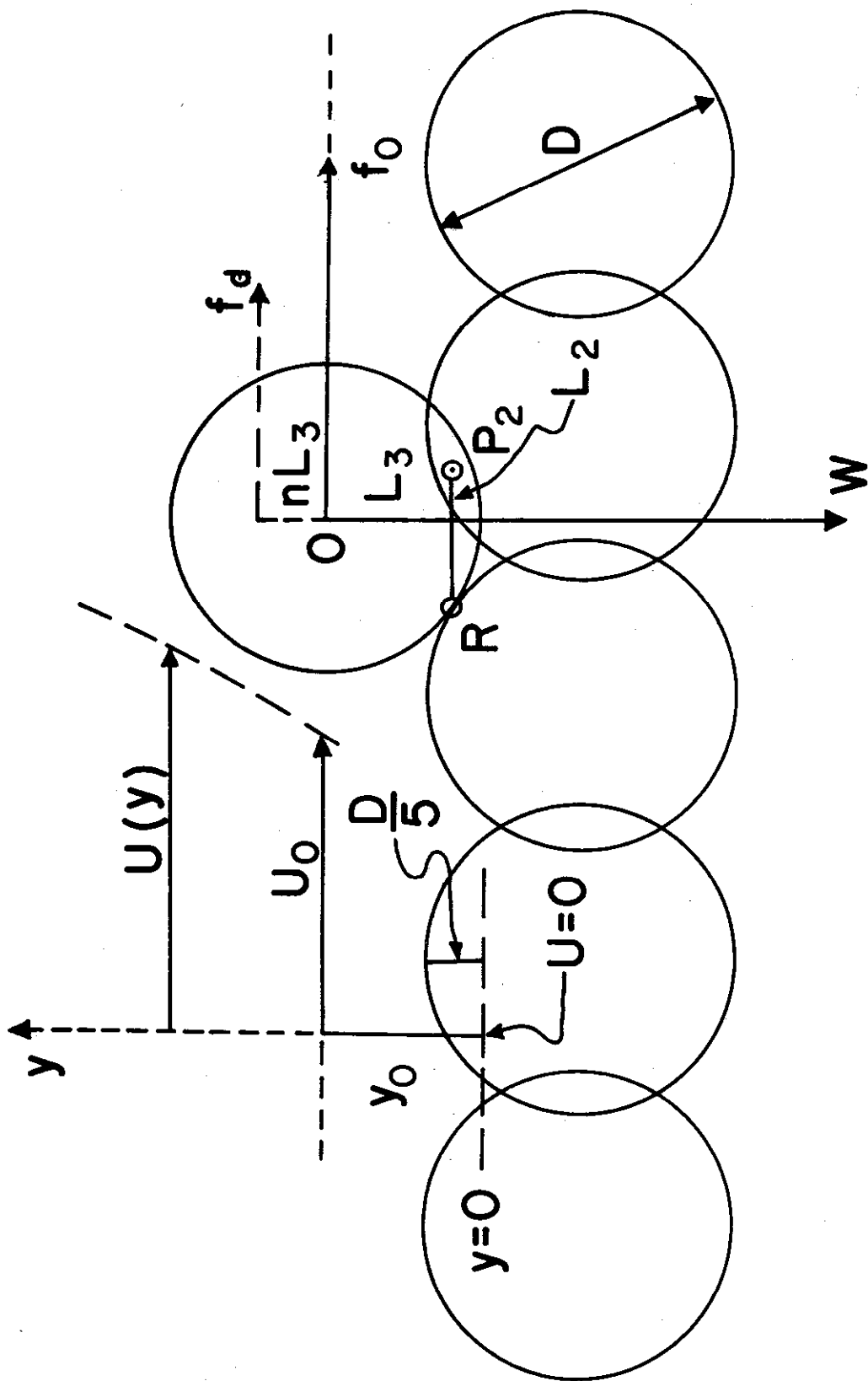


Fig. 1 Definition sketch for a particle on a streambed.



Fig. 2 Installation of the model particle in the water tunnel.

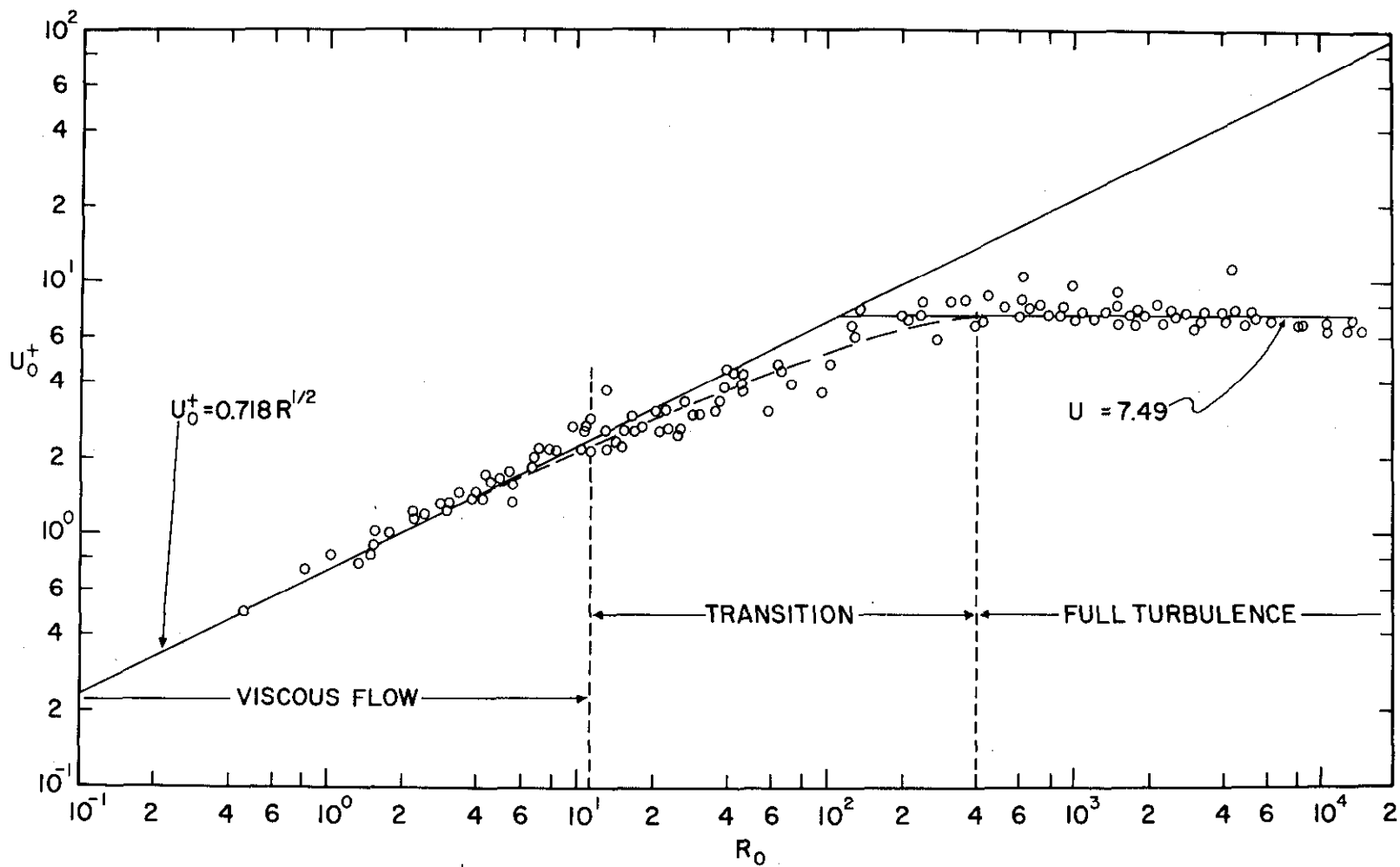


Fig. 3 The relation of U_0^+ to R_0

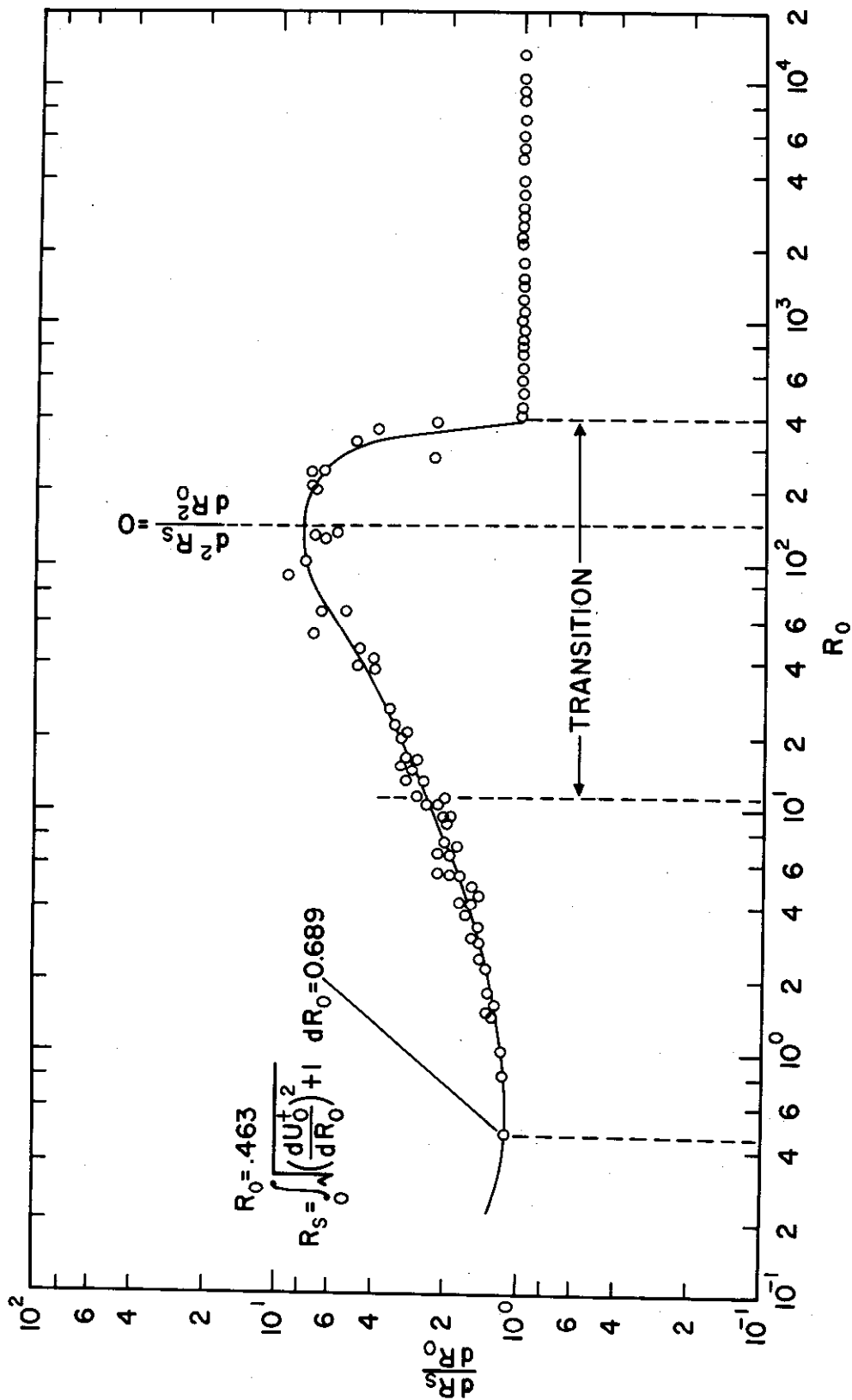


Fig. 4 The line derivative function.

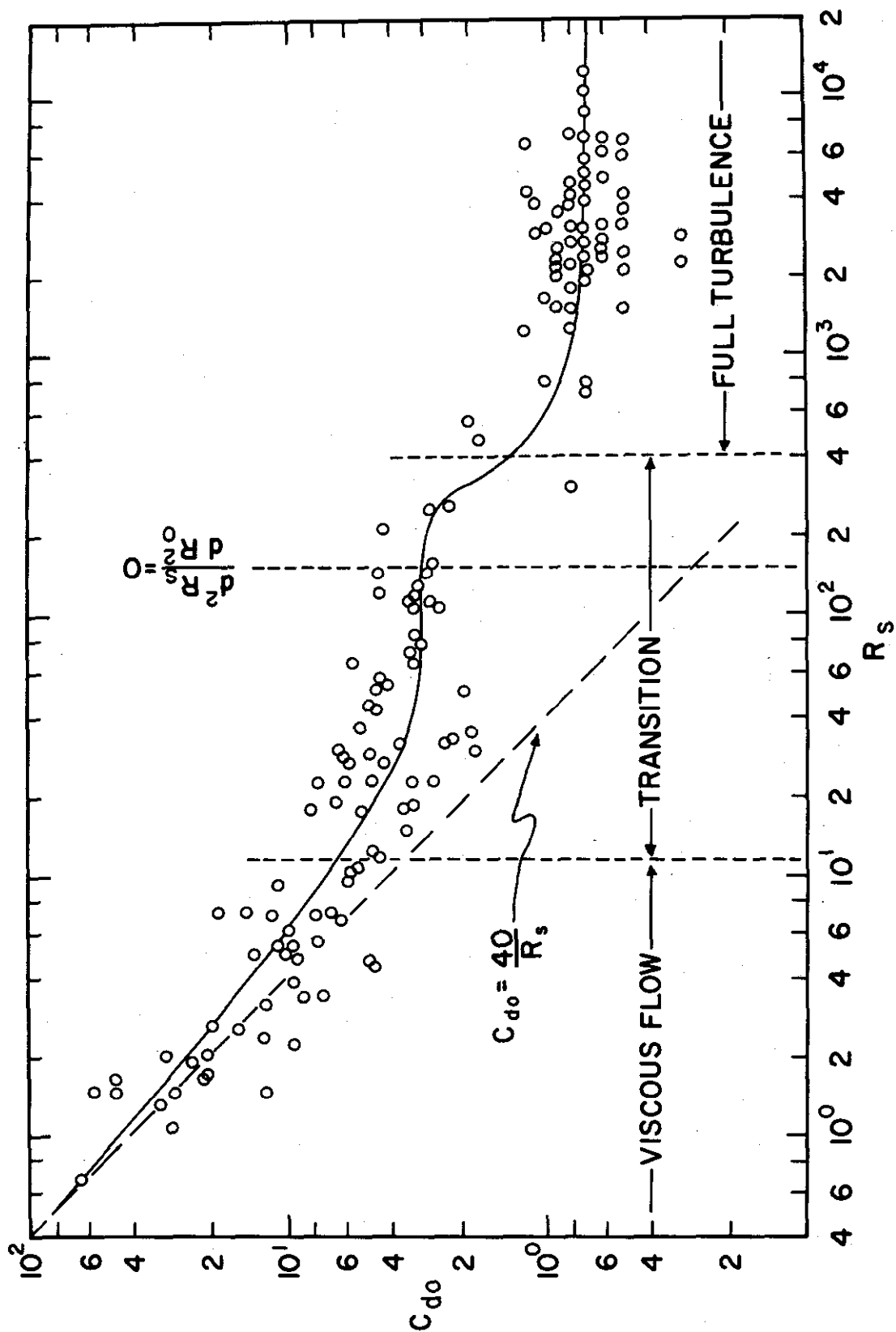


Fig. 5 The relation of C_{d0} to R_s

SEDIMENT TRANSPORT CHARACTERISTICS
OF THE UPPER SAN FRANCISCO
BAY-DELTA ESTUARY¹

by Matt Rumboltz², James F. Arthur³, and Melvin D. Ball⁴

ABSTRACT

Sediment transport in the San Francisco Bay-Delta Estuary of California is being investigated by an Interagency, Federal and State study group as part of an overall investigation to assess the impact of water resources development on the ecology of the Estuary.

Analysis of the river discharge and suspended solids data (1960-1969) indicates that over 88 percent of the sediment discharge by the rivers to the Delta originates in the Sacramento River basin. The analysis also indicates that during the November through April period, 80 percent of the annual sediment discharge from the rivers enter the Delta. Suspended solids transport through the Estuary is primarily controlled by river inflow, export pumping, tidal exchange, two-layered flow estuarine circulation, and surface wave action.

This paper presents a brief summary of sediment discharge to the Bay-Delta Estuary and a general description of the factors influencing transport of sediments through the Estuary.

INTRODUCTION

The San Francisco Bay-Delta Estuary of California is among the most complex in the World. Numerous studies describing this complexity have been compiled in several reports (California Department of Fish and Game, 1972; U.S. Army Corps of Engineers, 1975; Kaiser Engineers, 1969; California State Water Resources Control Board, 1975; and, California Department of Water Resources, 1974).

In 1970, an interagency study group consisting of the U.S. Department of Interior's Bureau of Reclamation and Fish and Wildlife Service and the California Resources Agency's Departments of Fish and Game and Water Resources was established to conduct a comprehensive investigation of the effect proposed Federal and State water projects will have on the Estuary.

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A major aspect of these studies is the prediction of future turbidity conditions in the Estuary resulting from proposed increased water diversion by the Federal and State water projects. The prediction of future turbidity is one of the major criteria required to assess the project's impact on the estuarine environment. Turbidity is thought to be the factor presently limiting algal growth to less than detrimental levels in the Delta (the portion of the Estuary east of Pittsburg as shown in figure 1) through Suisun Bay. Krone (1966 and 1974) states that present suspended material loads entering the Estuary will decrease as water exports are increased in the future. He suggests that this decrease of suspended load will result in decreased turbidity followed by a major increase in phytoplankton growth and possible detrimental dissolved oxygen depletions. Results of mathematical biological modeling for the Suisun-Bay-Western Delta by the Inter-agency study group (Hydroscience, 1975) also suggest increased phytoplankton levels if the turbidity levels decrease and discharges of nutrients increase.

The interagency phytoplankton model is presently capable of duplicating historical levels of algal growth. However, at the present time, predictions of biological growth under future conditions is limited since knowledge of sediment transport through the Delta, the factors influencing its distribution, and the future suspended material input to the Estuary are not fully understood.

DISCUSSION

Sources of sediment to the upper San Francisco-Bay-Delta Estuary include: riverborne inflow; municipal, industrial, and agricultural discharges; windblown materials; resuspended sediments; and, oceanic inflow. Sediments are lost from the Estuary in several ways including: discharge to the ocean, shoaling, local consumptive use; and export from the Delta via Federal, State, and local water projects.

This paper discusses transport characteristics of suspended sediment in the upper San Francisco Bay-Delta system east of San Pablo Bay. The transport is primarily controlled by: (1) river inflow, (2) export pumping, (3) tidal exchange, (4) two-layered flow estuarine circulation, and (5) surface wave action.

RIVER INFLOW

There are two major rivers entering the upper Bay Delta Estuary, the Sacramento and San Joaquin (figure 1). The Sacramento River drains the large basin north of the Delta while the San Joaquin drains the basin south of the Delta. Together these two river systems account for about 42 percent of the estimated natural runoff in the entire state of California. The Sacramento River including the Yolo Bypass (flood overflow channel for the Sacramento River) is by far the largest of the two, accounting for about 80 percent of the total inflow to the Delta. The San Joaquin Valley contributes 15 percent and the Delta east side streams and rivers account for the remaining 5 percent (Department of Water Resources, 1974).

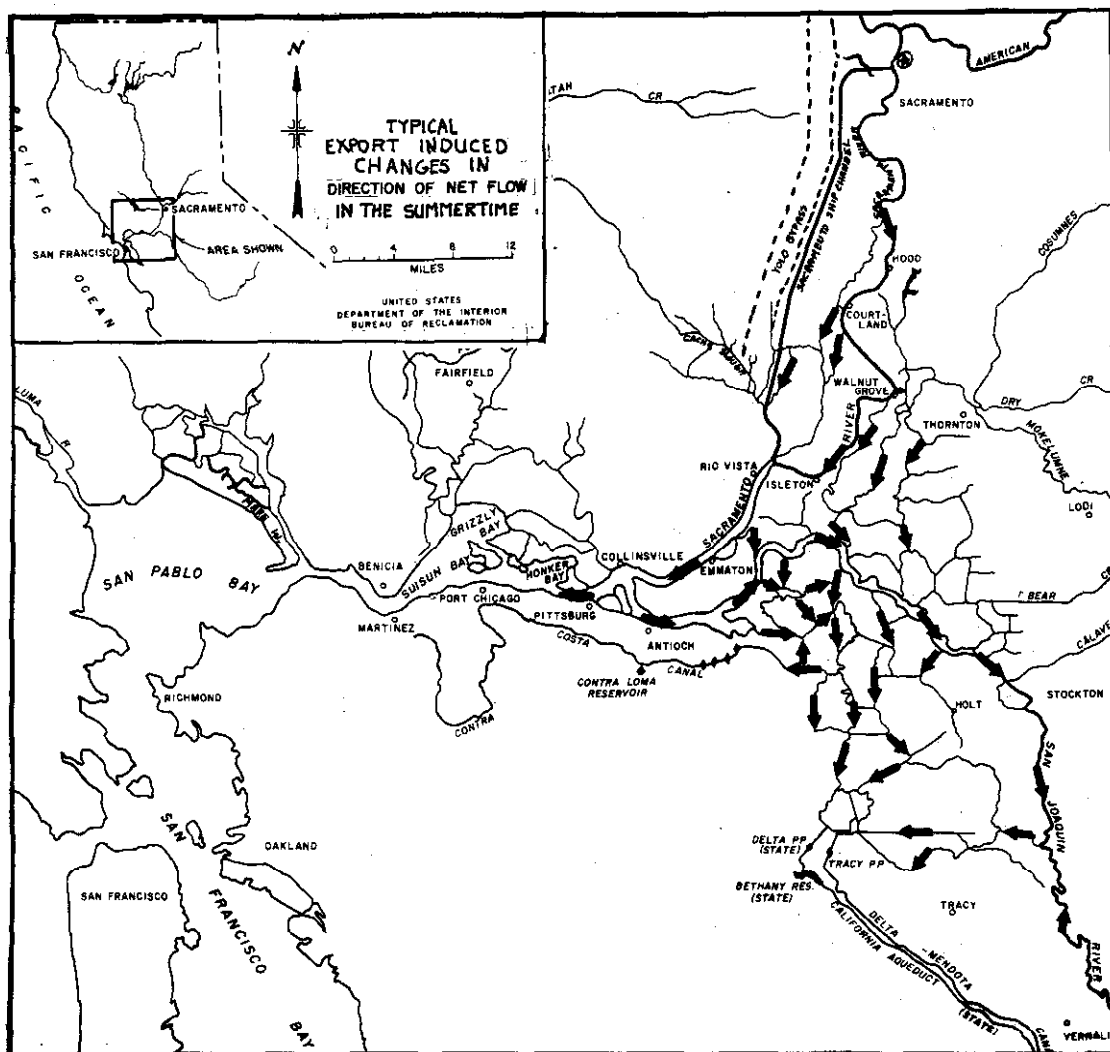
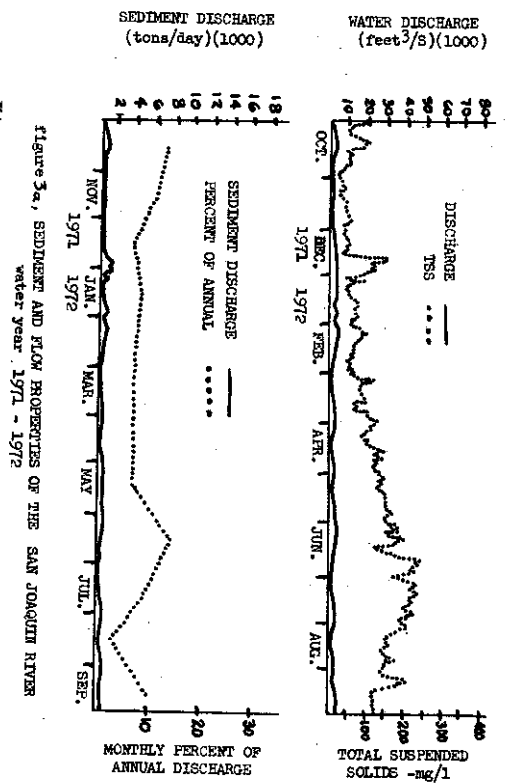
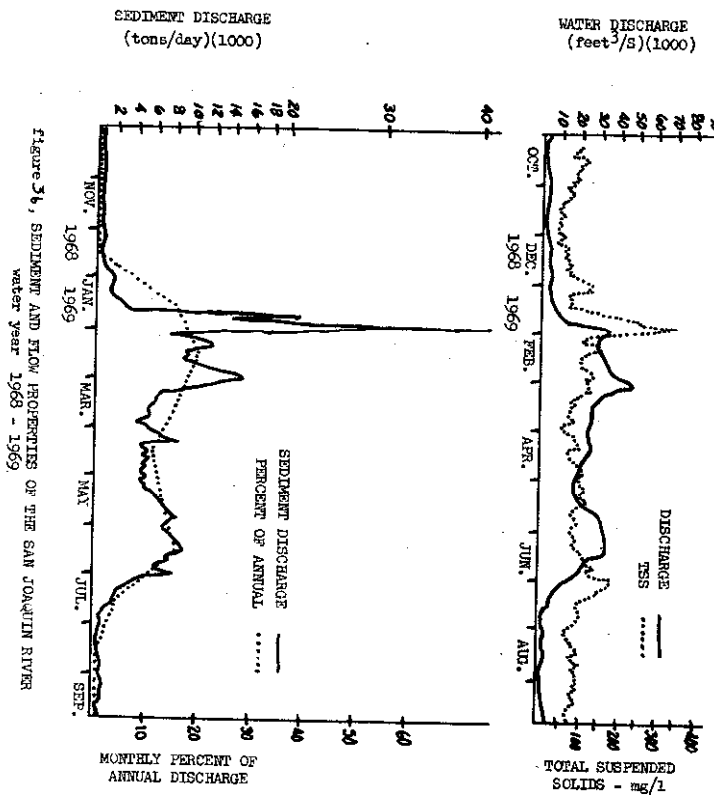
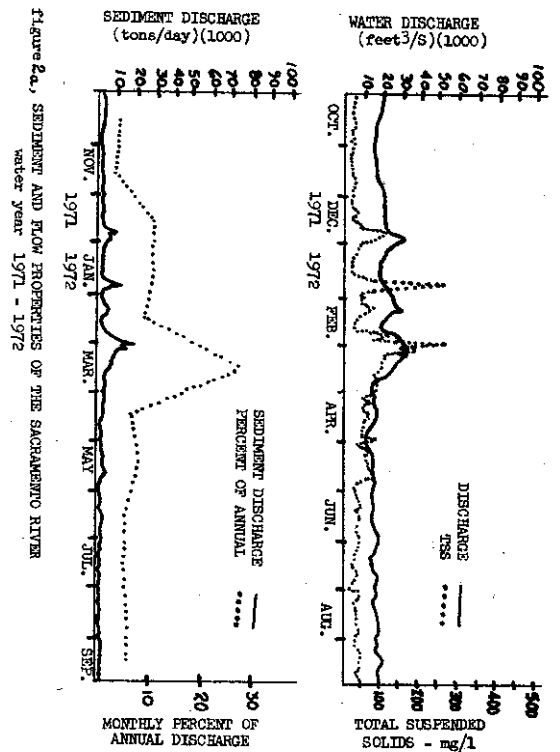
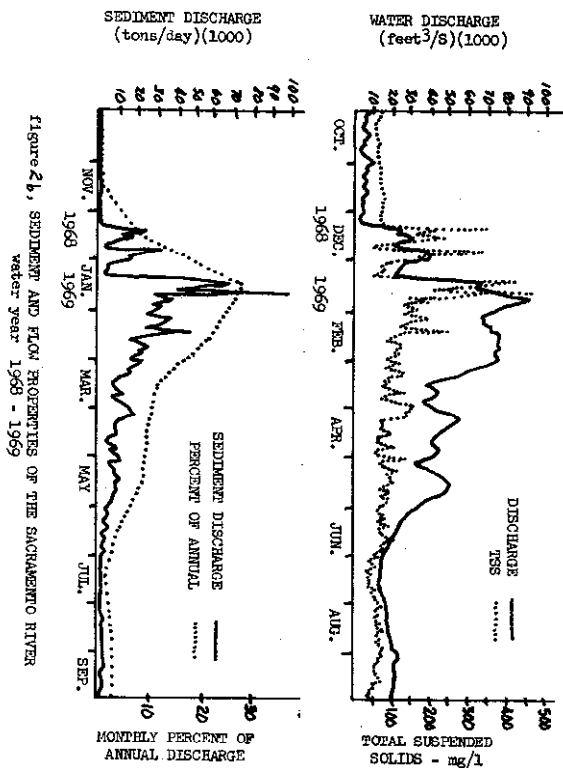


Figure 1. Study Area Including Typical Flow Reversals Induced by High Level Export Pumping and Low San Joaquin Riverflow.

During periods of exceptionally high river inflow, fresh river water flows over the surface of the Bay and out under the Golden Gate Bridge while during lower flows, the fresh water is dispersed and mixed into the bay waters to be removed by tidal exchange to the ocean. The amount of fresh water discharged from the rivers also effects a suspended solids entrapment area in the Bay.

Sacramento River (not including the Yolo Bypass) - This river is characterized by high unregulated flows occurring in the winter months of November to April. Changes in flow and sediment load between two recent low and high flow years are illustrated in figure 2. Daily flow rates measured at Sacramento since 1948 as recorded in the 1972 USGS report of the "Water Resources Data for California" have ranged from $158 \text{ m}^3/\text{s}$ ($5,590 \text{ ft}^3/\text{s}$) to $2,940 \text{ m}^3/\text{s}$ ($104,000 \text{ ft}^3/\text{s}$) with an average discharge of $668 \text{ m}^3/\text{s}$ ($23,660 \text{ ft}^3/\text{s}$). Highest suspended sediment concentrations also



peak between November and April but seem associated more with the initial storm runoff rather than the subsequent high flows developed by the individual storm period. Daily sediment concentrations collected at Sacramento since 1956 as reported in the 1969 USGS report of the "Quality of Surface Water of the United States" have ranged from 11 mg/l to 1,960 mg/l while the daily sediment discharge has range from 181,000 kg (200 tons) to 476,000,000 kg (525,000 tons). Sediment discharge best follows the pattern of the sediment concentration rather than the flow rate (figure 2). During the 1960-69 period, 82 percent of the sediment discharge for the river entered the Bay-Delta system from November through April (table 1). Organic suspended solids of the total suspended solids averaged 14 percent for the 1969-74 period (USBR data collected at Hood).

San Joaquin River - This river is characterized by high flows in the December to July period and low near constant flows in the rest of the year (figure 3). Since the 1970-71 water year, there has been little difference between the high and low flow periods as compared to the 1968-69 water year. Daily flow rates measured near Vernalis since 1948 have ranged from 0.54 m³/S (19 ft³/S) to 2,230 m³/S (79,000 ft³/S) (USGS, 1972). The 44-year average (1924, 1929-72) is 126 m³/S (4,464 ft³/S). Suspended sediment concentrations peaked sharply following heavy precipitation, as with the Sacramento River. Sediment concentrations increased gradually between the spring and fall and generally peaked in July (figure 3). The 1969-74 percentage of organic suspended solids in the total suspended solids is around 16 percent and remains near this amount year around (USBR data collected at Vernalis).

Daily sediment concentrations since 1956 have ranged from 9 mg/l to 1,590 mg/l while the daily sediment discharge has ranged from 1,810 kg (2 tons) to 49,100,000 kg (54,100 tons) (USGS 1969). Sediment discharge, at times followed both the pattern of sediment concentration and flow rate; however, neither displayed a dominate correlation. During the 1960-69 period, 62 percent of the sediment discharge for the river entered the Bay-Delta system from November through April (table 1). The higher percent summer sediment discharge in contrast to the Sacramento River is attributed to the high composition of agricultural return flows in the San Joaquin River.

Yolo Bypass - The Yolo Bypass was constructed primarily to contain flood waters in the Sacramento River Basin. Under normal operations, when the flow in the Sacramento River reaches about 1,400 m³/S (50,000 ft³/S), water starts overflowing into the Bypass approximately 32 km (20 miles) upstream from Sacramento. The daily Bypass flows during November to June can be quite large, and since 1939 have ranged from no flow to 7,700 m³/S (272,000 ft³/S) (USGS, 1972). Flows from the Bypass return to the Sacramento River system in the Cache Slough drainage near Rio Vista.

Suspended Sediment Discharge - The suspended sediment load carried by the Sacramento River (excluding the Yolo Bypass) was 88 percent of the combined annual sediment discharge of the Sacramento and San Joaquin Rivers to the Delta (table 1). During the November through April period, 80 percent of the annual sediment discharge from these two rivers entered the Delta with 91 percent of this amount being contributed by the Sacramento River. These

TABLE 1

YEAR	S E D I M E N T D I S C H A R G E		
	November through April		Annual
	Tons*	% of Annual	(Water Year) (Oct-Oct)
<u>Sacramento River (Less the Yolo Bypass)</u>			
1969	2,764,559	80	3,454,009
1968	1,332,760	83	1,602,088
1967	2,490,746	75	3,311,776
1966	1,776,360	86	2,064,670
1965	5,203,037	92	5,684,148
1964	807,978	76	1,069,009
1963	2,796,222	71	3,946,188
1962	1,749,227	87	2,006,347
1961	1,684,828	87	1,943,177
1960	<u>1,518,761</u>	<u>86</u>	<u>1,764,588</u>
*TOTAL	22,124,478	82	26,846,000
<u>San Joaquin River</u>			
1969	943,888	63	1,494,516
1968	77,208	64	120,402
1967	230,311	45	515,577
1966	137,698	74	186,607
1965	434,507	78	555,112
1964	60,298	60	99,991
1963	177,501	51	344,823
1962	166,122	64	258,267
1961	14,834	63	23,532
1960	<u>30,070</u>	<u>66</u>	<u>45,608</u>
*TOTAL	2,272,437	62	3,644,430
GRAND TOTAL	24,396,915	80	30,490,430
*1 ton = 907.2 kg			

These results indicate 73 percent of the annual sediment discharge of the two rivers enters the Delta during the November through April period from the Sacramento River.

Other sources of sediment to the Bay Delta include local municipal and industrial contributions. The California State Water Resources Control Board (1975) have published some estimates for the central and eastern Delta and the engineering firms of Little, et. al (1975) have published some preliminary estimates for the Suisun Bay western Delta. The values in table 2 are calculations of the data published in these two references.

Table 2

Industrial Total Suspended Solids Discharge Tons* per Day	Municipal	Location
23.4	11.9	Suisun Bay Western Delta
<u>25.8</u>	<u>7.3</u>	Central & Eastern Delta
49.2	19.2	Total
68.4		Grand Total

68.4 tons per day = 25,000 tons annually

*1 ton = 907.2 kg

The 25,000 tons annual sediment discharge from some of the major M&I sources is about 0.7 percent of the annual input from the two rivers (excluding the Yolo Bypass).

Levels of windblown sediment entering the Estuary and amounts of within-Delta agricultural-return-flow sediment discharges have not been determined.

EXPORT PUMPING

Water is exported from the Delta at three major facilities: the Delta Mendota Canal, the California Aqueduct, and the Contra Costa Canal (Figure 1). The Contra Costa Canal exports water from the central Delta while the other two export water from the southern Delta. Export pumping induces significant changes in the flow patterns in the Delta. Without export, the net flow from the two main rivers entering the Delta would be to the west. During high level export pumping and low San Joaquin River inflow, the flow from the Sacramento River is drawn south across the Delta and the San Joaquin is drawn along the southern edge of the Delta to the export pumps. This results in net flow reversals in many of the Delta channels (figure 1).

The transfer of Sacramento River water across the central Delta causes a distinct effect on the sediment concentrations in the San Joaquin River portion of the Delta. The concentrations of suspended solids in the San Joaquin River, in the area where water from the Sacramento River flows across the Delta, is 2 to 3 times less than either upstream or downstream. From March to October, the suspended sediment concentration in the San Joaquin at Vernalis typically ranged from 75 to 200 mg/l while in the Sacramento ranged from 50 to 75 mg/l. The cross Delta flow of the Sacramento River water explains the reduced suspended sediment concentration in the central Delta. In the western Delta the increased suspended sediment concentrations are caused by resuspension and entrapment which will be discussed in later sections of this report.

Delta water export by the Delta Mendota Canal (DMC) has increased from 2,035,000,000 m³ (1,650,000 acre-feet) in 1966 to 2,685,000,000 m³ (2,177,000 acre-feet) in 1974; and, export by the California Aqueduct has increased from 1,122,000,000 m³ (910,000 acre-feet) in 1968 (the

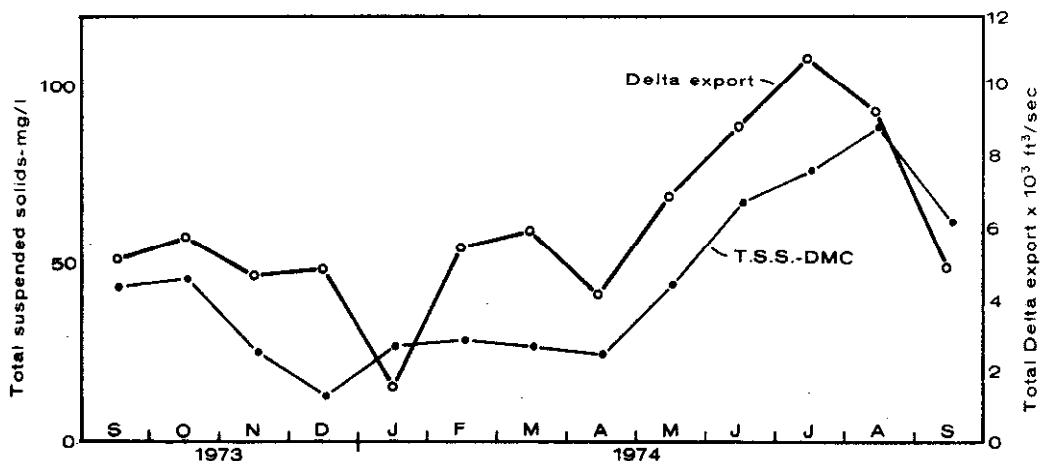


FIGURE 4—Comparison of average total delta export and average total suspended solids entering the Delta-Mendota Canal (USBR data)

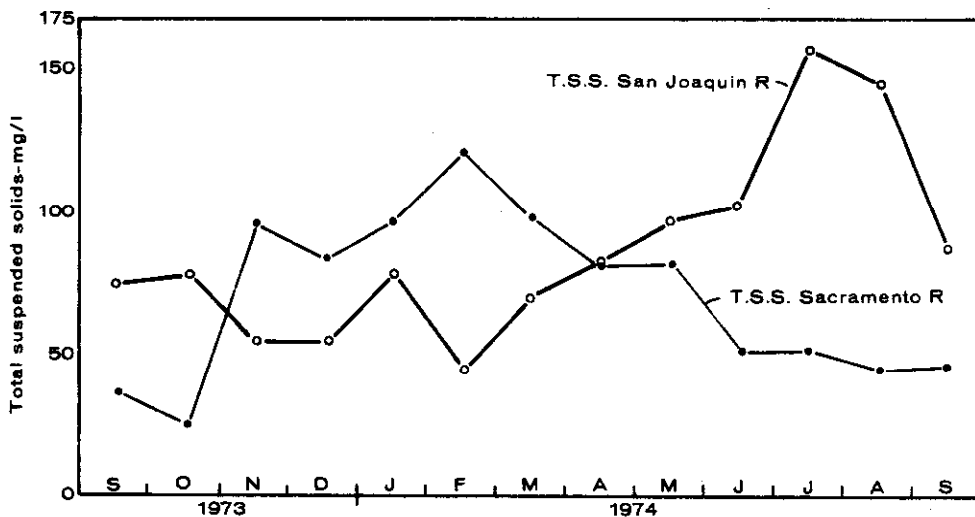


FIGURE 5—Average monthly total suspended solids in the San Joaquin River at Vernalis and Sacramento River at Sacramento (USGS data)

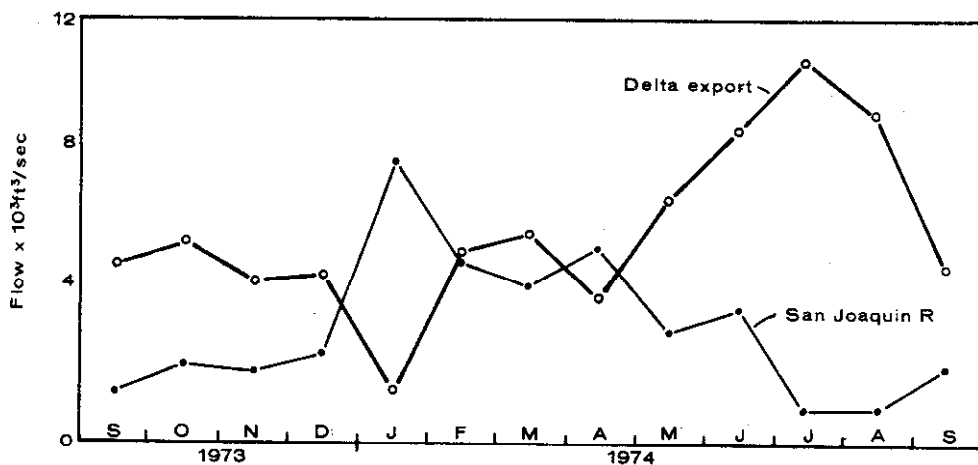


FIGURE 6—Comparison of average total delta export and average San Joaquin River flow at Vernalis (difference between export and river flow made up by Sacramento River flow)

first year of operation) to 2,085,000,000 m³ (1,860,000 acre-feet) in 1974. The increased export has resulted in increased sedimentation problems in the Delta Mendota Canal, particularly as a result of its intake structure. The increased sedimentation has required increased sediment removal from an average of 10,700 m³/yr (14,000 yd³/yr) during the 1956-1966 period to 15,300 m³/yr (20,000 yd³/yr) presently (Arthur and Cederquist, 1975). The increase in sedimentation rate is proportional to the increase in export pumping.

Measurements of suspended sediment entering the DMC (September 1973 to September 1974) indicated that the concentration of solids were low during the winter when export was low, and high during the summer when export was high (figure 4).

Seasonal variation in suspended solids concentration in the DMC water (figures 4 and 5) was similar to the variation found in the San Joaquin River, but varied inversely with concentration in the Sacramento River. This would seem to indicate that the major source of suspended solids load in the DMC was from the San Joaquin River. However, during periods of high Delta export, flow in the San Joaquin River was only 10 to 20 percent of total export and the remaining 80 to 90 percent of the export flow was supplied by cross Delta flow from the Sacramento River (figure 6).

A proposed facility (the Peripheral Canal) is envisioned to convey water from the Sacramento River at the northern end of the Delta to the export pumping facilities rather than pumping from the southern Delta. It will also release water at several locations into the Delta on its eastern periphery. If constructed, the less turbid Sacramento River water will not flow across the Delta and the more turbid San Joaquin water will flow through the Delta rather than being exported. The high suspended sediment loads during the winter will still be discharged from both rivers into the Delta; however, the March through October sediment transport characteristics will change. The assumption is made that the central and southern Delta will become more turbid and that the northern Delta will remain much the same. There are differences of opinions as to what changes will occur downstream in the western Delta-Suisun Bay where salinity stratification occurs.

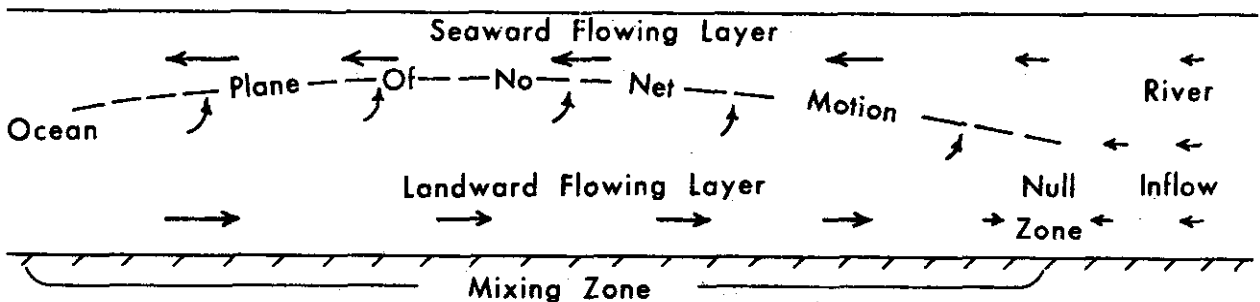
TIDAL EXCHANGE

The major sediment transport mechanism in the Bay-Delta Estuary is tidal exchange. Except during rare floods when the volume of fresh water is so great that it flows over the more saline bay water suspended materials are transported and mixed by the inward and outward pulsation of the tide. In the Suisun Bay area, the tidal excursion is up to 10 km (6.2 miles) with the net riverflow moving seaward. In general, there is a salinity gradient from the fresh water Delta to the saline ocean. The river continuously provides fresh water to the Estuary; however, tidal induced turbulences cause mixing and maintains the location of the salinity gradient zone relative to river outflow. Many of the shallow areas of the Bay are well mixed as a result of tidal induced turbulence.

TWO-LAYERED FLOW ESTUARINE CIRCULATION

A distinct two-layered-flow with vertical mixing circulation pattern has been identified in the Bay-Delta Estuary (Arthur et al., 1975). This circulation pattern is caused by the net downstream flow of fresh river water over the net upstream flow of more saline water, and results in a zone of suspended materials entrapment (figure 7). This entrapment zone moves seaward with increased salinity stratification as river discharge increases (figure 8). Laboratory studies demonstrated that above a specific conductivity of 1,000 umho/cm suspended materials tend to flocculate and settle into the lower depths at increased rates above fresh water settling rates. This influences the accumulation of suspended materials in the Bay. In studies of this zone at different river discharges, other factors were also noted by the USBR. During high outflow, near bottom total suspended solids (TSS) exceeded 2,000 mg/l in the entrapment zone. This was approximately 20 to 40 times the measurements at locations, seaward or landward from the zone. Surface measurements of TSS in the zone were 20 to 40 times less than the bottom. Maximum TSS in the entrapment zone during low outflow exceeded 300 mg/l. This was over 6 times the seaward and landward measurements, but less than double the surface measurements. The greater differences at high outflows was attributed to the fact that increased salinity stratification occurred at high outflows which resulted in less vertical

NET FLOW PATTERNS



SEDIMENT TRANSPORT PATTERNS

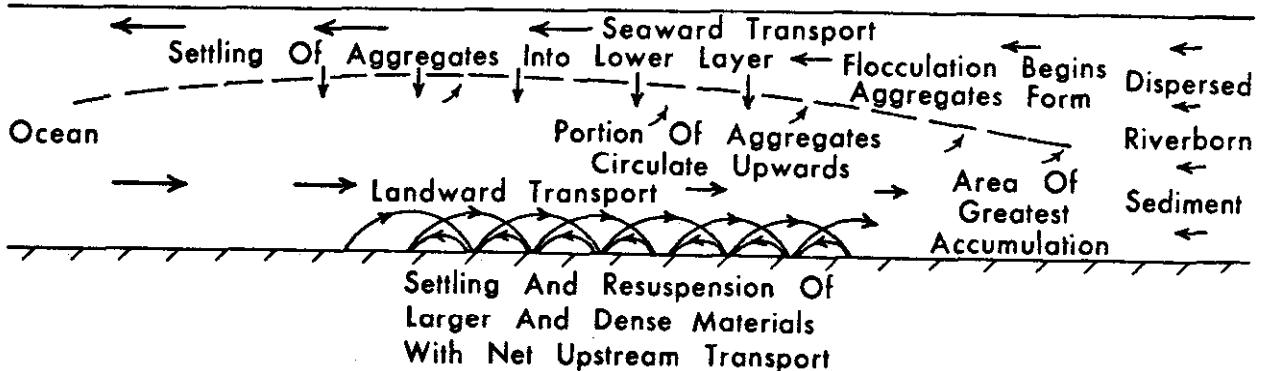


figure 7, TWO-LAYERED FLOW WITH VERTICAL MIXING ESTUARY

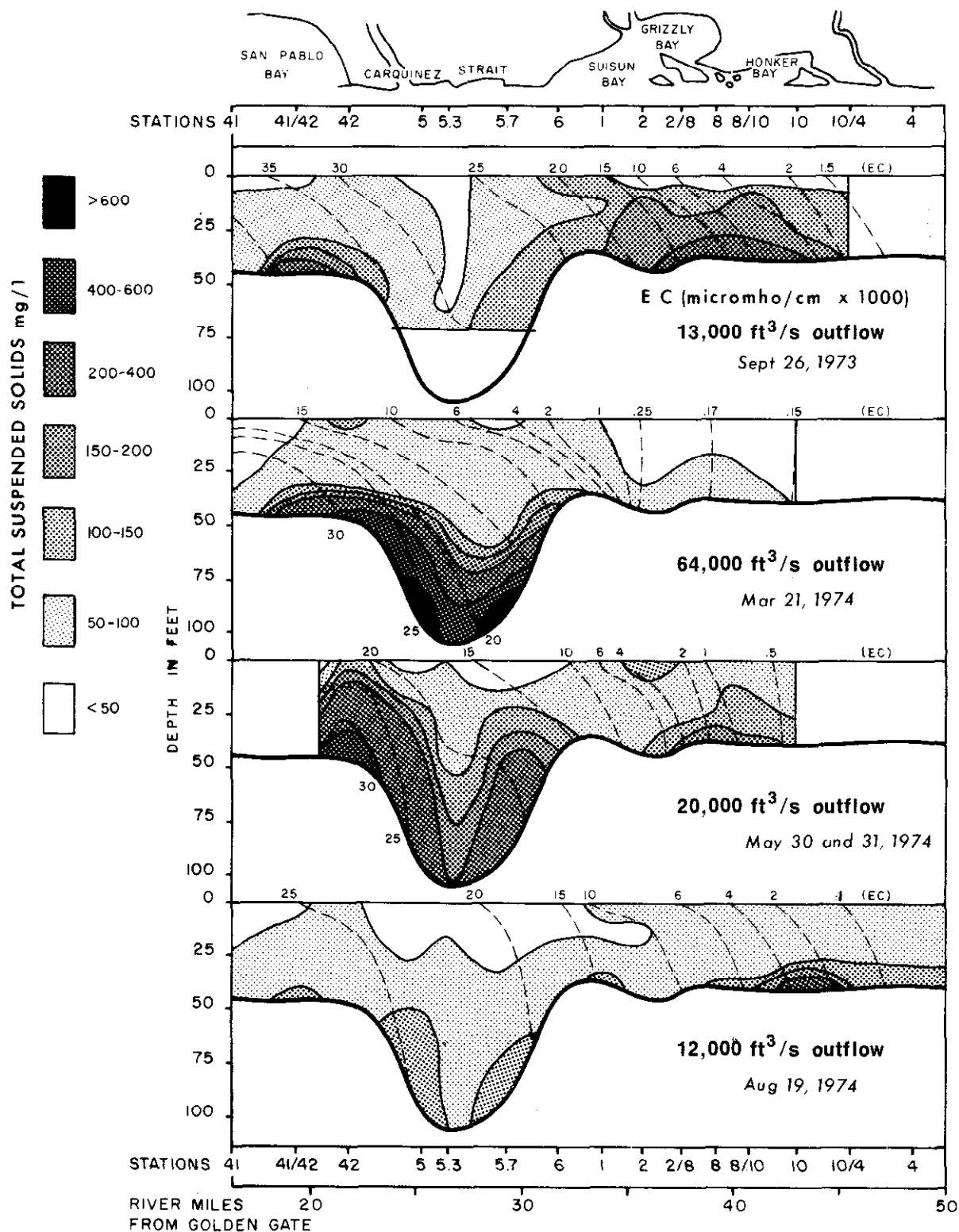


FIGURE 8 - RELATIONSHIP BETWEEN THE ACCUMULATION OF TOTAL SUSPENDED SOLIDS (SHADED AREAS) AND SALINITY (DASHED EC CONTOUR LINES) DURING HIGH SLACK TIDE

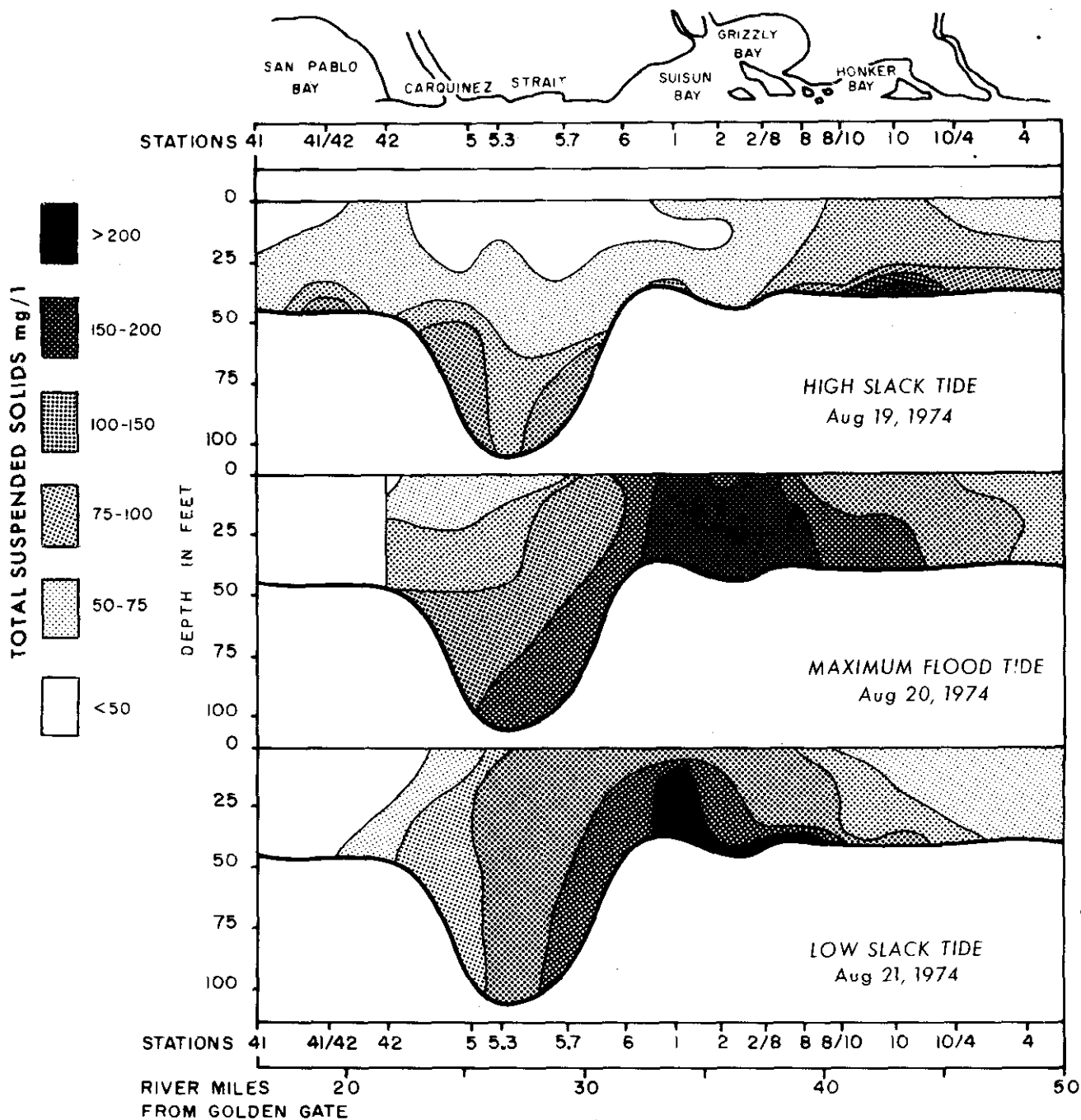


FIGURE 9 — ISOSUSPENDED SOLIDS CONTOURS DEMONSTRATING THE CHANGES IN LOCATION OF THE ENTRAPMENT ZONE AND THE SUSPENDED SOLIDS CONCENTRATIONS WITH DIFFERENT TIDAL PHASES

mixing than during low riverflows. Maximum tidal velocities during flood and ebb tides increased turbulence, resulting in resuspension of TSS in the entrainment zone to nearly double the level during slack periods (figure 9). The TSS composition was primarily inorganic generally 80-90 percent year around but varied seasonally.

Dredging results in resuspension and deposition of sediment even though it does not induce a sediment transport current. Dredged materials removed from the ship channels are disposed of in other parts of the Estuary. One such site is near Mare Island west of Carquinez Strait. The Corps of Engineers are presently conducting a study to trace the dispersion of dredged materials. Portions of the trace materials have been found up-stream in the Suisun Bay area demonstrating the two-layered flow transport of these materials. Tidal action also greatly influences the distribution of these trace materials.

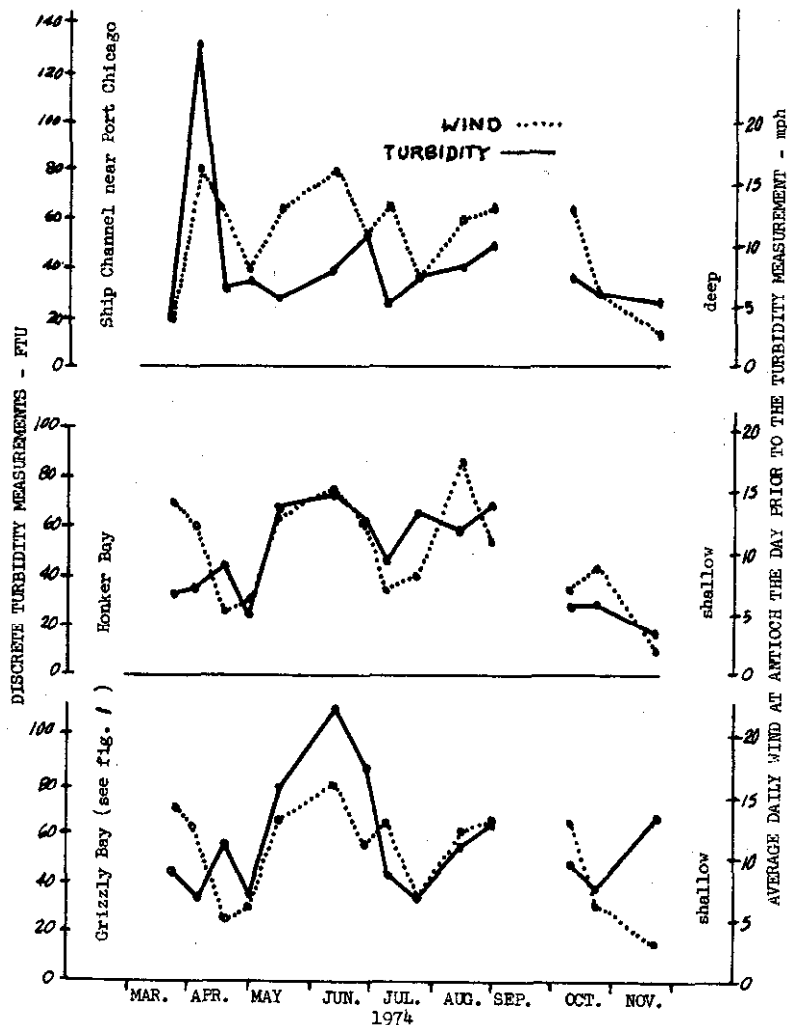


figure 10, WIND EFFECTS ON TURBIDITY IN THE SHALLOW AND DEEP WATERS

SURFACE WAVE ACTION

Wind has been observed to have a significant effect on sediment resuspension primarily in the shallow portions of the Estuary. The data presented in figure 10 illustrates the relationship between turbidity and the average wind velocity for the previous day. This figure does indicate some relationship in shallow waters between increases in wind velocity with increases in turbidity, believed to be a result of wind generated turbulence at the water surface.

Another factor often suspected to cause sediment resuspension is waves generated by the movement of watercraft; but this effect was not analyzed. The U.S. Army Corps of Engineers (1975) indicated that each year approximately 15 times the annual river sediment discharge to the Estuary is resuspended by tidal flows, wind-generated wave action, river inflows, watercraft turbulence, dredging, etc., and is redistributed within the Estuary.

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DIGITAL SIMULATION OF AGGRADATION AND DEGRADATION IN NATURAL STREAMS

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ABSTRACT

The analysis of scour and deposition by modeling the interaction between the water-sediment mixture, sediment material forming the stream's boundary and the hydraulic characteristics of flow is presented in this paper using the Hydrologic Engineering Center's computer program entitled "Scour and Deposition in Rivers and Reservoirs." This simulation program is particularly useful for analyzing the impact of changes in energy gradient, channel width, inflowing sediment load, or bed material grain size on future trends in channel aggradation or degradation. The results of the analysis are useful for estimating the impact of aggradation on the design profile for levees and the amount, frequency, and location of maintenance dredging, as well as the effect of alternative measures, for maintaining a channel.

Example applications illustrate the model calibration and demonstrate its applicability and usefulness for a wide range of hydraulic and sediment conditions.

INTRODUCTION

It is now well recognized that sediment transport and its effects on natural streams are significant factors to be considered in any rational and meaningful land and management study in river basins. Although the many factors affecting the mechanics of sediment transport are quite complex defying a rigorous analysis, it is possible to develop a mathematical model based on the existing knowledge of empirical relationships between the various hydraulic and hydrologic factors involved. These factors include characteristics and sources of inflowing sediment load and bed material grain-size, slopes and cross sections of the streams, the rate of streamflow and its variability and other hydraulic parameters.

The purpose of this paper is to present an analysis of scour and deposition in natural streams by modeling the interaction between the water-sediment mixture, sediment material forming the stream's boundary and the hydraulic characteristics of flow using the Hydrologic Engineering Center's computer program entitled "Scour and Deposition in Rivers and Reservoirs." This simulation technique is useful for analyzing the impact of changes in energy gradient, channel width, inflowing sediment load or bed material

grain-size on future trends in channel aggradation and degradation. More specifically, the method facilitates the prediction of areas of scour and deposition along the river and identifies the shifting tendencies of a rating curve due to sediment movement.

The results of the analysis are thus useful for estimating the amount, frequency and location of maintenance dredging as well as for evaluating the performance of alternative measures for maintaining a channel such as sediment traps and contraction works. The impact of aggradation on the design profile for levees and deposition in overbank floodways has also been studied.

Example applications illustrate the model calibration and demonstrate the model's applicability and usefulness for a wide range of hydraulic and sediment conditions. These include the Sacramento Deep Water Ship Channel and the Atchafalaya River Floodway Study.

THEORETICAL BASIS OF SIMULATION TECHNIQUE

Effective use of the simulation program necessitates a recognition of the limitations of the model. This is a state-of-the-art, one dimensional, steady flow, movable bed model with no provision for simulating the existence of meanders or specifying a lateral distribution of suspended sediment load across the river channel. In addition, density currents and secondary currents are not accounted for in the program.

The program first computes water depth, width, velocity and slope through all reaches in the study area for a specified inflowing water discharge using standard step method for the calculation of water surface profiles. Conveyance is calculated between adjacent cross sections. Next, the potential transport capacity of each reach is calculated from hydraulic parameters for each grain size classification ranging from clay to gravel (Hydrologic Engineering Center, 1974). The basis for simulating the movable bed is the finite difference solution of the continuity equation for sediment material - the Exner equation:

$$\frac{\partial G}{\partial X} + B_o \frac{\partial y_s}{\partial (DD)} = 0$$

where:

G = sediment load in cu. ft./sec.; DD = time in seconds;
 y_s = depth of sediment deposit above model bottom; X = distance along the channel; B_o = width of deposit (movable bed).

The primary controls on rate of scour are thickness of the active bed and amount of surface area armored. The thickness of active bed is calculated at the beginning of each time interval and is the layer of material between the bed surface and a hypothetical depth at which no transport will occur for the given gradation of bed material and flow conditions. The movable boundary problem is simplified into one involving only a movable bed in the channel. The horizontal location of the channel banks is considered fixed.

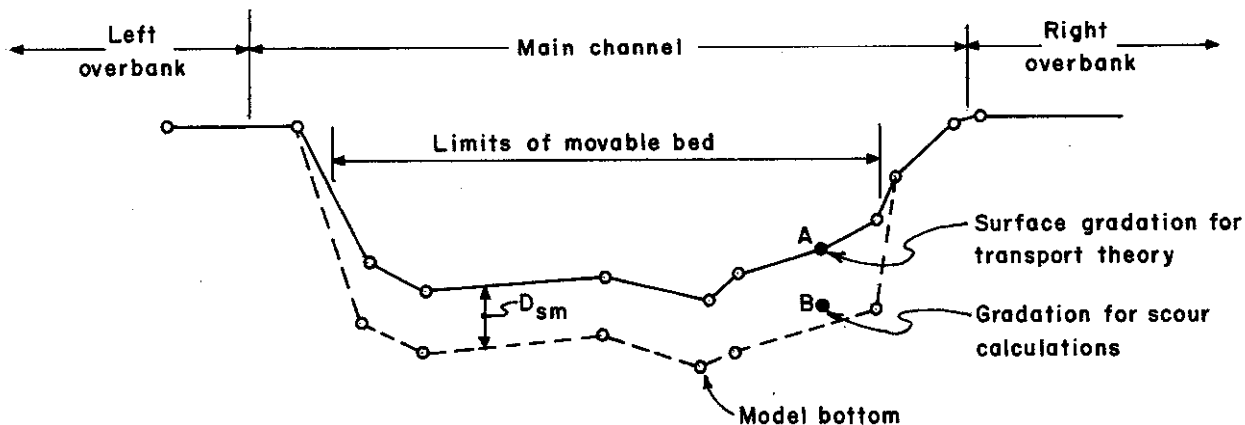


Figure 1. Description of the Movable Bed.

The difference between the sediment inflow to a reach and the computed transport capacity of the reach, considering sorting of grain sizes, is calculated for the incremental time period, and the resulting volume of sediment scoured or deposited is expressed in terms of equivalent changes in bed elevation in accordance with the above equation. The next discharge is entered and the entire cycle is repeated.

PERFORMANCE CRITERIA AND MODEL VERIFICATION

The authenticity and the range of applicability of any computational technique involving several assumptions, such as those outlined in the preceding section, for simulating the mechanics of sediment transport can only be evaluated when it is applied to a practical problem. Model calibration and verification must precede the actual problem analysis.

Model calibration is the intricate process of adjusting the n-values, inflowing sediment loads and natural levee elevations along the river channel until the model demonstrates the ability to reproduce, with reasonable accuracy, the historical water surface elevations, the changes in bed profile and the accompanying sediment deposition characteristics. Once calibrated, a reconstitution of the behavior of the prototype can be attempted. Thus, reasonable agreement between the computed results from the calibrated model and the available historical observations must form an essential criterion in ascertaining the reliability of the simulation technique for the hydraulic and sediment conditions encountered in the basin. This is called model verification and is accomplished by reconstituting an experienced event or period of record.

INPUT DATA REQUIREMENTS

Basic data for the digital model is grouped into three major categories:

Geometric Data describing the initial condition of cross sections, coded in coordinates, reach lengths and Manning's n-values for water surface profile calculations.

Sediment Data describing the grain size of sediment material in the streambed as well as the gradation and amount of total inflowing sediment load, fluid properties and sediment properties.

Hydrologic Data describing the water discharge hydrograph and temperatures.

EXAMPLE APPLICATION - THE ATCHAFALAYA FLOODWAY FLOW-SEDIMENT MODEL

The Atchafalaya floodway is a major component of the Mississippi River and Tributaries' flood control system. The floodway is about 15 miles wide and 150 miles long and extends from the Old River Control Structure near Simmesport, Louisiana to the Gulf of Mexico. There is evidence that sediment movement in the floodway is rapidly altering the existing, open-water-swamp-type habitat, and a fifty year projection into the future is needed for the environmentally oriented studies currently underway in the basin by the New Orleans District, Corps of Engineers. The Hydrologic Engineering Center was engaged to calibrate the scour and deposition model using observed conditions in the basin so that New Orleans District personnel could make the necessary sedimentation studies. The purpose of this flow-sediment model will be to simulate scour and deposition in the main channel from the Old River Control Structure to the Gulf plus deposition in the overbanks downstream from Whiskey Bay Gage, River Mile 55.

Figure 2 shows a sketch of the study area. Flow enters the floodway at three different points, all of which are located near the upstream end. Discharges from the Old River Control Structure and the Red River combine to furnish the main channel flow for the Atchafalaya River at Simmesport, Louisiana. The Morganza Control Structure is available to pass Mississippi River water into the East Atchafalaya Basin Floodway, and a fuse plug levee is available to pass water into the West Atchafalaya Basin Floodway. From Simmesport downstream to the Whiskey Bay Gage, a distance of 50 miles (80.5 km), the basin functions as three separate floodways with the Atchafalaya River being contained between levees along each bank of the channel. Downstream from the Whiskey Bay Gage, the basin functions as a single floodway. There are two outlets to the Gulf of Mexico. The main outlet, at Morgan City, conveys 70 percent of the total flow; and the Wax Lake Outlet, located on the west side of the basin, conveys the remaining 30 percent of the total flow.

The usual approach in calibrating the flow-sediment model is to select two points in time when channel geometry is known, to use the earlier geometry for initial conditions and to simulate the response of the prototype by analyzing the inflowing water-sediment mixture during the time interval between channel surveys. Since the flow-sediment model retains the original width of each cross section and adjusts the bed elevations in response to scour and deposition, reconstitution of the observed thalweg profile is not a feasible performance criteria. Rather, scour or deposition trends are reflected in cross sectional area changes and in terms of trends in stage-discharge curves.

The time period when an adequate data set was available for calibration of this model was 1963-1973. Actually, fifty years of historical data were available for sediment ranges across the floodway and for water discharges, but the detailed information required for sediment loads and gradation of material in the sediment load were available for only eight years, 1966 through 1973. Sediment data during the period 1963 through 1965 were developed by averaging the available record so initial channel conditions could be established with the 1963 survey.

During the calibration period rather extensive channel alignment and dredging work was performed downstream from the Whiskey Bay Gage. Although the computer program can accommodate dredging in the main channel, it cannot simulate the construction of cutoffs and channel realignment. Therefore, calibration of channel scour and deposition utilized the reach from Simmesport to the Whiskey Bay Gage. In subsequent verification tests, performance of the model was compared with historical records for the lower part of the basin. Those tests will not be presented in this paper.

The first step in calibration was to establish n-values. This was accomplished by reconstituting water surface elevations observed during 1963 at six gage locations in the floodway for a range of discharges from 20,000 cfs (556 cms) to 850,000 cfs (24,072 cms). The calculated values agreed with observed elevations within half a foot (15 cm). The resulting n-values were considered representative of hydraulic roughness throughout the calibration period. This step utilized the 1963 geometry and a fixed bed condition.

The next step addressed movable bed conditions. The total inflowing sediment load was subdivided into clay (grain size less than .004 mm), two sizes of silt (.008 mm - .016 mm and .016 mm - .032 mm) and four sizes of sand (very fine, fine, medium and coarse).

The gradation of the bed was taken from 1951 samples. This appeared reasonable since periodic sampling at Simmesport did not indicate a trend to change bed gradation. (Nevertheless, the District is in the process of obtaining more recent bed samples at ten mile intervals throughout the basin).

The discharge hydrograph at Simmesport was converted into a discharge histogram for the calibration period and a simulation run was made with this flow-sediment model. The trend expected at Simmesport based upon the observed lowering of the Simmesport stage discharge rating curve was one of degradation during this period. The flow-sediment model, on the other hand, predicted aggradation during this first calibration run. A close look at the computations revealed that the inflowing sand load in the medium and coarse sand sizes was far greater than could be transported by the Toffaleti transport relationship (Toffaleti, 1968). The inflowing sand load was reduced to 30 percent of the observed value and results of the simulation are shown on figure 3. The 100,000 cfs (2,832 cms) flow was selected for comparison because it represents a low flow and consequently reflects a high sensitivity to the impact of scour and deposition.

Figure 4 shows the observed and calculated trends at two gages in the reach. Melville is located at River Mile 29, and Krotz Springs is at River Mile 41. The discharge in figure 4, 400,000 cfs (11,328 cms), is about bank full. These curves tend to verify the calibrated model since only the Simmesport Gage was used in the calibration.

A typical cross section is shown in figure 5. The 1963 initial conditions, the 1973 observed section, and the 1973 calculated section are all three shown. Whereas the actual cross section shape changes between 1963 and 1973, the 1963 cross section shape is retained in the computed section. The overall cross sectional area is reasonably close to the observed section. It should be pointed out that major errors in cross sections were obvious when the bed was permitted to change in response to sediment transport capacity. Reevaluation of several sections in the calibration reach revealed inconsistent geometry with respect to time, and resulted in changes to several cross sections based on the size and shape of adjacent sections. These changes did not significantly affect n-value calibration, but they were important in sediment transport calculations.

The reduction in sand load had a significant impact on channel behavior, but it amounted to only 10 percent of the total volume of sediment entering the basin. Therefore, it will not impact significantly on total deposition.

CONCLUSIONS

It is possible to perform movable bed digital model studies as demonstrated by the calibration of the flow-sediment model in the Simmesport to Whiskey Bay reach. The calculation technique is sensitive to geometry, n-values, inflowing sediment load, gradation of material in the stream bed and in the inflowing sediment load, water discharge hydrograph and water temperature. Whereas the analytical procedures are a simple extension of one-dimensional, fixed-bed hydraulic theory to incorporate sediment transport, armoring and sorting of grain sizes, the interpretation of results requires a considerable understanding of river behavior. Data requirements are flexible, but provisions in the computer program permit the utilization of a very detailed data set if one is available.

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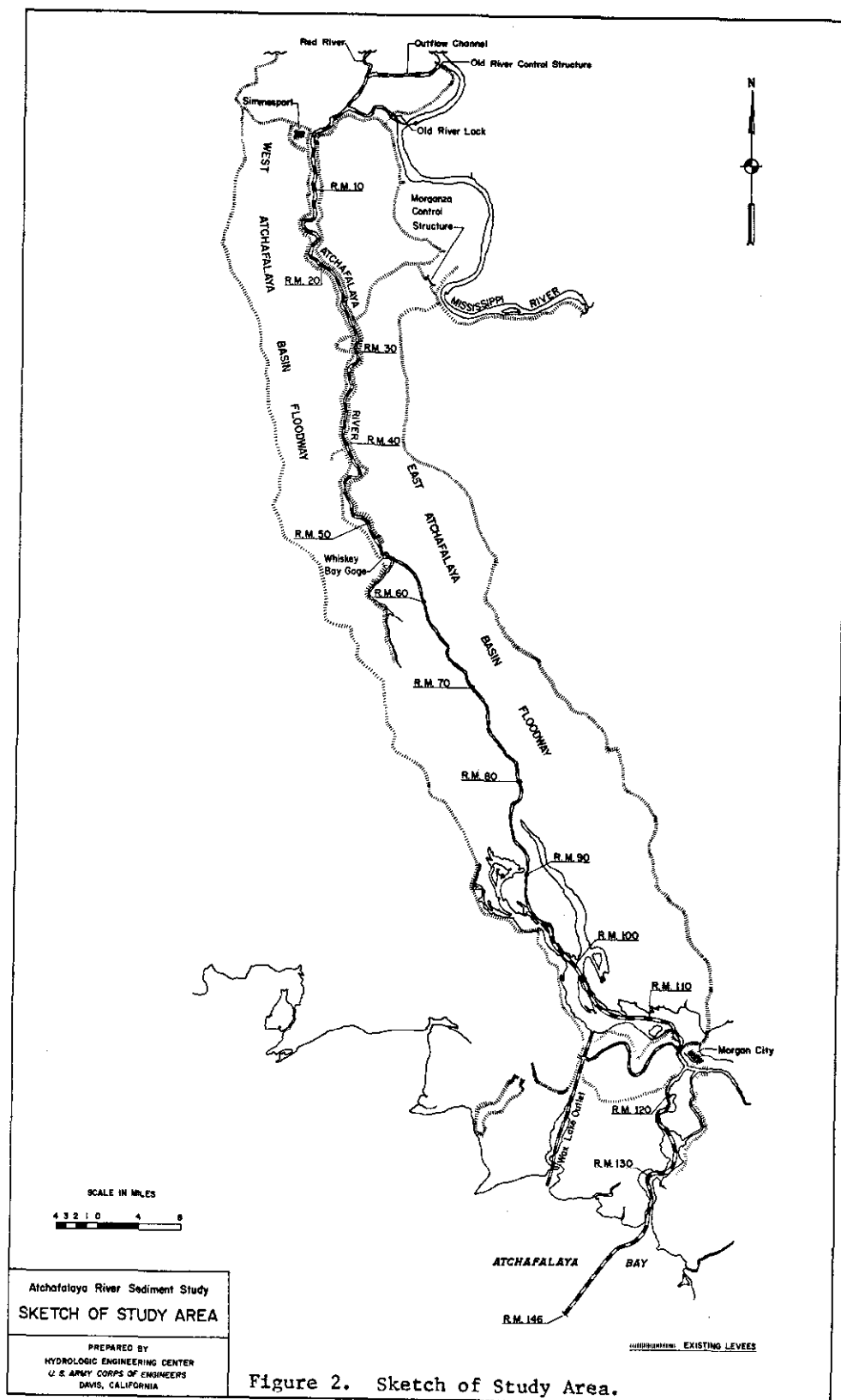


Figure 2. Sketch of Study Area.

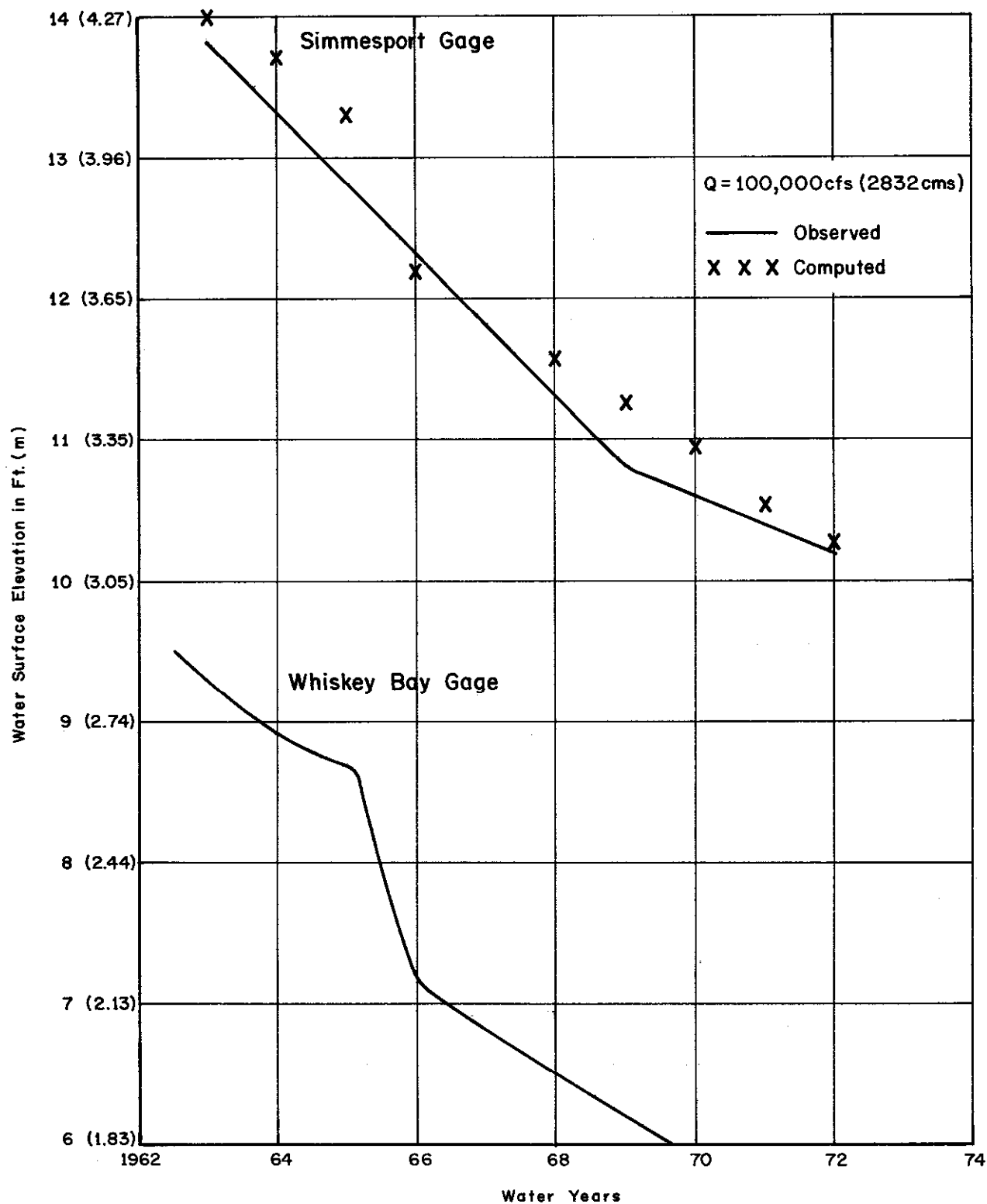


Figure 3. Trend at Simmesport.

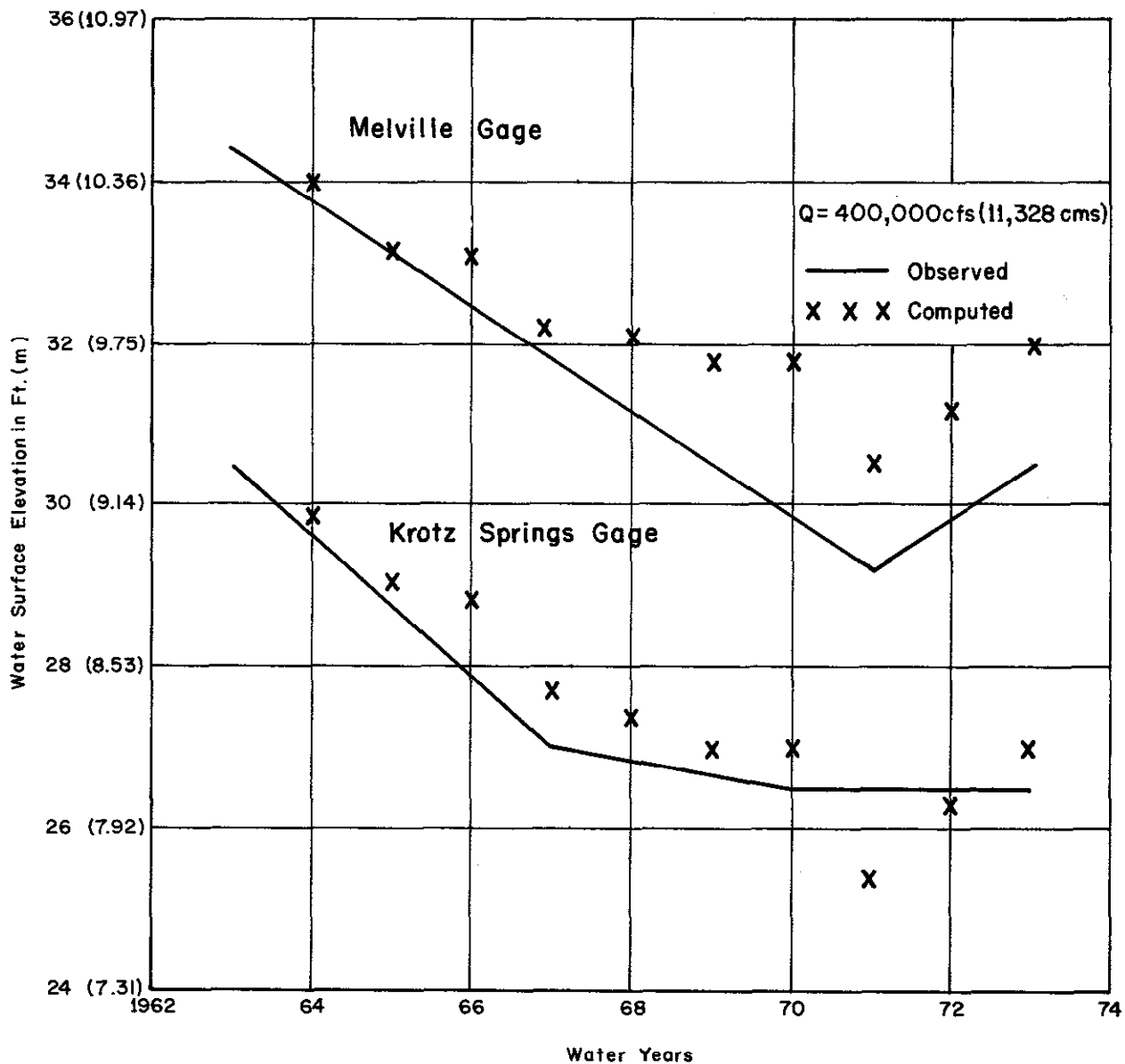


Figure 4. Verification of Trend at Melville and Krotz Springs.

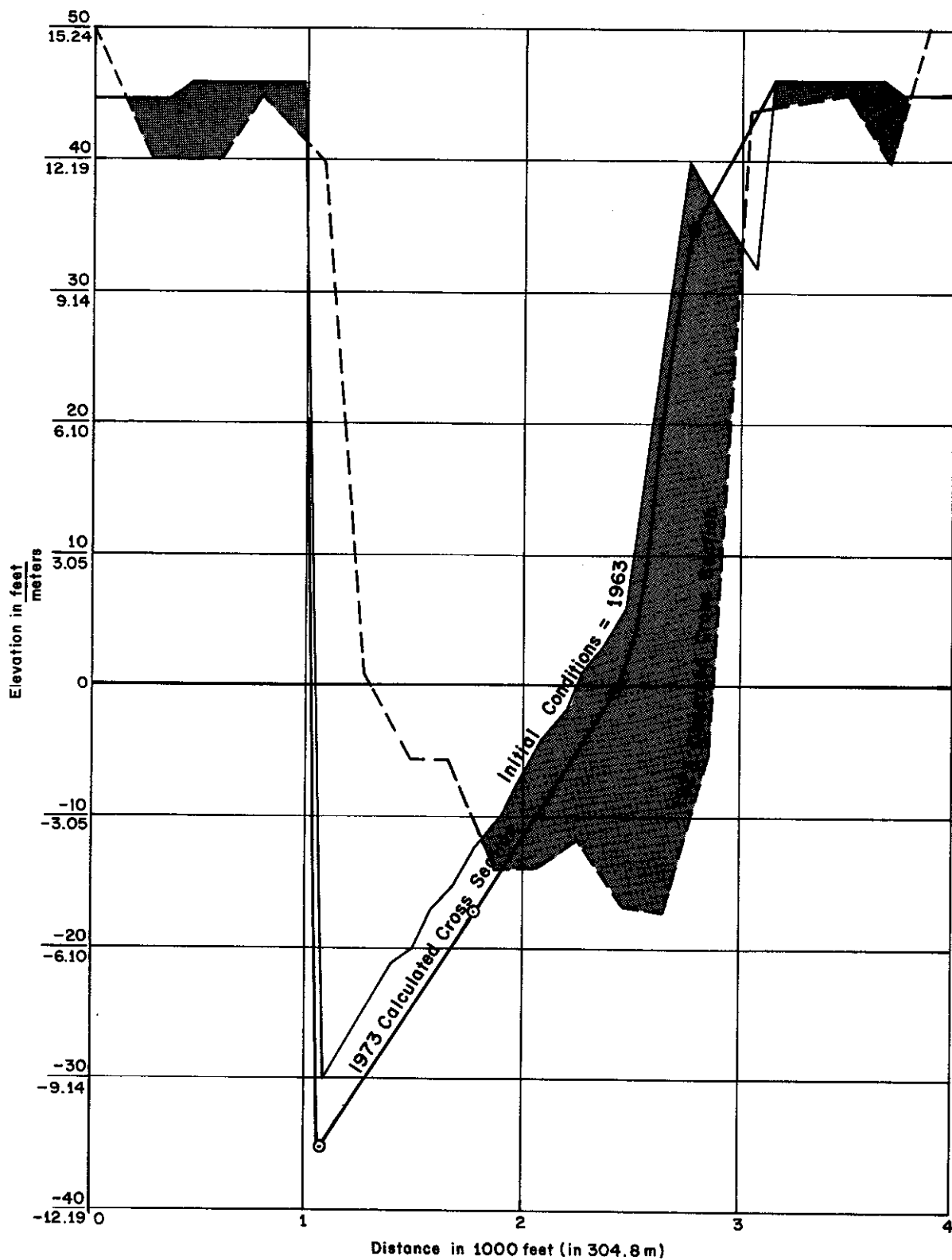


Figure 5. Typical Cross Section.

PROPOSED REVISION TO THE MODIFIED EINSTEIN METHOD OF COMPUTING TOTAL
SEDIMENT DISCHARGE

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ABSTRACT

Corrections are applied to four equations used in the modified Einstein method of computing total sediment discharge in open channels. The corrections eliminate some of the empiricism in the procedure without any apparent degradation in the accuracy of prediction.

INTRODUCTION

The modified Einstein method (Colby, 1955) of computing total sediment discharge in open channels is widely used in the United States and abroad. The method involves several equations and graphs, many of which are based on Keulegan's (1938) solutions of the Prandtl-von Kármán velocity distribution equation (Prandtl, 1926; von Kármán, 1930). The Keulegan (1938) equations are:

$$u_y = v_* (8.5 + 5.75 \log y/k_s) \quad (1)$$

and

$$V = v_* [(8.5 - .9b) + 5.75 \log R/k_s] \quad (2)$$

in which

- u_y = average point velocity at a distance y from the closest boundary;
- v_* = average shear velocity at the boundary; equals $\sqrt{\tau_0/\rho}$ where τ_0 equals the shearing stress at the boundary and ρ equals the density of the fluid;
- k_s = equivalent sand roughness for a particular roughness k ; k is the effective height of the roughness elements in a channel;
- V = mean cross-sectional velocity;
- V_* = mean cross-sectional shear velocity; equals $\sqrt{gRS_e}$;
- R = hydraulic radius;
- g = acceleration due to gravity;
- S_e = slope of the energy grade line;
- b = 1 divided by κ ; and
- κ = universal constant characterizing turbulence; assumed to be 0.4.

A general form of equation (2) is (Keulegan, 1938):

$$V=V_* (C-.9b)+5.75 \log (R/k) \quad (2A)$$

in which

C = a coefficient.

Equations (1) and (2) are applicable for fully developed turbulent flow in non-wavy rough uniform channels in which the resistance to flow is produced by the boundary surface. To use equations (1) and (2), however, it is necessary to know the relation between the roughness factor, k , for a data set of interest and the equivalent sand roughness, k_s .

A general equation relating k and k_s is (Keulegan, 1938):

$$5.75 \log k_s = 8.5 - C + 5.75 \log k \quad (3)$$

Equation (3), according to Keulegan (1938), is the expression which gives the equivalent sand roughness k_s for the particular roughness k . Physically, if a velocity u is observed at a distance y from a wall of arbitrary roughness k under a known shear, the same velocity will be obtained at the same point and for the same shear if the particular roughness is replaced by closely packed uniform sand grains of roughness k_s . It is not practical to compute the equivalent sand roughness, k_s , using equation (3), for every roughness height, k , found along the boundaries of uniform open channels. Instead, the effort traditionally has been to find a readily measurable parameter that could be used to approximate k_s in equation (3).

Equations (2A) and (3) can be combined to give a general equation that is applicable directly to turbulent flow in non-wavy rough uniform channels. Note that equation (3) can be rearranged to give:

$$C = 8.5 - 5.75 \log (k_s/k). \quad (3A)$$

Equations (2A) and (3A) are combined to give:

$$V/V_* = (8.5 - 0.9b - 5.75 \log x_o) + 5.75 \log (R/k) \quad (4)$$

in which

$$b = 2.50, \text{ and}$$

$$x_o = k_s/k.$$

The term x_o is introduced to simplify later discussions. Equation (4) is Keulegan's theoretical resistance equation for turbulent flow in non-wavy rough uniform open channels. Burkham (1975) has developed a relation from which, by using equation (3A), x_o can be approximated for a wide range of sediment sizes (fig. 1).

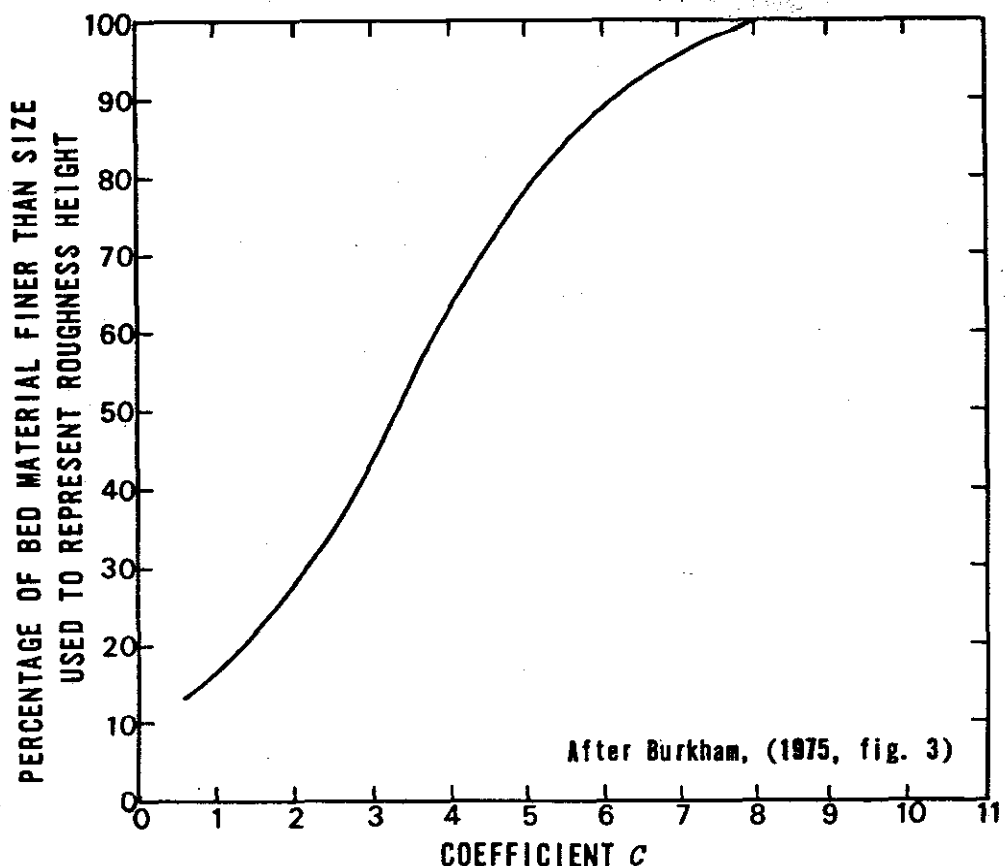


FIGURE 1.--Relation between percentage of bed material finer than size used to represent roughness height and the coefficient C in the Keulegan equation;
 $V/V_* = (C - .9b) + 5.75 \log R/k$.

In the modified Einstein method of computing total sediment discharge, D_{65} is used directly to represent the roughness height, k_s , in equations developed from equations (1) and (2) (Colby, 1955); the correction factor $-5.75 \log x_0$ (equals $-5.75 k_s/D_{65}$) was not applied. The term D_{65} is defined as the particle size for which 65 percent of the bed is finer. Burkhams (1975) has shown that x_0 is about 5.5 when D_{65} is used to represent the roughness height; therefore, the use of D_{65} to directly represent roughness height in the modified Einstein method (Colby, 1955) of computing total sediment discharge may introduce significant bias. Bias is introduced when the mean of an infinite number of computed (or estimated) values for a parameter is not equal to the true mean for the parameter.

PURPOSE AND SCOPE

The primary objective of this investigation was to apply the factor $-5.75 \log 5.5$ to pertinent equations used in the modified Einstein method and to determine whether significant improvement results. Three criteria for improvement of the results were used. First, improvement is indicated if the accuracy of prediction of total sediment discharge is improved. Second, improvement is indicated if the correction of pertinent equations eliminates some of the empirical adjustments contained in the method. Third, improvement is indicated if the elimination of empirical adjustments does not degrade the accuracy of prediction.

Data presented in the Colby (1955) report on the development of the modified Einstein method were used in the current analysis. The data, which are for different times from 1949 to 1953 for cross sections in a 3-mile reach of the Niobrara River near Cody, Nebraska, are of two types--hydraulic and sediment. The hydraulic data for each time and section are: discharge; cross-sectional area, width, depth, and velocity; water-surface slope; and temperature ($^{\circ}\text{F}$). The sediment data are: concentration, size distribution, and discharge of suspended sediment and of total sediment load; average of depths at the sampled verticals; and size distribution of the sediment in the channel bed.

This report describes the progress that has been made thus far in the study. The result should be considered a first approximation of the final result. A major point should be understood concerning this paper. As stated previously, equations (1) and (2) are applicable for fully developed turbulent flow in non-wavy rough uniform channels; the resistance to flow is due to grain roughness only. Einstein's (1938) rationale for using equations (1) and (2) to develop a method of estimating sediment transport in sand channels having significant bed forms is based on the assumption that the energy of turbulence corresponding to shape resistance does not contribute significantly to bedload motion and may be largely neglected in the entire sediment picture. The effect of dunes on sediment transport was not considered in Einstein's mathematical formulation of his bedload equation; however, the effect of dunes is considered empirically in his adjustment curves. The effects of dunes on sediment transport also is considered only empirically in the modified Einstein method and in the method described in this progress report.

METHODS

The study procedure was as follows:

1. Pertinent equations and graphs used in the modified Einstein procedure for determining total sediment load were corrected. This required a review of the Einstein and modified Einstein procedures; identification of theoretical and empirical adjustments made by Einstein (1950) so equations and graphs for uniform particles would apply to mixtures; and identification of empirical adjustments made by Colby (1955) to eliminate bias in the modified Einstein method.

2. Computer programs for the modified and the revised modified methods were developed.

3. Using the Colby data, sediment discharges were computed and compared with measured sediment discharges; comparisons were made by size ranges and for total sediment discharges. The standard error of estimate for sediment discharges computed by the modified Einstein method was compared with the standard error of estimate of discharges computed by the first approximation of the revised method; this comparison gives an indication of improvement.

CORRECTION OF PERTINENT EQUATIONS

The development of the revised procedure required that four equations used by Colby (1955) be corrected or replaced. They are:

$$V = 5.75 \sqrt{32.2 (RS)_m} \left[\log \left(\frac{12.27 (d)x}{D_{65}} \right) \right] \quad (5)$$

$$P = \frac{1}{0.434} \log \left[\frac{30.2 [d]x}{D_{65}} \right] \quad (6)$$

$$\Psi^*(m) = (1.65 D_{35}) / (RS)_m; \text{ and} \quad (7)$$

$$\Psi^*(m) = (0.66 D) / (RS)_m \quad (8)$$

in which

$(RS)_m$ = a term representing the product of the hydraulic radius, R , and the hydraulic slope, S ; the subscript m is used to denote a quantity that is computed according to the modified Einstein procedure;

d = depth of water;

P = a parameter of total load transport introduced by Einstein (1950) to simplify the discussion of his procedure;

x = a dimensionless corrective parameter introduced by Einstein (1950);

$\Psi^*(m)$ = intensity of shear for sediment grains of a size range; the parameter Ψ^* was introduced by Einstein (1950) to simplify the discussion of his procedure.

Equation (5) is used in the modified Einstein procedure to compute, directly or indirectly, values for $(RS)_m$, $(V^*)_m$, $(x)_m$, and $(\delta)_m$. Values for V to be used in equation (5) are determined directly from

measured discharges and cross-sectional areas. A value for $(RS)_m$ is determined from a trial-and-error solution of equation (5) by using x as a trial factor and the relation in Colby's figure 44 (Einstein, 1950, fig. 4), in which x is a function of k_s/δ . As previously discussed, the term k_s in the modified Einstein procedure is assumed to be equal to D_{65} . The term δ , the thickness of the laminar sublayer, is equal to $(11.6\nu)/(V_*)_m$ in which ν is the kinematic viscosity and $(V_*)_m$ is shear velocity (equals $\sqrt{g(RS)_m}$).

Equation (5) is corrected for the revised modified Einstein procedure by replacing D_{65} with $5.5D_{65}$ to give:

$$V = 5.75 \sqrt{32.2(RS)_{mc}} \log \left[\frac{2.23(d)x}{D_{65}} \right] \quad (9)$$

The subscript mc is used to denote a quantity that is computed according to the corrected procedure. Equation (9) is used in accordance with the modified Einstein procedure to determine values for $(RS)_{mc}$, $(V_*)_{mc}$, $(x)_{mc}$, and $(\delta)_{mc}$.

An equation for $(P)_{mc}$ is obtained by replacing D_{65} in equation (6) with $5.5D_{65}$. The resulting equation is:

$$(P)_{mc} = \frac{1}{0.434} \log \left[\frac{5.49(d)x}{D_{65}} \right] \quad (10)$$

Colby (1955) used equations (7) and (8) to replace Einstein's (1950) equation (54):

$$\Psi_* = \xi Y (\beta/\beta_x)^{2\Psi} \quad (11)$$

in which

ξ = Einstein's (1950) hiding coefficient for mixtures of particle sizes;

Y = Einstein's (1950) correction for mixtures of particle sizes for the lift coefficient, c_L ;

c_L = lift coefficient for uniform particles;

$\beta = \log (10.6)$;

$\beta_x = \log (10.6X/\Delta)$;

X = a term introduced by Einstein (1950) to represent a characteristic grain size in a mixture;

$\Delta = k_s/x$, the apparent roughness of a channel; and

Ψ = intensity of shear for particles of a uniform size.

Equations (7), (8), and (11) were used by the respective authors in the computation of bedload. In the Einstein procedure for determining bedload, equation (11) was used to determine a value for Ψ_* for each grain-size range; each Ψ_* was applied to Einstein's figure 9 to determine a value for Φ_* --the intensity of transport for the size range; and the Φ_* was used directly to determine load of bed material for the range. In the modified Einstein procedure for determining bedload, equations (7) and (8) are used to determine values for $\Psi_{*(m)}$ for each particle-size range; the largest of the two values was applied in place of Ψ_* to Einstein's figure 9 to determine a value for Φ_* for the range; and, $\frac{1}{2}$ of the value of Φ_* was used in the computation of load of bed material for the size range.

There is no direct way of determining corrections needed for the empirical equations (7) and (8); therefore, the approach in developing equations to replace them was indirect, included several assumptions, and involved several steps.

The first step in developing equations to replace equations (7) and (8) was to develop a general equation for the intensity of shear, $\Psi_{*(mc)}$. The procedure used to develop the general equation was similar to that used by Einstein (1950) to develop his equation (54). The first question to be resolved in developing the equation is "what velocity will be used to represent the velocity that causes motion?" Einstein (1950) found that the representative velocity acting on a uniform particle must be measured at a distance $0.35D$ from the theoretical bed. We assume that the representative velocity acting on particles (in a mixture) that have an average roughness height equal to k_s must be measured at a distance of $0.35k_s$ from the theoretical bed; furthermore, we assume that this velocity is acting on all particles in the mixture. The general equation is:

$$\Psi_{*(mc)} = \xi_{(mc)} Y_{(mc)} \left(\beta / \beta_x(mc) \right)^2 \Psi_{(mc)} \quad (12)$$

in which

$$\beta_x(mc) = \log(10.6x);$$

$$\xi_{(mc)} = \text{hiding coefficient};$$

$$Y_{(mc)} = \text{correction for the lift coefficient } c_L; \text{ and}$$

$$\Psi_{(mc)} = (1.65) / (RS)_{(mc)}.$$

Values computed using equation (12) will be used with the relation in Einstein's figure 9 to determine values for Φ_* , therefore, results obtained using equation (12) must be applicable directly to the Einstein relation.

The next step in developing equations to be used instead of equations (7) and (8) was to develop an equation for the intensity of shear, $(\Psi_*)_{k_s}$, on a particle having a diameter, D , equal to k_s . To develop this equation, the first question that had to be resolved is "what value should be used for the correction factor $(Y)_{k_s}$?" Einstein (1950) found that the correction, Y , was 0.53 for relatively large particles in a mixture. Burkham (1975) found that a relatively large particle in a mixture was the size needed to represent k_s in equation (2). Therefore, 0.53 is assumed to be a reasonable first approximation for $(Y)_{k_s}$. Using arguments similar to those used to approximate a numerical value for $(Y)_{k_s}$, we assume that a value for the hiding coefficient $(\xi)_{k_s}$ is 1. Considering the assumptions for $(Y)_{k_s}$ and $(\xi)_{k_s}$, the intensity of shear on a particle having a size equal to k_s is:

$$(\Psi_*)_{k_s} = 0.53 \left(\frac{\beta}{\beta_x} \right)^2 1.65 \left[\frac{(D)_{k_s}}{(RS)_{mc}} \right] \quad (13)$$

in which the quantity $(S_s - S_f)/(S_f)$ is assumed to be 1.65 and S_s and S_f are the densities of the solids and of the fluid, respectively.

The next step in developing an equation to represent $(\Psi_*)_{k_s}$ is to replace $(D)_{k_s}$ in equation (13) with an optimal particle size. As the tail of a bed-material distribution is approached, sampling errors in determining a particle's fractile increases. We choose D_{95} as the optimal size to be used in equation (13); however, a relation which relates D_{95} to D_{k_s} is needed. The correct size of sediment to be used for $(D)_{k_s}$ is not known; however, it apparently is larger than D_{95} (Burkham, 1975). It is assumed that $(D)_{k_s}$ can be represented by $(x_1)D_{95}$ in which x_1 equals D_{k_s}/D_{95} . In order to be consistent with results of a previous study (Burkham, 1975), $D_{99.8}$ was selected to represent $(D)_{k_s}$. Assuming a normal distribution of sediment sizes in a mixture, the expected range of x_1 can be represented by

$$1 \leq x_1 = \frac{1+2.88\sigma}{1+1.64\sigma} \leq 1.76$$

The midpoint of the expected range is 1.38.

A value for σ for the Colby (1955) data is about 1.6; therefore, x_1 would be about 1.54 for the Colby data. However, because we are attempting to develop an equation for $(\Psi_*)_{k_s}$ which we hope will be representative of the expected range of distributions, we select 1.38 to represent x_1 .

If 1.38 d_{95} is used in equation (13) to replace $(D)_{k_s}$, the following equation results:

$$\begin{aligned} (\Psi_*)_{k_s} &= [0.53(1.38)(\beta/\beta_x)^2 1.65 D_{95}] / (RS)_{mc} \\ &= [1.2(\beta/\beta_x)^2 D_{95}] / (RS)_{mc} \end{aligned} \quad (14)$$

Our intent is to numerate coefficients so that equation (12) will be applicable to all ranges of sediment sizes in a mixture. However, equation (14) is used in the first approximation of the revised modified procedure for all ranges of sediment sizes having geometric means equal to or smaller than $1.38 D_{95}$. For the ranges of sediment sizes having a geometric mean larger than $1.38 D_{95}$, the equation used to determine the intensity of shear on the particle was:

$$\begin{aligned} (\Psi_*)_{mc} &= [0.53(\beta/\beta_{x(mc)})^2 1.65D]/(RS)_{mc} \\ &= [0.87(\beta/\beta_{x(mc)})^2 D]/(RS)_{mc} \end{aligned} \quad (15)$$

We now apply equations (9), (10), (13), (14), and (15) in appropriate places in the modified Einstein procedure and compute total sediment discharge for selected sites where total sediment discharge has been measured. This permits a comparison between computed and measured sediment discharges.

As previously noted, the data used by Colby (1955) in his comparison of computed and measured total sediment discharge also is used in the current study. He made comparisons for 24 sets of data, and these comparisons are summarized in his table 34. However, only 21 of the 24 data sets were used in the current study. According to Colby (1955), two data sets for the normal sections contained "obviously incorrect suspended-sediment size distribution." These two data sets were not used in the current study. For a third set, Colby (1955) made "separate computations for each of the two parts of the cross section" in computing total sediment discharge for the normal section. The third set was not used in the current study because of the uncertainty of what cross-sectional properties should be used in computing total sediment discharge for the normal section. Of the remaining 21 sets, some of the data used by Colby (1955)--stream width, depth, velocity; concentration of suspended sediment; size distribution of suspended sediment; average depth at the verticals where the suspended-sediment samples were collected; size distribution of bed material; and water temperature--could not be duplicated exactly. Therefore, the values of sediment discharge computed for this study using the modified Einstein procedure do not agree exactly with those contained in Colby's table 34.

COMPARISON OF COMPUTED AND MEASURED SEDIMENT DISCHARGE

The first approximation of the revised modified Einstein method gives results that apparently are as accurate as those obtained using the modified Einstein procedure (fig. 2). Values for the square root of the mean variance, computed for the 21 data sets, indicates that the revised procedure gives better accuracy; however, the indicated improvement is small and is assumed to be insignificant (table 1). It should be noted that the Colby data were used to develop the modified Einstein procedure; therefore, values for the square root of the mean variance for the modified Einstein procedure probably are not a good indicator of the standard error of prediction. Comparisons using data

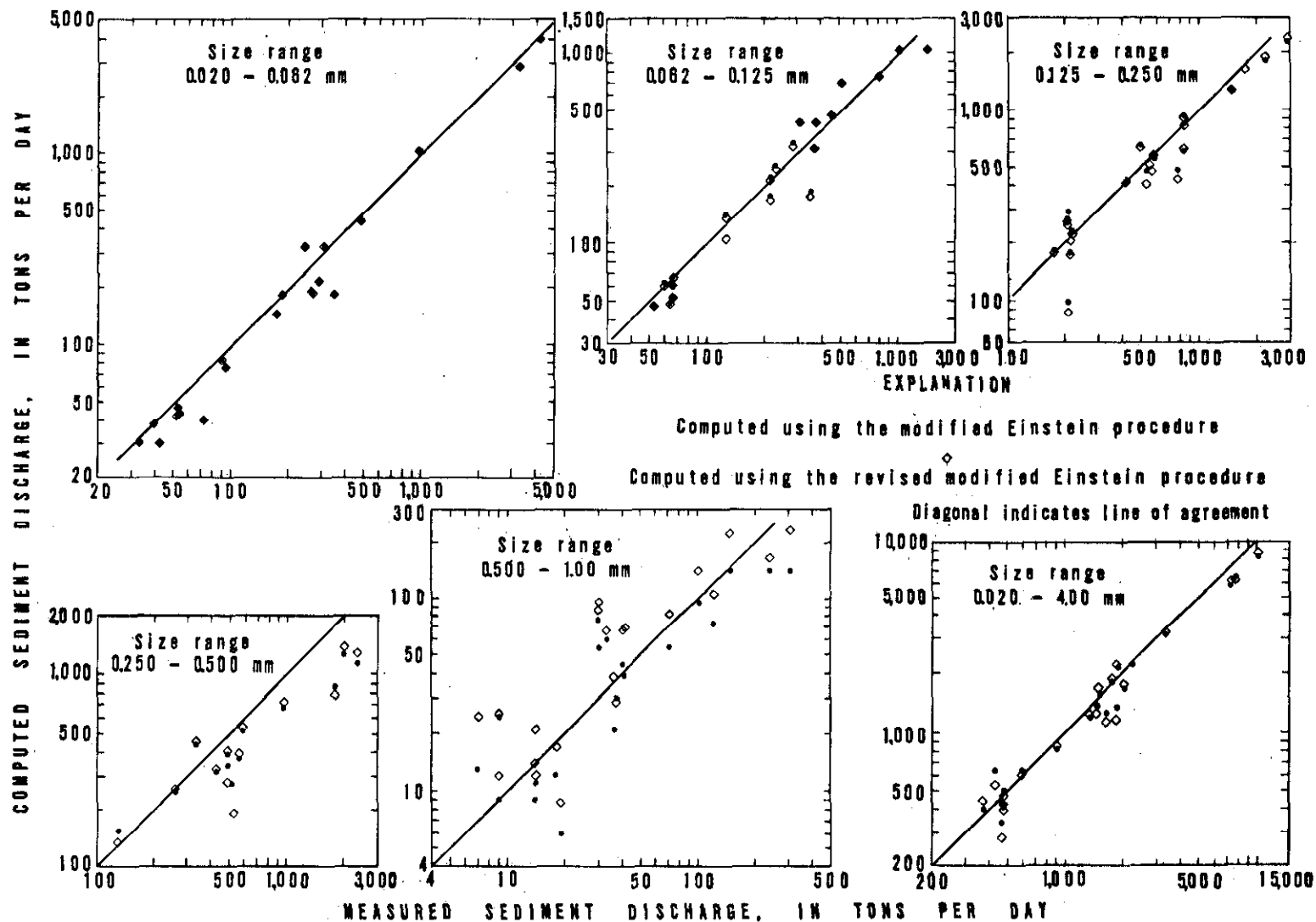


FIGURE 2.--Comparisons of computed sediment discharges from modified and revised modified Einstein procedures applied to normal sections with measured discharge at a contracted section.

Table 1.--Comparison of computed sediment discharges from modified and revised modified Einstein procedures applied to normal sections with measured discharge at a contracted section

Size range	0.020 to .062 millimetres		0.062 to .125 millimetres		0.125 to .250 millimetres	
	Revised		Revised		Revised	
Method	Modi- fied	modi- fied	Modi- fied	modi- fied	Modi- fied	modi- fied
Square root of mean variance for 21 data sets (tons per day) ¹	95	95	107	100	193	182
Mean of measured sediment discharges for 21 data sets (tons per day)	542		338		769	
Ratio of square root of mean variance to mean of sediment discharges	0.175	0.175	0.315	0.295	0.250	0.237

Size range	0.250 to .500 millimetres		0.500 to 1.000 millimetres		0.020 to 4.000 millimetres	
	Revised		Revised		Revised	
Method	Modi- fied	modi- fied	Modi- fied	modi- fied	Modi- fied	modi- fied
Square root of mean variance for 21 data sets (tons per day) ¹	388	367	45	40	674	611
Mean of measured sediment discharges for 21 data sets (tons per day)	586		63		2,170	
Ratio of square root of mean variance to mean of sediment discharges	0.661	0.625	0.718	0.633	0.310	0.281

¹Variance for each of 21 data sets computed as difference between computed and measured sediment discharge.

not used in developing either method are needed in order to evaluate the relative sizes of the standard error of prediction for the two methods.

Both the modified procedure and the revised modified procedure apparently are less accurate for the ranges of large particle sizes than they are for the ranges of small sizes; this premise is based on the fact that the variability ratios--ratio of the square root of the mean variance to the mean of the sediment discharges--are smaller for the ranges of the relatively small particles than they are in the ranges of relatively large particles (table 1).

Based on the premises that $5.5D_{65}$ correctly represents k_s in Keulegan's equations--equations (1) and (2), that equation (1) represents the vertical velocity distribution at a point, and that equation (2) represents average velocity at a cross section, the conclusion is reached that the flow in the Niobrara River near Cody, Nebraska, was turbulent and the channel was rough during the periods when the 21 data sets were obtained. This conclusion is based on the fact that the x values for the 21 data sets for the revised procedure differ only insignificantly from 1, and an x of 1 indicates turbulent flow and a hydraulically rough channel. The x values for the 21 data sets for the modified Einstein procedure ranged from 1.50 to 1.62.

The shear velocity, $\sqrt{(RS)_{mc}} g$, for the 21 data sets for the revised modified procedure is significantly larger than the shear velocity, $\sqrt{(RS)_m} g$, for the modified procedure. The relation between the two shear velocities can be represented by:

$$(RSg)_{mc}^{\frac{1}{2}} = \left(1 + \frac{4.26}{V}\right) (RSg)_m^{\frac{1}{2}} \quad (16)$$

in which 4.26 is $5.75 \log x_0$ and x_0 is 5.5. The mean velocity for the 21 data sets ranged from 1.82 to 4.39 feet per second; therefore, the ratio $(RSg)_{mc}^{\frac{1}{2}} / (RSg)_m^{\frac{1}{2}}$ for the data sets ranged from 1.97 to 3.34.

SUMMARY AND CONCLUSIONS

The modified Einstein procedure (Colby, 1955) is revised by applying corrections to equations (5) and (6), and by replacing equations (7) and (8) with equations (14) and (15). The revision was needed because D_{65} was used directly in the modified Einstein procedure to replace the equivalent sand roughness, k_s , in Keulegan's velocity equations--equations (1) and (2)--whereas, $5.5D_{65}$ apparently should have been used (Burkham, 1975). The revised procedure eliminated the need to arbitrarily divide the bedload transport intensity, Φ_*^1 , by 2, which was done in the modified Einstein procedure (Colby, 1955). For the same data, the revised procedure gives shear velocities that are significantly larger and x values that are significantly smaller than the corresponding values computed using the modified Einstein procedure. The revised

¹The reader should refer to a report by Einstein (1950) to resolve questions about Φ_* .

modified Einstein procedure gives computed sediment discharges that apparently are as accurate as the sediment discharges computed using the modified Einstein procedure. The test for the relative accuracy of the two methods was made using data that had been used in developing the modified Einstein procedure; more tests are needed using data that have not been used in developing either method in order to correctly evaluate the relative magnitudes of the standard errors of prediction.

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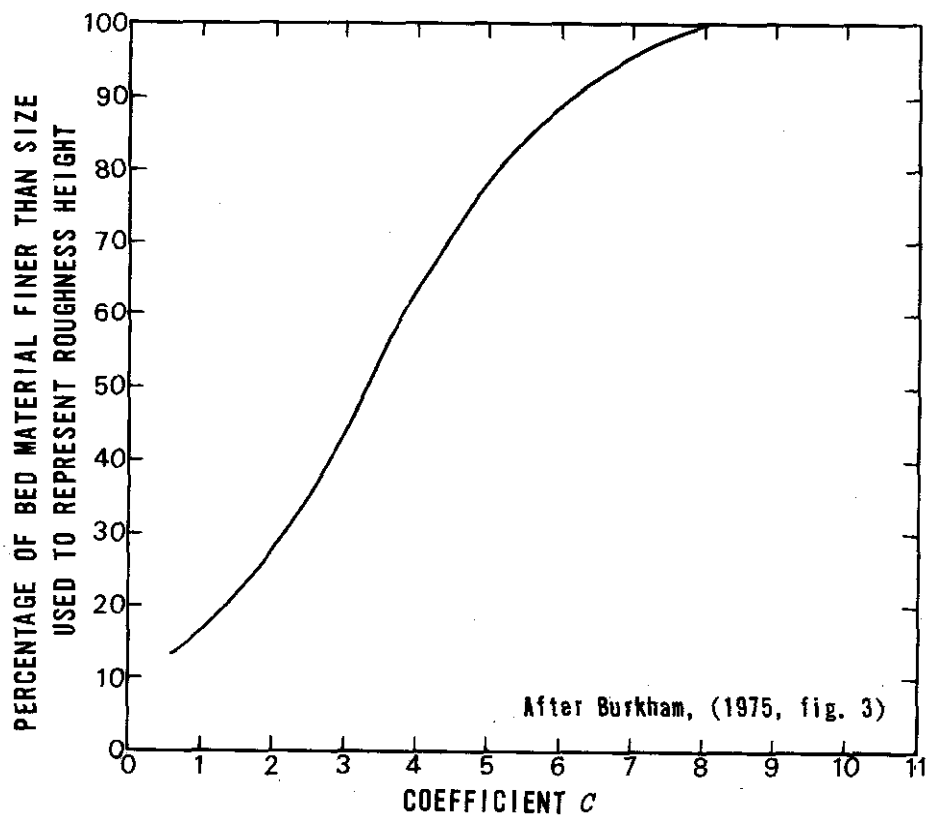


FIGURE 1.--Relation between percentage of bed material finer than size used to represent roughness height and the coefficient C in the Keulegan equation;
 $V/V_* = (C - 0.9b) + 5.75 \log R/k$.

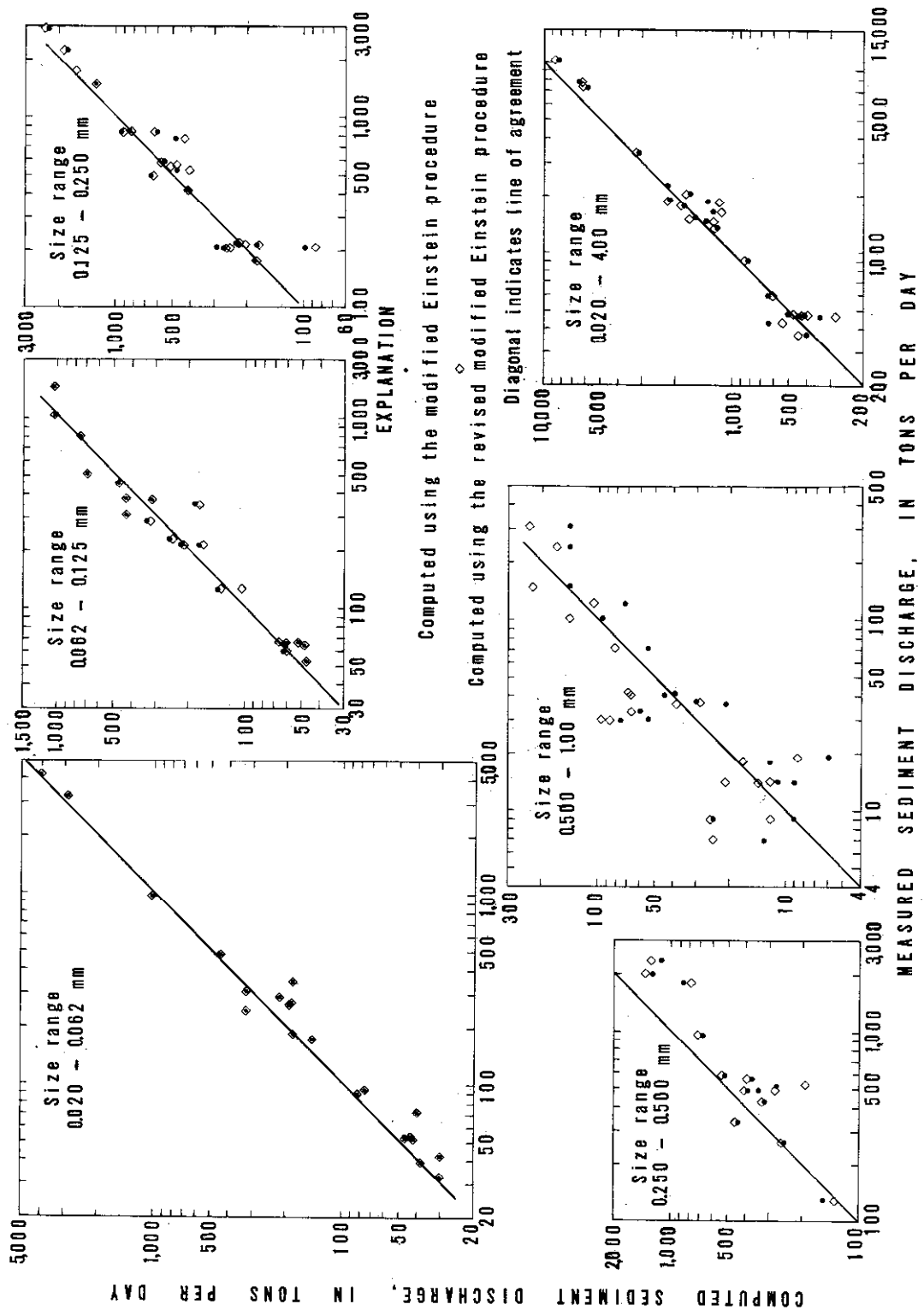


FIGURE 2.--Comparisons of computed sediment discharges from modified and revised modified Einstein procedures applied to normal sections with measured discharge at a contracted section.

LAKE POWELL SEDIMENTATION SURVEYS

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ABSTRACT

Glen Canyon Dam, major unit of the Colorado River Storage Project, is located on the Colorado River near the Arizona-Utah border. Storage in the 27,000,000 acre-foot reservoir began in 1963. Preconstruction studies indicated that sediment would accumulate at an average rate of 85,400 acre-feet per year over a 100 year period. In 1968, 1970, and 1973 the Bureau of Reclamation conducted limited, reconnaissance grade surveys in the delta areas of the reservoir in order to monitor the sediment build-up and its relationship to recreational uses on the lake. Deltas in the two major tributary arms of the reservoir were found to be developing approximately as expected, with their configuration, at this early stage in the reservoir's life, strongly influenced by the original channel topography and the rising water surface. Relatively large delta build-ups have been noted in some of the smaller tributary canyons. Experience and data gained during these limited surveys will be put to use in laying out a monumented sediment range network when the time comes for the first complete volumetric survey of the reservoir. Baseline data for the volumetric survey will come from existing large scale, small contour interval topographic maps of the reservoir basin.

INTRODUCTION

The Colorado River Storage Project provides for the comprehensive development of the Upper Colorado River Basin. The project furnishes long-time regulatory storage needed to permit states in the Upper Basin to meet their flow obligation to the Lower Basin as defined in the Colorado River Compact, and still utilize their apportioned water. Figure 1 shows the locations of the reservoirs and principal geographic features of the Upper Colorado River Basin.

Glen Canyon Dam, located 15 miles upstream from Lees Ferry, Arizona, is the key feature of the Colorado River Storage Project. It has been assigned the task of providing final regulation of the Colorado River before the stream leaves the Upper Basin. It is also an important source of hydroelectric power production. Construction of the dam was initiated in 1956 and completed in 1963. Closure was made in March 1963. Since that time the reservoir has been operated to meet initial filling

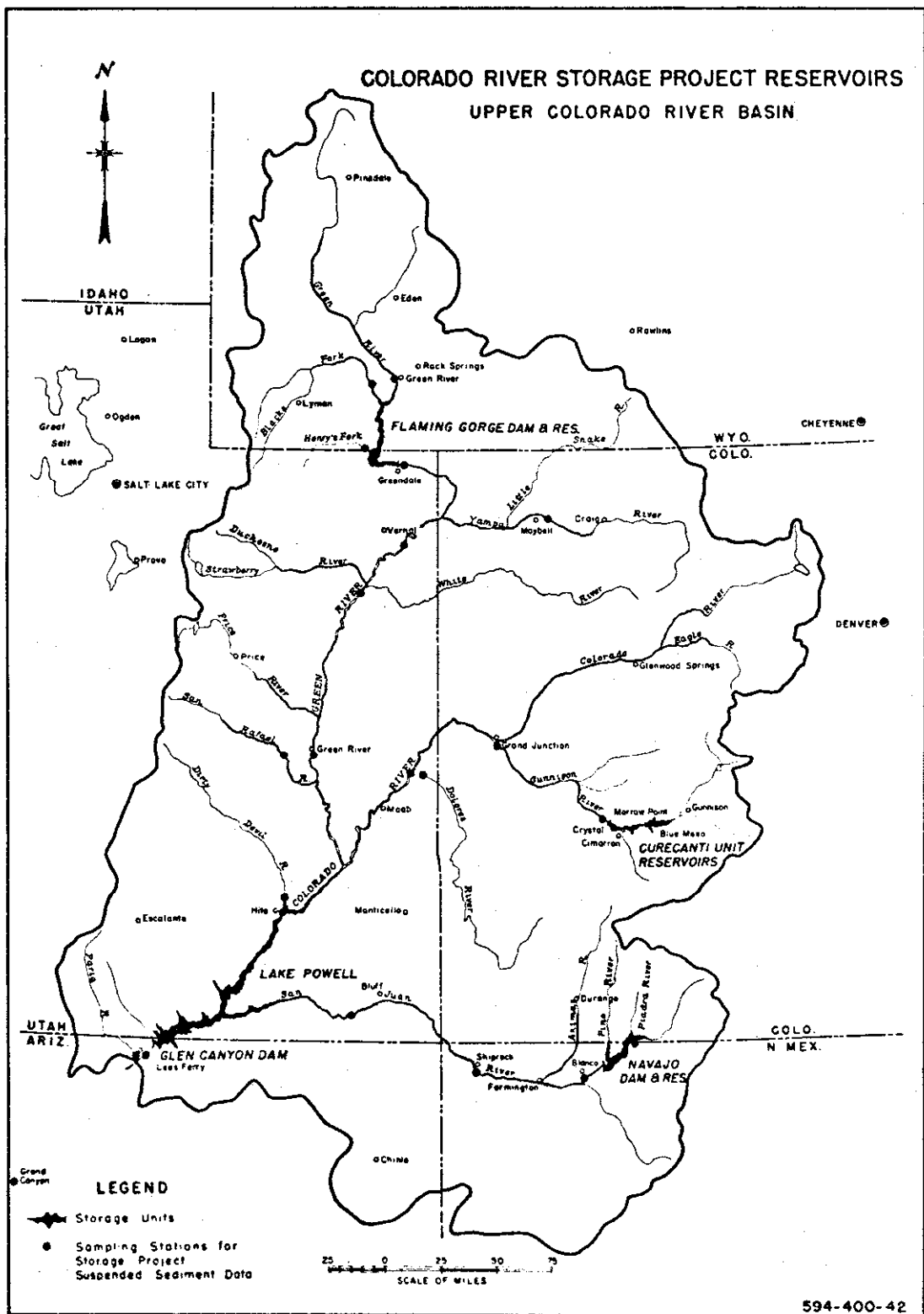


Figure 1.—Reservoirs and principal geographic features of the Upper Colorado River Basin.

criteria and, except for limited periods of drawdown, the water surface has consistently risen. During the summer of 1975, an all time high elevation of 3,675 feet was reached, just 25 feet below normal water surface elevation. In years to come, the reservoir will be operated to meet downstream delivery requirements in a manner that will optimize power production and provide flows for recreational boating through Grand Canyon. During its lifetime, the reservoir will experience cycles of filling and drawdown that will cover many years, similar to the pattern of operations at Lake Mead.

Lake Powell, the reservoir behind Glen Canyon Dam, is one of the remarkable reservoirs of the world. Its desert setting in the sandstone gorges of the Colorado and San Juan Rivers, primarily in southeastern Utah, gives it an unsurpassed beauty. When full to normal pool it will reach 186 miles up the Colorado River to Cataract Canyon and an additional 71 miles up the other principal tributary, the San Juan River, to the vicinity of Grand Gulch. See Figure 2 for a map of the reservoir. Total initial storage capacity at normal water surface elevation of 3,700 feet m.s.l. is 27 million acre-feet. The bulk of the reservoir is confined in the narrow canyons of the San Juan and Colorado Rivers and local tributaries except at the lower end between Wahweap and Last Chance Creeks, the Halls Creek-Bullfrog area, and the Hite reach where large basins are forming. This configuration produces a 1,940 mile shoreline.

Provision for sediment storage was a major consideration in the planning, selection, and design of the principal reservoirs of the Colorado River Storage Project. A basic concept adhered to was that a specified minimum active storage content should remain at the end of a 200 year period. The largest sediment load affecting any of the project reservoirs is that carried by the Colorado and San Juan Rivers into and stored in Lake Powell. Preconstruction estimates of long-time sediment accumulation were made for Lake Powell based on extensive suspended sediment records which date back as far as 1925. Allowing for upstream depletion by the other Storage Project reservoirs, it was estimated that an average of 104,000 acre-feet of sediment would flow into Lake Powell annually. Taking compaction into account, the estimated 100 and 200 year accumulated volumes are 8,540,000 and 16,800,000 acre-feet, respectively. The latter figure is 62 percent of the total initial capacity. A more complete description of the sediment storage study will be found in the article by Gessel (1963).

It is usual Bureau of Reclamation practice to establish a network of monumented and surveyed sediment ranges in a reservoir basin prior to initial filling for those reservoirs where long-time sediment accumulation is likely to be a significant portion of the initial total capacity. However, Lake Powell presented special problems in carrying out this policy due to its large size, the several hundred miles of main channels and tributaries, the extremely rugged terrain surrounding the reservoir, the vertical walls along the

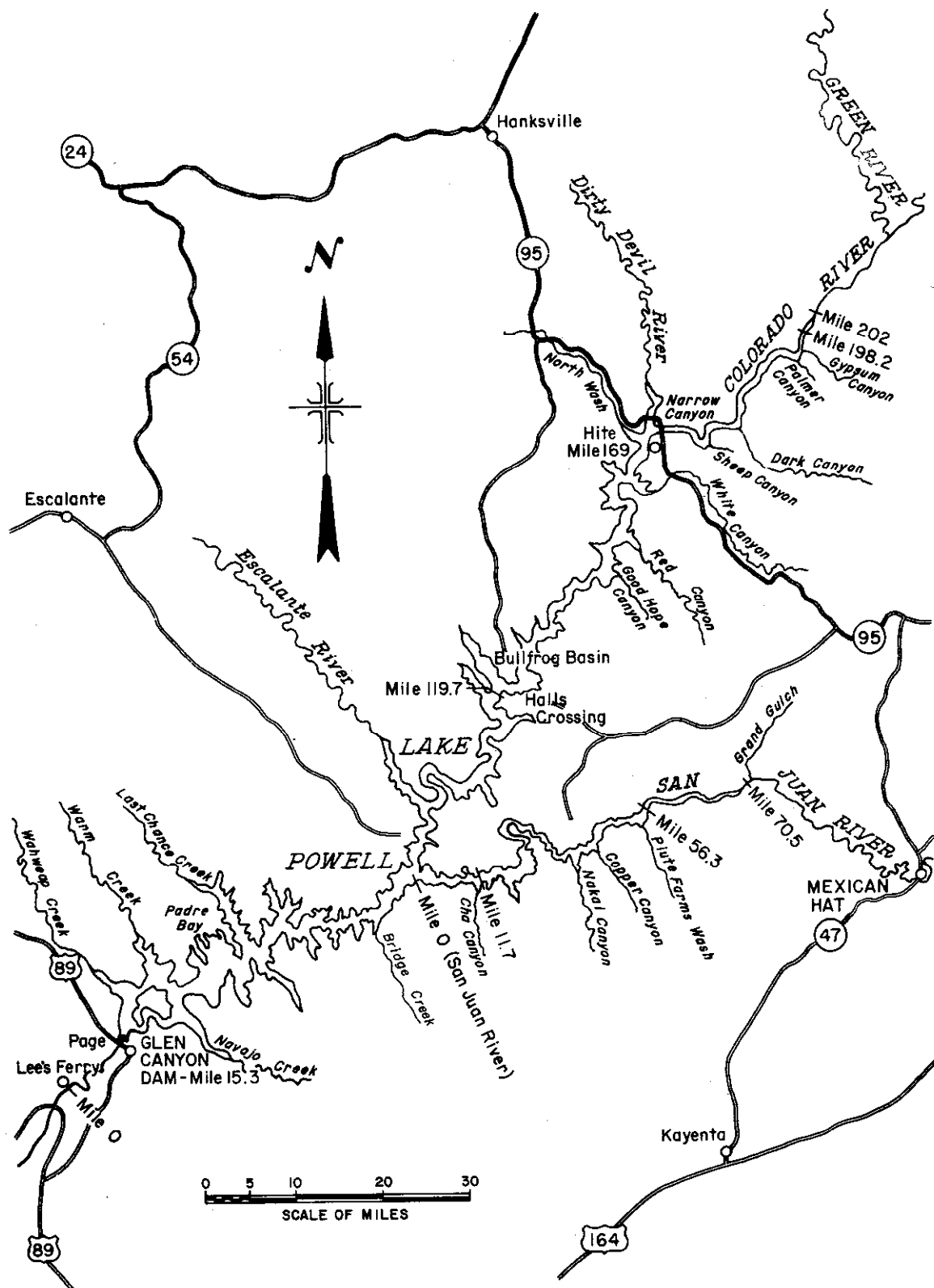


FIGURE 2 - Lake Powell showing principal tributaries and pertinent mileage points.

bulk of the channel, the swift flowing water of the natural stream, and the lack of a high density network of horizontal and vertical control.

Several alternative plans were considered for gathering the baseline data necessary for later measurement of sediment accumulation. After careful consideration of the costs of the various schemes, the decision was made to remap the reservoir basin at a large scale and small contour interval. Establishment of the sediment range system was to be deferred until later when the reservoir was storing water and operating personnel could work from the surface of the lake. The map would be used to define initial cross sections at each range once the network had been established. The mapping scheme would also offer more freedom in measuring sediment accumulation at unrange locations on the lake. (An important by-product of the mapping scheme was the accurate determination of area and capacity data.)

Through the cooperative efforts of three agencies of the Federal Government and several private firms, a high resolution topographic map of the reservoir basin was produced. Map coverage of the Colorado River portion of the reservoir is at a scale of 1 inch equals 400 feet with a 10 foot contour interval. The San Juan River arm of the reservoir was mapped at a scale of 1 inch equals 1,000 feet with a 20 foot contour interval. In all, the reservoir basin map covers 353 large sheets. A more complete description of the mapping job is described in the article by Gessel and Rutledge (1962).

No complete sediment surveys of Lake Powell have yet been made by the Bureau, due to the doubtful value that revised area-capacity information would have this early in the life of the reservoir.^{1/} The establishment of a sediment range network and the conduct of a comprehensive volumetric survey are tentatively scheduled for 1979 and 1980. Due to normal questions that have arisen since storage began about the distribution and volume of sediment deposit (particularly in its relationship to recreational aspects of the lake), limited reconnaissance-type surveys have been and will be conducted every three years in order to monitor changes that are occurring. These surveys will also provide experience and data which will be of value in the establishment of the permanent range network later. The conduct and findings of these limited surveys will be the subject of the remainder of this paper.

^{1/} Independent sedimentation studies and surveys were initiated in 1971 by the Lake Powell Research Project, a consortium of academic institutions funded by the National Science Foundation. They intend to prepare estimates of total and average annual rates of sediment deposit.

CONDUCT AND FINDINGS OF THE SURVEYS

1968 Survey

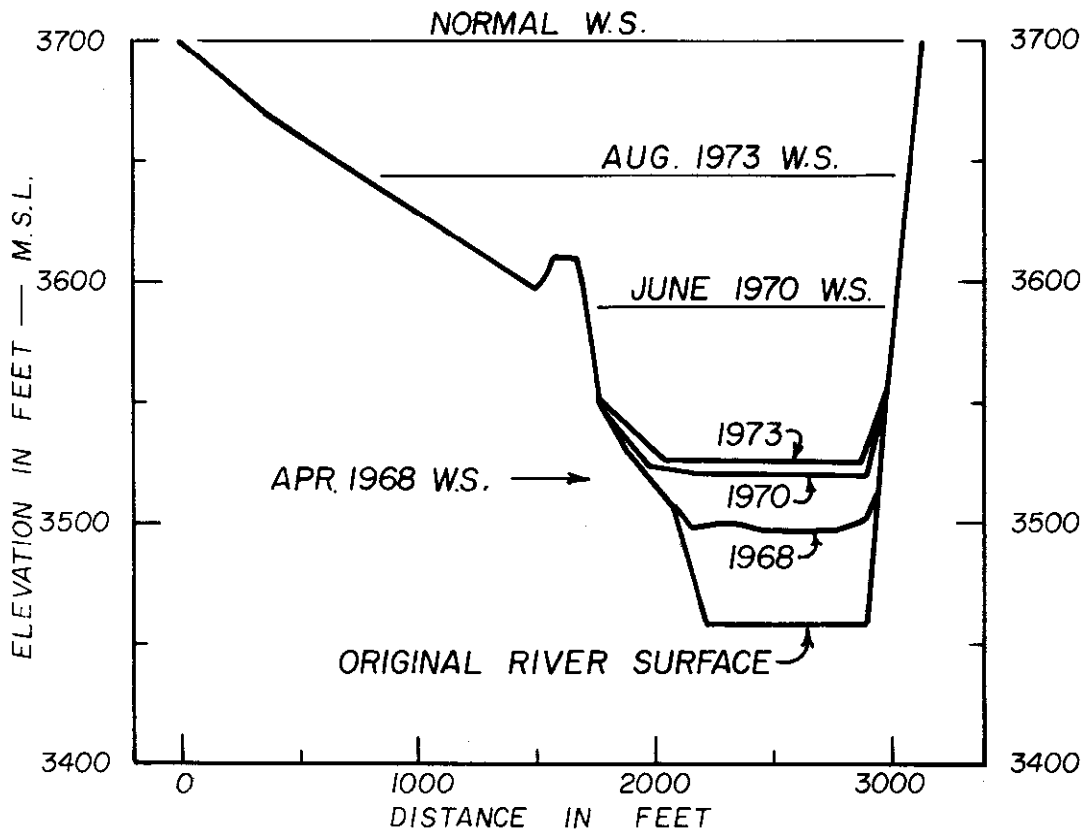
In April 1968, twelve cross sections were surveyed on the Colorado River arm of the reservoir between Mile 145.5 and Mile 169.6 (original river mileage above Lees Ferry, Arizona). This particular reach was chosen because it encompassed the stretch of reservoir where delta build-up was known to be occurring. The area begins at Good Hope Canyon and ends just above the Dirty Devil River. The reach includes an important recreational complex at Hite, Utah (Mile 169) which provides one of the few areas of boat access to the lake.^{2/} Water surface during the survey stood at elevation 3,518 feet m.s.l. and the reservoir extended upstream to Mile 181 between Sheep and Dark Canyons.

Locations for the cross section surveys were tentatively selected from U.S.G.S. quadrangle maps with final location selection made in the field on the basis of ease of relocating them on later surveys. Cross sections were not monumented but were carefully located on the large scale reservoir basin topographic maps and referenced to prominent landmarks. Terminal points of each cross section were documented with photographs.

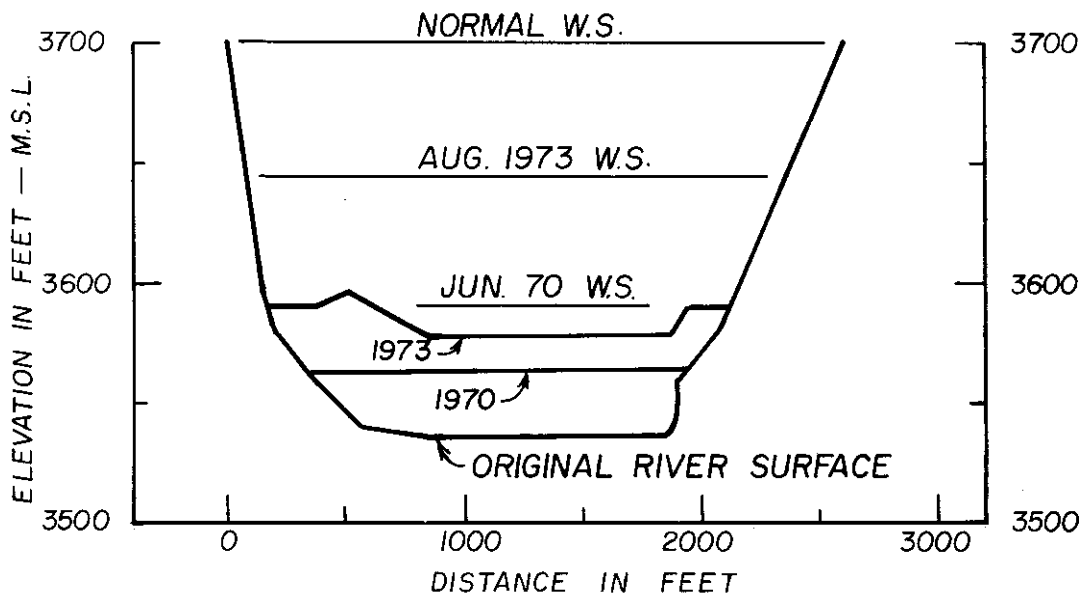
Depths from water surface to top of sediment deposit were determined using a Bendix Depth Recorder DR-23 mounted on a 31-foot cabin cruiser. The depth recorder records the depth soundings on a 2 inch paper tape. Five different scale settings of up to 100 fathoms or 600 feet are available. This range is sufficient to provide data on all of the reservoir. This particular sounder was not designed to be used as a precise surveying instrument, so much of the data presented in this report and its interpretation should be considered as approximate. However, for use in these general monitoring investigations, it has performed satisfactorily. For the most part, the shoreline was against rock cliffs or steep talus slopes, so soundings could begin within 10 or 15 feet from the water's edge. Some difficulties were encountered in maintaining a constant boat speed and keeping a straight line course during the recordings due to wind and water currents and the lack of concise control points.

Original cross sections were developed for each surveyed section using large scale topographic maps. Data from the depth recording tapes was plotted on these with adjustments made where necessary to compensate for the variable speed of the boat. Prominent underwater features aided in plotting the sounding data correctly on the original cross sections. The plots indicated the depth and distribution of sediment at each location. Typical plots are shown in Figure 3. A

^{2/} Preconstruction studies had shown that any major recreational development at Hite would have a comparatively short life due to sediment delta encroachment. Consequently, only temporary facilities are located there now.



COLORADO RIVER—MILE 167.9



SAN JUAN RIVER—MILE 41.2

FIGURE 3-TYPICAL CROSS SECTIONS ON LAKE POWELL

profile of the original Colorado River from Mile 119.7 to Mile 200 was prepared and data from the 12 surveyed cross sections was used to develop a profile of sediment deposit in the reach surveyed. Figure 4 shows a plot of the profiles. Only about 10 feet of deposit was measured at Mile 145, whereas sediment had increased in depth to about 40 feet between Mile 164 and Mile 169. At Hite (Mile 169) a deposit of about 40 feet was indicated. Since the water surface was about 20 feet above the sediment bed at Hite, no recreational problems had developed, except for deposition of fine sediments on the boat ramp due to the turbid waters in the area.

1970 Survey

In June 1970, 19 cross sections were surveyed on the Colorado River arm of the reservoir between Mile 119.7 and Mile 190.6. Water level stood at elevation 3,590 feet, 72 feet higher than in 1968, with the reservoir ending at Mile 196 near Gypsum Canyon. As many as possible of the cross sections were surveyed at the same locations as in the 1968 survey. However, in several instances the higher reservoir level had covered or obscured the reference points documented in 1968 and the cross sections had to be taken at estimated locations. Fortunately, precise relocation proved to be unnecessary due to the gradual slope of the sediment deposit. Except at the mouth of the tributary streams, the topset slope of the delta varies very little from mile to mile along the reservoir bottom.

The profile developed from the 1970 survey is shown in Figure 4. It is based on data derived from the cross section surveys, supplemented by a limited amount of centerline profile soundings. The profile indicates that the zone of greatest sediment depth had moved upstream and was concentrated in a reach extending from North Wash (Mile 167.5) to Dark Canyon (Mile 182.8). Depths in this reach were on the order of 60 to 70 feet. This condition is believed to be the result of the confining nature of the reservoir through Narrow Canyon and the rising reservoir.

During the 1970 survey, 6 cross sections and a few miles of centerline profile were surveyed on the San Juan River arm of the reservoir from Mile 11.68 to Mile 46.7 (mileage measured from original confluence with the Colorado River). A profile is shown in Figure 5. The profile depicts a total accumulation of sediment which varies from approximately 40 feet at Mile 11.68 in the vicinity of Cha Canyon to 15 feet at Mile 46.72 in the vicinity of Copper Canyon.

In addition to surveys on the main stem and San Juan River arm, miscellaneous cross sections and profiles were picked up in the mouths of some of the lesser tributary streams. These measurements indicated that significant deltas were being formed in the tributary canyons as a result of both the backflow from the Colorado River during initial filling and from material deposited by local flood flows. On the Dirty Devil River, sediment accumulation ranged from 60 feet in the

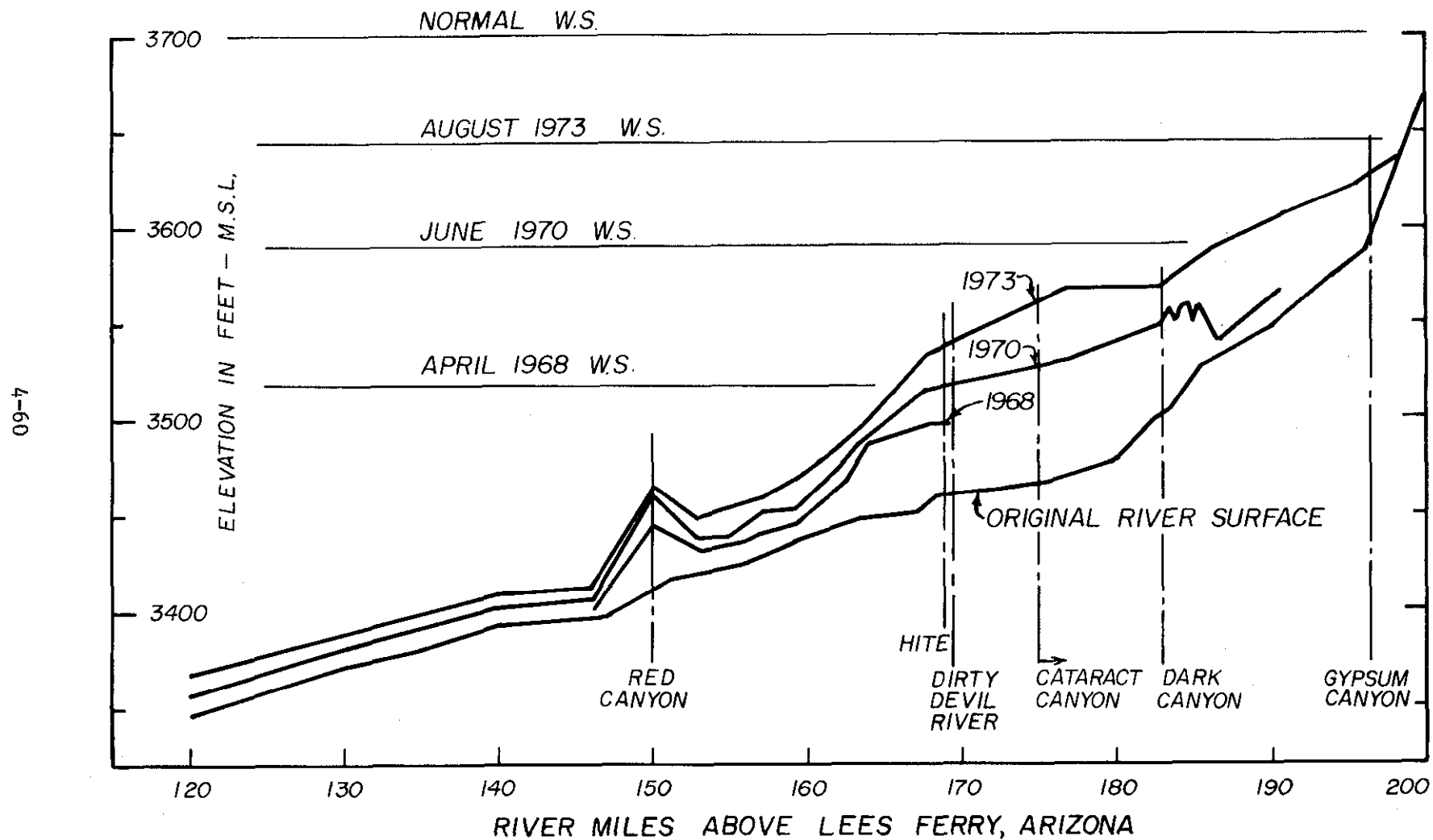


FIGURE 4 - PROFILES ALONG COLORADO RIVER ARM OF LAKE POWELL

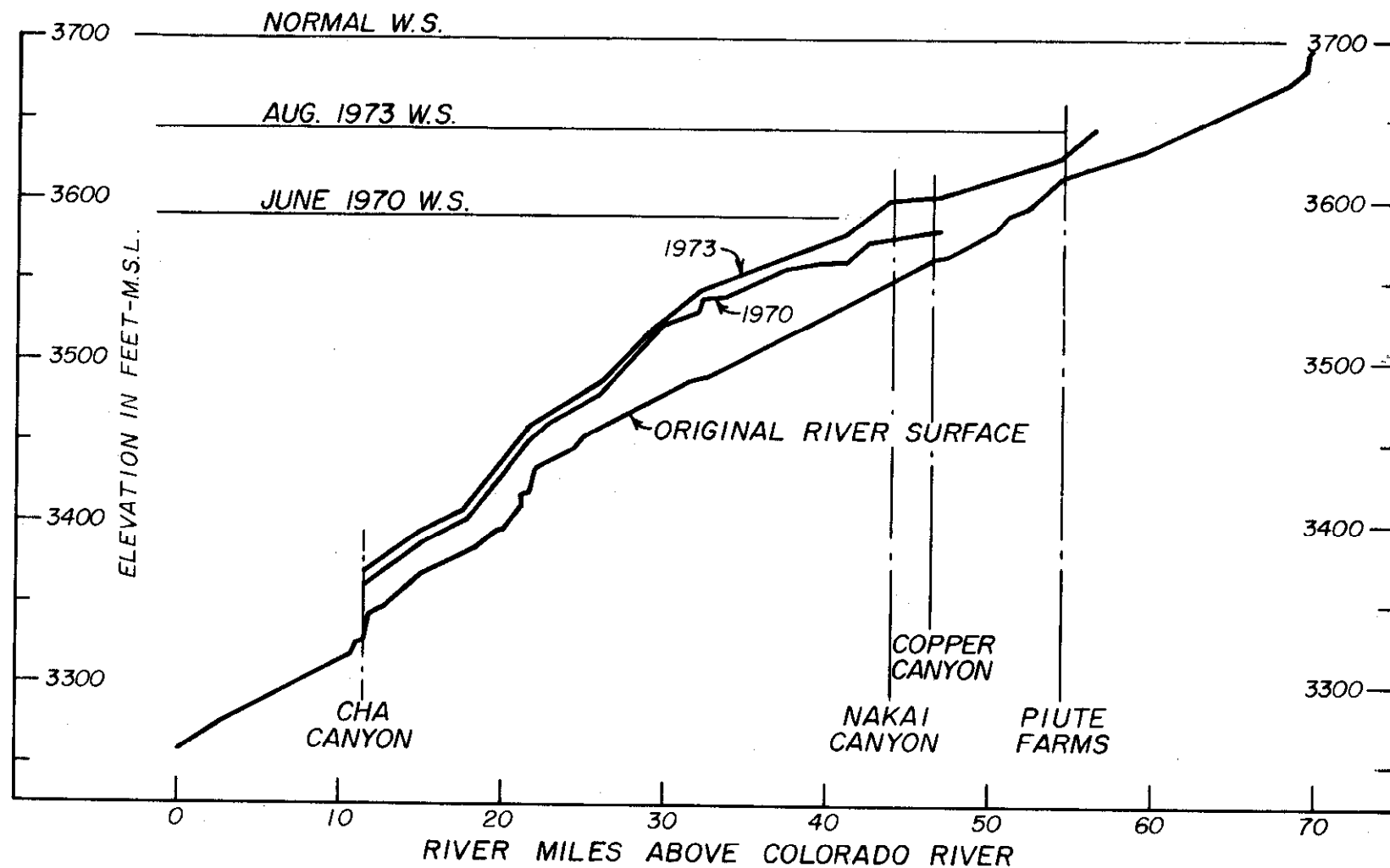


FIGURE 5-PROFILES ALONG SAN JUAN RIVER ARM OF LAKE POWELL

vicinity of the confluence with the Colorado River to 30 feet about six miles upstream on the river.

1973 Survey

In August 1973, water surface elevation had risen to elevation 3,644, 54 feet higher than in 1970 and 126 feet higher than in 1968. Twenty-one cross sections were surveyed on the Colorado River arm, including, where possible, locations that had been surveyed previously. The surveyed reach extended from Mile 119.7 to Mile 198.2. It was found that the greatest accumulation of sediment again occurred between North Wash (Mile 167.5) and Dark Canyon (Mile 182.8) with a maximum depth of almost 100 feet at Mile 177. Only about 20 feet of total deposit was noted at Mile 119.7 and Mile 146.2. Eight cross sections were surveyed on the San Juan arm in a reach between Mile 11.7 and Mile 56.3. Maximum sediment depth appeared to be on the order of 50 feet near Mile 32. Cross section and profile runs on some of the lesser lake tributaries, such as Dirty Devil River, Escalante River, and Last Chance Creek showed considerable delta build-up in the mouths of these streams, in some cases even approaching the existing water surface elevation.

Special Surveillance Studies

In 1974, a special network of sediment ranges was surveyed and monumented on Bridge Creek, a small tributary located about 30 miles northeast of the dam. Bridge Creek is the site of Rainbow Bridge National Monument, the world's largest natural rock bridge. At elevation 3,606 feet, the waters of Lake Powell back up into Bridge Creek and reach the downstream boundary of the monument. At normal water surface elevation 3,700 feet, the reservoir will extend to the upstream boundary and water will stand 45 feet deep directly under the bridge. The sediment range system was installed as part of an extensive geologic surveillance program designed to monitor the effects which the rising lake waters might have on the bridge. The 22 ranges, located in a one mile reach encompassing the bridge and monument boundaries, will be surveyed annually.

In addition to the sedimentation surveys, the Bureau conducts a landslide surveillance program for all reservoirs constructed and operated. Landslides into the reservoirs would have some relevance to total sediment accumulation if they were of major proportions. Periodic inspection of Lake Powell has indicated that talus accumulations of dune sand have been eroded by wave action in numerous locations and the dunes had slipped into the reservoir. Some large ones were formerly located in the Escalante River arm. Some slumping has also developed on the surface of the outcrops of Chinle shale in the Rincon area and up the San Juan River for about 15 miles. The volume from these slides into Lake Powell is small when compared to the volume of the reservoir and the total accumulation of deposited sediment caused by inflow. Also, in most cases the prospective slump areas are already

located in the reservoir basin below 3,700 feet and are merely reworked to a lower level by the action of the reservoir.

CONCLUSIONS

The surveys have shown that the pattern of sediment deposition in Lake Powell is developing approximately as expected. Most of the sediment accumulation so far is concentrated in the delta areas. Average topset slope of the Colorado River delta as measured between Mile 167.9 and Mile 198.2 in 1973 was 3.5-feet-per-mile. Foreset slope between Mile 159.2 and Mile 167.9 was 7.0-feet-per-mile. As a comparison, observations show that the Lake Mead delta has a one-foot-per-mile topset slope and a 100-foot-per-mile foreset slope. The delta on the San Juan River arm is more uniform in thickness and closely follows the original streambed profile. At this point in time, both delta configurations appear to be strongly influenced by the original channel topography and the rising water surface elevation. Rough volumetric calculations indicate that the amount of delta material is fairly close to the predicted value.

As the water surface elevation of the reservoir continues to rise, marinas now existing on the reservoir should be able to maintain operation without problems resulting from sedimentation; however, as the reservoir water surface elevation declines, operation of the marinas may become critical depending upon the pattern with which the sediment has deposited within the reservoir for these areas. Potential problem areas will be carefully monitored by both the Bureau of Reclamation and the National Park Service.

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SEDIMENT PROBLEMS IN THE MOHAVE VALLEY - A CASE HISTORY

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ABSTRACT

Sedimentation technology plays an important role in the Bureau of Reclamation's Lower Colorado River Management Program. Public demands also receive major consideration in the decision-making process.

The experience of the Bureau of Reclamation in the Mohave Valley after construction of Parker Dam is discussed. Aggradation above Lake Havasu caused a critical situation near Needles, California, where railroad facilities and other developments were inundated or threatened by the rising river. This prompted passage of Federal legislation and authorized the Bureau of Reclamation to take the needed action.

In its early stages, the river management program centered on reconstruction of the river channel by dredging and related work. It evolved by stages into a comprehensive program which now includes the improvement of the river and related features for fish and wildlife, general recreation, and other uses.

The gathering of sediment and hydraulic data is an essential part of the overall program. Selected information from the data collection program is summarized to illustrate some of the changes brought about. The changing requirements in the Mohave Valley are discussed.

Future management will be governed increasingly by publicly expressed environmental concerns. Methods of assuring appropriate input in program planning such as formation of the Lower Colorado River Management Program Coordinating Committee are described.

INTRODUCTION

The Bureau of Reclamation's responsibility for the management of the lower Colorado River dates from the early 1940's. This management responsibility centers on the 444 kilometers (276 miles) of river between Davis Dam near the southern tip of Nevada and the International Boundary with Mexico, although general authority extends upstream as far as Lees Ferry, Arizona. The following discussion is confined to a history of the management program in the Mohave Valley.

As it flows through the southwestern United States, the Colorado River travels alternately through mountain canyons and alluvial valleys. The Mohave Valley is typical of the river valleys along the lower Colorado River where the alluvial character of the bed and banks provides little

resistance to the meandering and transport of riverbed sediments. The valley is situated in three states: Arizona, California, and Nevada. Principal geographic features are shown in Figure 1. The valley extends south from Davis Dam about 64 kilometers (40 miles) to Topock, Arizona, where the river enters Topock Gorge. The 23-kilometer (14-mile) gorge connects the Mohave Valley with the headwaters of Lake Havasu, the impoundment behind Parker Dam.

The river, which is now largely controlled by upstream impoundments, averages about 425 cubic meters per second (15,000 ft³/s) in the summer and 200 cubic meters per second (7,000 ft³/s) in the winter. Releases from the various dams are made to satisfy downstream demands for irrigation and municipal and industrial water. At the major dams, the released water generates power and both monthly and daily release patterns are adjusted to maximize power generation. Although the reservoirs have removed most of the extreme flow variation of the river, significant floods can occur from both the main stem and tributary watersheds.

The area lies within one of the most severe desert areas in the United States. The climate is hot and dry with annual precipitation averaging only 8 to 13 centimeters (3 to 5 inches). Much of the precipitation occurs from thunderstorms. Maximum summer temperatures range between 46° and 52°C (115° and 125°F). Winters are mild. Low flood plain areas contain heavy growths of salt cedar and arrowweed, while areas of somewhat higher elevation support mesquite and other desert vegetation. Marshes are mainly bulrush and cattail. The Fort Mohave Indian Reservation and the Havasu National Wildlife Refuge occupy most of the valley. Much of the remaining land is undeveloped, although some clearing has been accomplished for agricultural purposes. Considerable recreational development in the form of campgrounds, marinas, and parks has taken place along the river. The largest city is Needles, California, with a population of 4,050.

The "Needles Situation"

Before the construction of storage dams on the Colorado River and its tributaries, the river was wild and uncontrolled. Its regimen was typical of a river carrying a heavy sediment load on an alluvial bed. There was wide meandering in the river valleys accompanied by a general aggradation of the valley floors. The river ranked high on the list of sediment bearing rivers of the world, transporting an average annual sediment load in excess of 145 million metric tons (160 million tons). The regulation of the lower river began with completion of Hoover Dam in 1935. The subsequent construction of Parker Dam in 1938 and Davis Dam in 1950 also had a significant effect upon the river. The most noticeable change caused by the dams was the marked reduction in flood peaks. Also important, however, was the change in sediment characteristics. Most of the sediment carried by the river was trapped in the reservoirs and the relatively clear water released from the dams began scouring the alluvium downstream. The energy formerly expended in transporting the sediment load was now expended in attacking the banks and bed of the river channel. Immediately below Davis Dam in the upper Mohave Valley,

for example, the riverbed has degraded about 5.5 meters (18 feet) since the dam was completed. This scour was most severe near the dam, diminishing progressively downstream for about 40 kilometers (25 miles).

In the lower Mohave Valley, nearly the opposite situation prevailed. Here the river entered the backwater zone above Parker Dam. As the riverflow entered the headwater area of Lake Havasu in Topock Gorge, the river velocity and hydraulic energy diminished with a corresponding loss of sediment-carrying capacity.

Thus the sediment from upstream sources was deposited in the head of Lake Havasu which in turn resulted in increased water-surface elevations upstream. Following the closure of Parker Dam in 1938, the water levels at Topock rose about 1.8 meters (6 feet) within a year and about 3.0 meters (10 feet) by 1944.

During this process, the channel in the lower part of the valley deteriorated. For about 19 kilometers (12 miles) above Topock, the river meandered aimlessly through a series of swamps and sloughs. The situation was described as follows (Vetter, 1944):

"The result of the rise at Topock Bridge, brought about by the filling of Lake Havasu above Parker Dam, was the formation of a pool over the flat, bottom lands upstream. The reduced velocities of flow caused the silt to be dropped, forming extensive sand bars in the former river channel. When the sand bars neared the water surface, tules and willows started to grow, forming an almost impenetrable jungle across the former river channel and river bottom lands.

"Through this mass of vegetation, the water would filter, dropping its silt load as it went. The result was a further rise of water surface upstream, further formation of sand bars, new growth of tules and willows, and so on.

"By 1942, the process had reached half way from Topock to Needles, and by late 1944, it had reached the town of Needles itself. The resultant rise of water surface at Needles has been approximately 1.5 meters (5 feet)."

Nearly 100 dwellings had to be abandoned in the Needles vicinity. The main line of the Atchison, Topeka and Santa Fe Railway was also threatened by the rising waters. The "Needles Situation," as it came to be known, led to the authorization of stabilization work by the Bureau of Reclamation under amendments to the Colorado River Front Work and Levee System Act of March 3, 1925.

Early river work centered on channel rectification, primarily to reduce the water levels in the Mohave Valley. In 1947, the Bureau of Reclamation purchased the 50-centimeter (20-inch) dredge "Colorado." The dredge began to cut a new channel from Needles to Topock on January 31, 1949. The initial cut was completed on April 21, 1951, and

the river was diverted into the new channel on June 25, 1951, causing an immediate drop of 1.5 meters (5 feet) in water levels at Needles. This ended the emergency that existed at Needles by replacing a shallow, meandering channel with one that was both deeper and shorter.

RIVER MANAGEMENT PROGRAM

The river management program is a descriptive title embracing the activities of the Bureau of Reclamation that have grown out of the Colorado River Front Work and Levee System Act and subsequent legislation. The following summarizes the major elements of the management program as it evolved from the emergency conditions at Needles.

Comprehensive Planning

Although the emergency channel work alleviated the Needles situation, it was realized that the effect would be only temporary. Sediment originating in the upper Mohave Valley was still contributing material to the Needles-Topock reach at an unabated rate. The solution to the problem was presented in the first of a series of comprehensive plans for river management (Bureau of Reclamation, 1955). According to the plan for the Mohave Valley, the river in the upper valley was to be controlled and stabilized. This would extend and make permanent the temporary flood protection afforded to Needles. Also, by narrowing and confining the river to a single channel, water losses by evaporation and phreatophyte transpiration would be reduced. Other benefits outlined in the comprehensive plan were flood protection from levee construction and enhancement of recreational facilities. Work began in early 1953 to carry out the objectives of the comprehensive plan.

Data Gathering

In order to arrive at a comprehensive plan for a stable, controlled channel, specific information and data had to be gathered. A series of river ranges or cross sections were established during the period 1935-1940 to be surveyed on a recurring basis. From these, the pattern of scour and fill was monitored. Present surveys are conducted annually at the 86 ranges between Davis Dam and Lake Havasu. Beginning in 1955, a sediment sampling program has been used to monitor sediment movement throughout the river system. Suspended sediment data (since 1955) and total sediment load data (since 1964) are taken at three locations relating to the Mohave Valley: Needles Bridge near Needles, California, and River Sections 41 and 43 in the Topock Gorge. Total sediment load at these locations is computed by the Modified Einstein Procedure (Colby, 1955). Suspended and total sediment load data at Needles and in the Topock Gorge are compared in the table on the following page.

Average Annual
Sediment Concentrations
Mohave Valley

Year	Suspended Load, ppm		Total Load, ppm	
	Needles Bridge	Topock Gorge RS-41	Needles Bridge	Topock Gorge RS-41
1956	249	249		
1957	243	239		
1958	310	393		
1959	392	167		
1960	222	118		
1961	87	83		
1962	101	72		
1963	92	64		
1964	66	47	123	64
1965	58	50	127	70
1966	60	41	132	57
1967	86	50	161	46
1968	86	28	159	44
1969	79	33	141	52
1970	88	36	146	55
1971	68	23	97	34
1972	70	24	115	38
1973	68	26	144	36

The difference in sediment transport between the two locations is attributable to the Topock Settling Basin which is discussed in the next section.

The data gathering program has revealed considerable information about the nature and amount of sediment movement. It has been determined that 75 percent of the total sediment transport occurs in the period March through August, the period of highest river discharge. Peak sediment rates occur in April and again in July. The bedload, amounting to 20-50 percent of the total load, is composed of uniformly graded fine to medium sand in the form of traveling bars.

Most importantly, the data gathering program has provided a means of quantifying changes in hydraulic characteristics and gauging the effect of the river management program. For example, a comparison of recent sediment data in Figure 2, shows that the completion of channelization and stabilization of the Mohave Valley in 1960 has drastically reduced the quantity of sediment entering the lower valley. A trend line based on sediment data collected since 1961 (also shown in Figure 2), indicates that the rate of sediment transport at Needles is decreasing slowly and has reached a level of about 765,000 cubic meters (one million cubic yards) annually.

Topock Settling Basin

After dredging in the Needles to Topock reach was discontinued in 1951, sediment deposition downstream from Topock caused a rise in the water

surface at Topock. This rise moved upstream to Needles where it amounted to 0.6 meter (two feet). Thus, aggradation in the Topock Gorge remained a problem. Although the upstream channelization and stabilization program would eventually control sediment originating from the river's banks and bed, it was not possible to estimate the duration of sediment production from this source before equilibrium was reached.

Also, another source of sediment became apparent--the steeply sloped tributary washes. Infrequent floods of high intensity resulting from cloud bursts over the relatively barren, impermeable terrain were capable of carrying great quantities of material into the river. This sediment source would be very difficult if not impossible to control.

The Topock Settling Basin was conceived to retard the aggradation from the tributary washes and, until they were stabilized, the banks and bed of the river. Construction of the Topock Settling Basin by dredging began August 1955 and was completed April 1956. It consists of a 6.4-kilometer (4-mile) long reach of the Colorado River which has been widened and deepened to form a sediment trap. The basin, located at the upper end of Topock Gorge, effectively prevents sediment from being carried through the gorge and the upper reaches of Lake Havasu where it can cause further aggradation. When it is empty, the settling basin is capable of trapping about 90 percent of the entering material. High trap efficiencies can be maintained up to an effective capacity of about 1.53 million cubic meters (2 million cubic yards). The accumulated material is removed periodically with a 30-centimeter (12-inch) cutter-head suction dredge. Since it is expected that about 765,000 cubic meters (1 million cubic yards) of sediment will enter the basin annually, it cannot remain unattended for longer than a two-year interval.

Following its construction in 1956, the settling basin was maintained in an essentially continuous operation until late 1967. Only recently has the quantity of sediment diminished to where it is no longer necessary to continually maintain the basin. During its existence, the Topock Settling Basin has intercepted over 15 million cubic meters (20 million cubic yards) of sediment from the upper Mohave Valley. It remains a vital tool in today's river management program.

Topock Gorge

A comprehensive plan covering the Topock Gorge (Bureau of Reclamation, 1967) was approved that would protect the completed upstream work against rising levels in the gorge. Additionally, it would provide water salvage, sediment control, recreational, and fish and wildlife benefits. A key feature of the plan was the dredging of 9.2 kilometers (5.7 miles) of river channel.

Channel stabilization of the lower Colorado River generally takes two forms: one involves the control of channel meander and width by use of armored river training structures such as jetties and groins; the other a dredge channel with riprap bankline stabilization. It was the latter method that had generated most of the public concern about the

management program. The reasons for the concern stemmed from the initial wasteland-like appearance of the dredge spoil, the reduction of certain types of wildlife habitat, the artificial appearance of the constructed channel, and a popular belief that dredging always produced significant negative environmental impacts. Responding to changes in public opinion, the Secretary of the Interior issued a directive which suspended the dredging in Topock Gorge in 1968, shortly after it had begun. The Fish and Wildlife Service and the Bureau of Reclamation were instructed to evaluate the physical and ecological changes taking place in the gorge before any further work was done.

This decision brought to a close the major river rectification work connected with the Mohave Valley. Since that time, dredging has been conducted primarily for maintenance or for purposes other than channel rectification.

Topock Marsh

About 1,600 hectares (4,000 acres) of the lower Mohave Valley is covered by Topock Marsh. The marsh was created between 1938 and 1940 as a direct result of the filling of Lake Havasu and the subsequent aggradation in the Topock Gorge and lower Mohave Valley. The potential of the new marsh and its surrounding area for fish and wildlife was quickly recognized and the 16,600-hectare (41,000-acre) Havasu National Wildlife Refuge was created in 1941.

The marsh's value as a wildlife refuge is directly related to a water level which must be held near a constant elevation. This level was initially dependent upon the existence of Lake Havasu and the related backwater effect through Topock Gorge. In 1951, it was feared that the marsh would be drained because of the river channelization activity.

Plans were developed to protect the marsh from loss of water by the construction of protective features. These features, which were undertaken as part of the river management program, included a 6.6-kilometer (4.1-mile) dike to stabilize and maintain the water levels in the marsh and a 2.8-m³/sec. (100-ft³/s) capacity inlet channel to divert fresh water into the marsh from the Colorado River. Flow circulation through the marsh is controlled by an inlet structure at the diversion point and an outlet structure in the dike. A system of dikes and channels is currently being constructed through the marsh to improve the water circulation and otherwise aid in refuge management.

The refuge and, in particular, the marsh provides habitat for a countless variety of fish and wildlife. It creates a larder for migrating geese and shelters herons, egrets, and the endangered Yuma clapper rail.

Broadening the Program

The expanding use of the Colorado River for recreation and related pursuits has made it apparent that preservation and enhancement of fish and wildlife, recreation, and other values be given full consideration along with other objectives of the program.

The lower Colorado River provides a unique opportunity for recreational enjoyment because it supports a contrasting wetlands environment as it courses through a land of desert terrain and vegetation. A rich variety of recreational pursuits is within range of large southern California population centers. Features of the management program have vastly improved boating, water skiing, and related recreational usage of the lower Colorado River. The management program has also been directly involved in the planning and construction of public parks and marinas along the river.

The previous discussion covering the Topock Marsh illustrates the program's participation in the preservation and enhancement of the fish and wildlife resources. The area contains habitat for many species of wildlife including one that is rare and endangered. The Rare and Endangered Species Act directs Federal agencies to make certain their resource development programs do not jeopardize the continued existence of endangered or threatened species. A current goal of the management program is to incorporate measures that will enhance the habitat of such species. This is evidenced by participation in the recent habitat improvement program in Topock Marsh.

Historically, the fishery of the lower Colorado River included the hump-backed sucker, Colorado River squawfish, and bony-tailed chub. No game species were native to this portion of the river. The construction of dams and the reduction of sediment have allowed the introduction of more desirable species such as bass and catfish. Fishing is now a major recreational activity. Proposals are being studied for the improvement of additional habitat areas along the river for fish as well as wildlife.

Environmental considerations are now of prime importance. The National Environmental Policy Act directs the thorough and systematic evaluation of the impacts of Federal actions and requires an environmental impact statement to be filed for major actions significantly affecting the quality of the human environment. Although most of the work in the Mohave Valley was completed prior to the National Environmental Policy Act, environmental factors were recognized in the earlier comprehensive river planning.

Environmental and other concerns have brought about increased public and interagency interest. As the program grew in scope and complexity, it

became apparent that an expeditious means of obtaining proper inter-agency input and review would be valuable in the program's planning process. The river serves as the political boundary for three states, thus compounding the coordination problem. In 1971, the Lower Colorado River Management Program Coordinating Committee was formed. This advisory group is made up of representatives from the Federal Government and the State Governments of Arizona, California, and Nevada. Fish and wildlife, recreation, land management, Indian, and water user interests are represented. Temporary membership is extended to local agencies and other interested groups as the need arises.

The regular meetings of the Committee and its Work Group provide a forum for the discussion of proposals, the exchange of information, and early input in the planning process. This approach has been very successful. The Lower Colorado River Management Program Coordinating Committee is certain to play a more important role in future river development.

Summary

The initial program of limited scope has evolved into a broad, multi-purpose program. Over 51 kilometers (32 miles) of river channel in the Mohave Valley have been modified and stabilized. The sediment load has been cut to less than 25 percent of preprogram levels. The program is credited with saving 80 million cubic meters (65,000 acre-feet) of water annually through the elimination of excess phreatophytes and open water surfaces. Local protection from flooding is afforded by channelization and the associated levee system. Countless opportunities have been created for a wide variety of recreational pursuits including boating, fishing, hunting, camping, swimming, picnicking, and sightseeing. Direct assistance has been given to the development of recreational facilities such as public parks and marinas. Particularly noteworthy is the program's assistance in the preservation and improvement of the 1,600-hectare (4,000-acre) Topock Marsh.

Data collection has also expanded in scope. Contemporary planning requires factual information about fish and wildlife resources, recreational use and environmental impacts. Recent cooperative field surveys have resulted in a better understanding of the endangered Yuma clapper rail. A number of special studies of environmental resources are under contract to universities and fish and wildlife resource agencies. These will provide additional information on fish and wildlife resources, recreational use, and the environment. This data gathering program rivals the earlier hydraulic and sediment data gathering program in scope and importance illustrating the overall direction that the program has recently taken.

LOOKING AHEAD

The Bureau of Reclamation's responsibilities under the Lower Colorado River Management Program have undergone significant changes in objectives based upon changes in policies, laws, and directives. Many of these changes have resulted from the needs of groups and agencies

directly concerned with the river, others from public attitudes and opinion. At its inception, almost 30 years ago, the goals of the river management program were to prevent flooding and destruction of land occasioned by the river's meandering and deposition of sediment. As the program progressed and demands for the river's water grew, the water salvage aspects of the program gained importance. More recently the expanding utilization of the Colorado River for recreational pursuits made it apparent that enhancement of fish and wildlife and recreational values warrant full recognition in the management program. Finally, legislation has directed a careful and appropriate review of the environmental impacts. The basic multipurpose objectives of the management program now include river stabilization, flood and sediment control, water salvage, fish and wildlife enhancement, recreation development, and protection of the environment.

The basic philosophy of the present river management program is to account for each of the various objectives. While one or more of the objectives may be in direct conflict with others, the Bureau of Reclamation must provide stewardship so that one type of interest is not maximized to the degree that other worthwhile uses are excluded.

Emergency conditions no longer exist in the Mohave Valley. However, the basic management responsibilities remain. Future actions will be guided by publicly expressed environmental concerns. Development and/or preservation of fish and wildlife habitat will continue to have public support. Care will be taken to preserve endangered species. To accomplish this, some aspects or objectives of the management program may have to be deferred or de-emphasized. Eventually, however, some of the more traditional uses of the river will take on renewed importance. Increasingly, people are turning to the Colorado River for water supply and recreation. The competition for the limited resource is expected to be tremendous.

It is clear that sedimentation technology will play a continuing role in river management. Sediment movement will be constantly monitored. Maintenance activity, including the periodic cleaning of the Topock Settling Basin, will continue to be carried out.

Under the guidance of the Lower Colorado River Management Program Coordinating Committee, river management will continue to be directed toward meeting a broad array of multipurpose objectives with participation by all interests.

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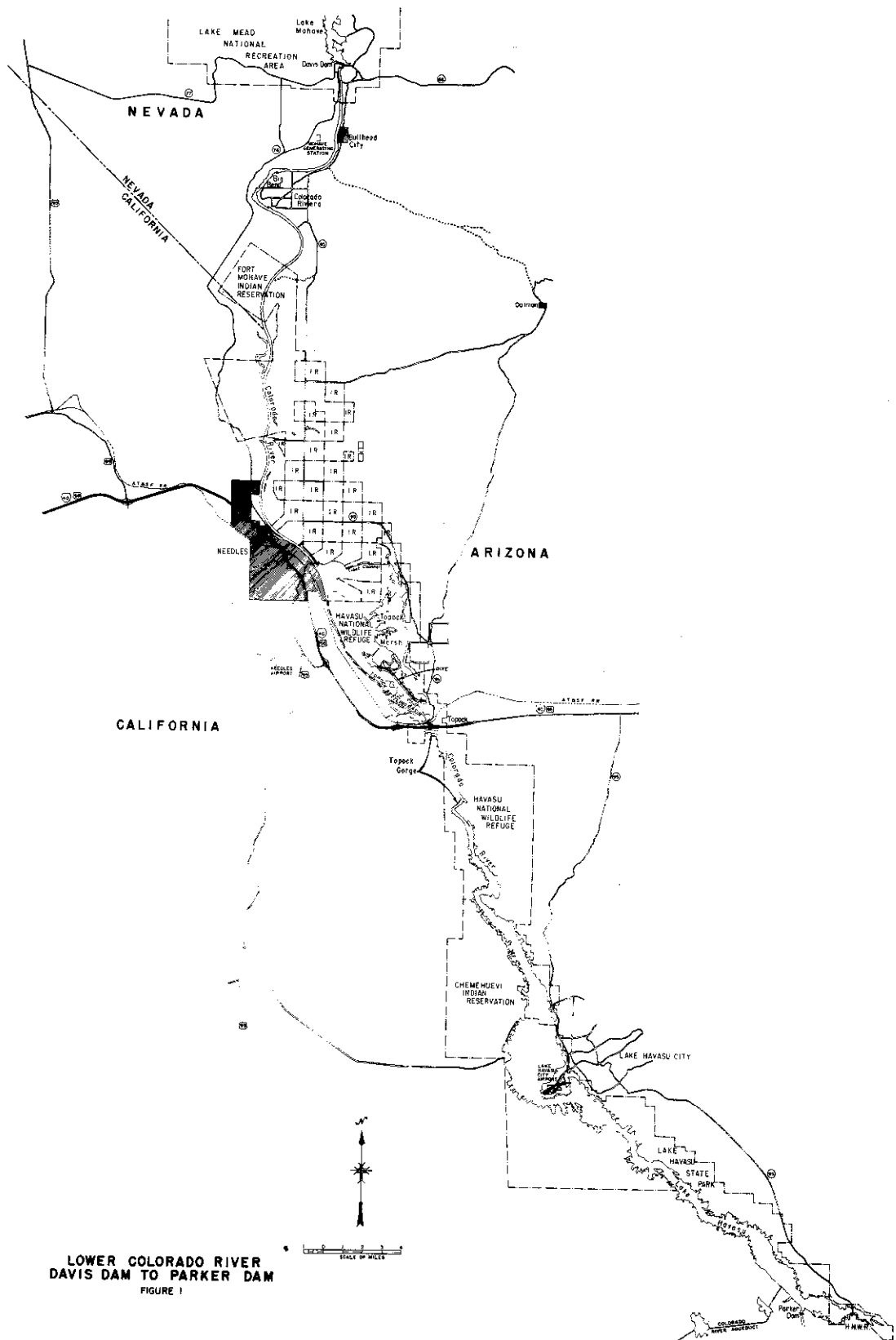
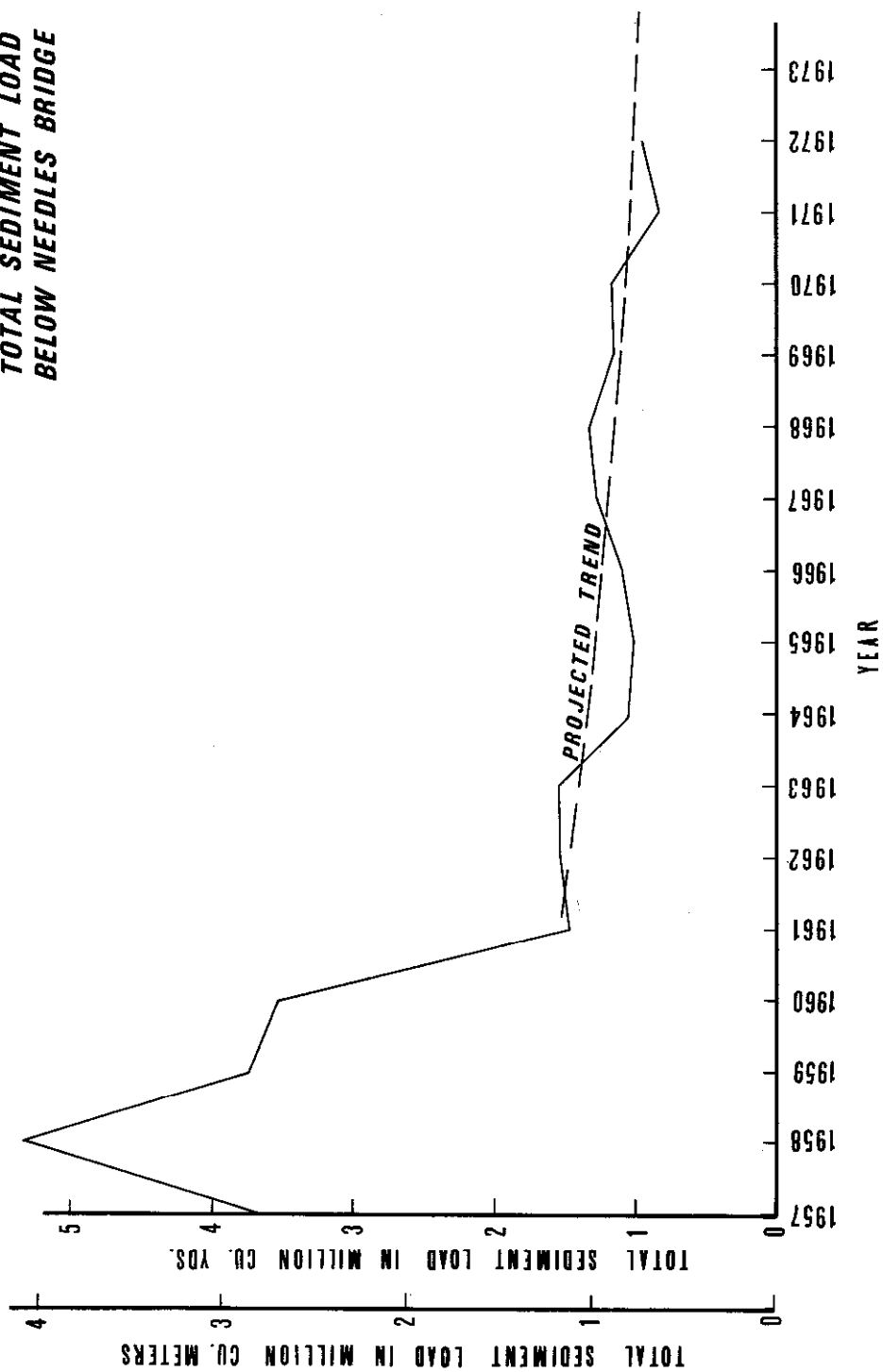


FIGURE 2
TOTAL SEDIMENT LOAD
BELOW NEEDLES BRIDGE



ON SEDIMENT TRANSPORT THROUGH THE GRAND CANYON

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ABSTRACT

With the closure of the Colorado River by Glen Canyon Dam, both the amount and the nature of the sediment movement through the Grand Canyon has changed. At present, the mean annual capacity of the river to carry beach-building material is about 12 million metric tons per year. The tributaries supply about 2.7 metric tons of beach-building sediment per year. The difference of about 9 million metric tons per year must be obtained through scour of bed and/or banks. It is estimated that without remedial measures it may take somewhat more than 200 years before the beaches and sand bars between Glen Canyon Dam and Lake Mead disappear.

THE PROBLEM

With the closure of the Colorado River by Glen Canyon Dam, both the amount and the nature of the sediment movement through the reach from Glen Canyon to the headwaters of Lake Mead changed. In the past the flow of the Colorado was characterized by long periods of low flow and by floods of varying magnitude and duration. Most of the sediment load over a year was moved during the flood period; and the amount was a function of the flood magnitude, the flood duration, and the composition of the sediment supplied to the stream from the watershed. Table I shows the yearly water discharge and sediment load at Lees Ferry and Grand Canyon. The Lees Ferry gaging station is 26 kilometers (16 miles); the Grand Canyon station 167 kilometers (104 miles), and the headwaters of Lake Mead, about 480 kilometers (300 miles) downstream from Glen Canyon Dam. The major tributaries below Glen Canyon are the Little Colorado River and Paria River; their flows and loads are shown in Table II. It is probable that the minor tributaries below Lees Ferry add to the Colorado considerably less sediment than the two major tributaries. Although high water years in general move more sediment than low water years, annual discharge alone is not sufficient to allow the prediction of the load.

Figure 3 shows the relationship between the instantaneous discharge and load; the scatter is correlated, more or less, with the percent of the suspended load finer than 0.016 mm. The scatter is great: For a discharge of 300 m³/sec (10,000 cfs) the sediment load can be anything from 27,000 to 1,350,000 metric tons per day (30,000 to 1,500,000 tons) and the size distribution of the sediment is the primary factor in the scatter. Other factors are the variations in the flow characteristics and the temperature of the water.

Lake Powell behind Glen Canyon Dam is a regulating reservoir for the flow and a trap for the sediment load from the upper basin. Essentially all the flow at Lees Ferry, 98 percent of the flow at Grand Canyon, and 97 percent of the flow entering Lake Mead from the Colorado are released from Lake Powell in a

TABLE I

Measured Yearly Discharge and Sediment Load
of the Colorado River at Lees Ferry and Near Grand Canyon

Water Year	LEES FERRY		GRAND CANYON	
	Average Discharge Cubic Meters per Second	Suspended Sediment Million Metric Tons	Average Discharge Cubic Meters per Second	Suspended Sediment Million Metric Tons
1948	534	99.5	542	131.0
1949	560	87.8	561	108.0
1950	432	48.5	433	54.1
1951	384	43.4	385	44.1
1952	703	104.0	710	135.6
1953	344	37.6	347	44.2
1954	239	31.3	244	37.0
1955	285	52.5	296	75.5
1956	342	46.0	347	69.0*
1957	677	109.0	685	141.0
1958	565	102.0	570	120.0*
1959	264	14.4	272	22.2*
1960	359	25.4	374	36.0
1961	260	35.8	275	42.5
1962	577	56.7	595	77.5
1963	98	14.0	107	18.3
1964	95	4.0	106	18.5
1965	424	5.4	430	36.0
1966			326	8.2
1967			323	21.2
1968			350	14.8
1969			464	13.1

* Estimated

regulated, non-natural pattern. Since Lake Powell traps all the sediment delivered to it, very little sediment is now supplied to the river between Glen Canyon Dam and Lees Ferry. The small sediment load moving past Lees Ferry now is coming largely from the bed and banks in the reach between Glen Canyon Dam and Lees Ferry. Eventually the flow past Lees Ferry will be almost always clear, as the bed, banks and beaches are swept away or gradually armored through self sorting, leaving a natural riprap protecting the surface.

Because the sediment supplied to the flow past Lees Ferry most of the time will be zero, there will be general degradation in the reach above; and it is

TABLE II

Measured Yearly Discharge and Sediment Load of the Paria River
at Lees Ferry and the Little Colorado River Near Cameron

Water Year	PARIA RIVER AT LEES FERRY		LITTLE COLORADO NEAR CAMERON	
	Average Discharge Cubic Meters per Second	Suspended Sediment Million Metric Tons	Average Discharge Cubic Meters per Second	Suspended Sediment Million Metric Tons
1948	0.75		7.86	4.03
1949	0.77	1.96	11.22	12.95
1950	0.53	1.31	1.84	2.92
1951	0.54	1.38	1.96	4.51
1952	0.74	1.80	13.77	17.30
1953	0.70	4.14	2.40	3.77
1954	0.61	2.09	4.25	8.24
1955	0.69	3.92	7.65	20.20
1956	0.39	0.95	0.76	1.83
1957	0.65	2.91	6.85	9.93
1958	1.54	10.20	6.55	10.40
1959	0.54	2.52	2.01	6.21
1960	0.41	0.37	7.59	8.01
1961	1.21	11.80	1.51	3.18
1962	0.59	1.89	6.17	5.31
1963	0.78	5.40	3.30	9.82
1964	0.52	1.15	6.69	15.58
1965	0.60	1.23	8.82	8.76
1966	0.63	1.22	7.92	9.59
1967	1.02	5.15	7.45	17.31
1968	0.93	6.21	8.35	8.64
1969	1.04	3.50	5.46	7.81

the competence rather than the capacity of the stream, in respect to sediment movement, which is of interest. In the next reach down to Grand Canyon it is the comparison between capacity to transport sediment and sediment supplied by the tributaries which is of interest. If the capacity exceeds the supply, there will be general degradation and a tendency to enlarge the stream until the reduced capacity equals the supply. If the supply should exceed the capacity, there will be a tendency to aggrade until a smaller, swifter stream is formed with a capacity equal to the supply. In the reach below Grand Canyon and above Lake Mead there should be a smaller tendency to degrade, mainly because the attenuation of the surges in the releases from Glen Canyon reduces the capacity to transport.

THE ANALYSIS

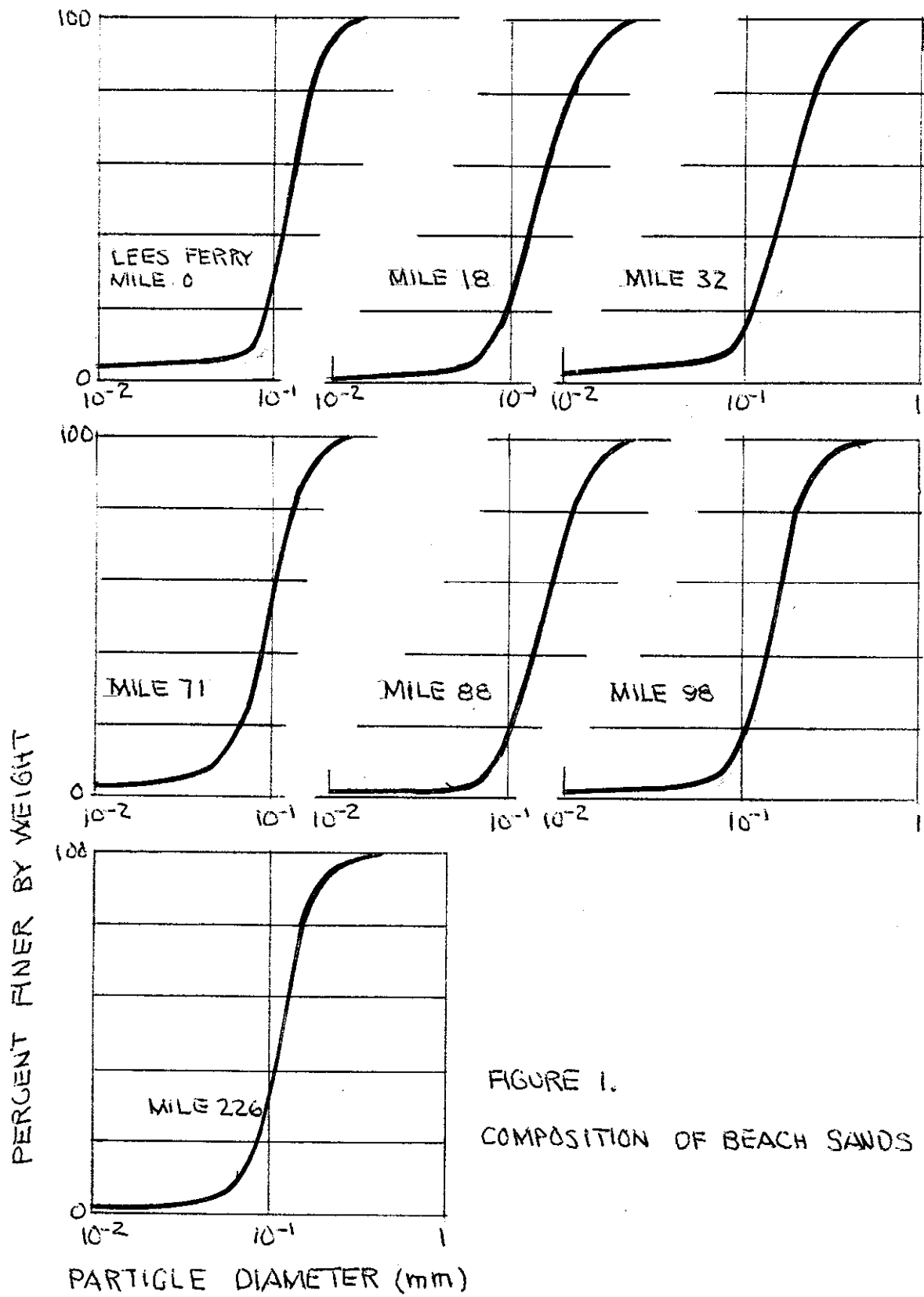
In order to explain the variability of the sediment load, a sediment-transport relationship was needed which would predict the sediment concentration and composition as a function of the flow, the cross section of the stream, and the composition of the bed material. For obvious reasons - nepotism and solidarity - the Laursen relationship was chosen (Laursen, 1958). Zernial has demonstrated that this relationship is capable of explaining the commonly encountered scatter of the sediment load versus discharge plot (Zernial, 1963).

In applying the relationship to the gaging stations at Lees Ferry and Grand Canyon, the cross section was divided up as it was when gaged by the U.S. Geological Survey; and the measured depth and velocity for each subsection were used in the computations. The temperature was always assumed to be 11°C (51.8°F), and the fall velocity of the sediment was taken as that of a quartz sphere. The slope at Lees Ferry was taken as 0.00028 and the slope at Grand Canyon as 0.00063. The sediment load of the subsections was combined for a value for the entire section. Using the measured bed material samples, the concentration computed was less than the measured suspended load concentration, and the composition of the computed suspended sediment was coarser than that measured.

It is very difficult to obtain a true bed material sample; the presence of the sampler near the bottom can cause high velocities at the bed which remove the fine particles before the sampler can take the sample. Therefore, small amounts of fines were arbitrarily added to the measured bed material and the computations were repeated. Reasonably close agreement was then achieved for both the concentration and composition of the suspended load. Since the suspension tended to be largely fine material not found in the bed material samples, it was concluded that the imagined bed material was closer to correct and that the Laursen relationships would probably describe the sediment-transporting characteristics of the stream reasonably well.

As is shown in Figure 1, the size composition of the beach material collected during a reconnaissance trip down the Colorado River by the principal investigators was remarkably constant all the way from Lees Ferry to Diamond Creek. Several bottom samples obtained during the same trip were also quite uniform as shown in Figure 2. Although the bed material samples were obtained with a crude grab bucket, they gave a computed suspended sediment composition which could explain that of the beach material, and computed sediment loads along the low side of the scatter band of measured sediment load and discharge (Figure 3). The scatter of the computed points in Figure 3 is the result of channel instability in that actual velocities and depths were used from stream gaging records.

The computed beach-building suspended sediment load has a somewhat greater range of size variation than the beach sand, which is to be expected. The coarser fractions of the suspended load are in greater concentration near the bed than up toward the water surface; thus the coarser material tends not to be supplied to the deposition area of the beach. The finer fraction of the suspended load, on the other hand, does not tend as much to deposit in the eddy and is swept on away. Thus, it is the middle fraction of the suspended load which builds the beaches.



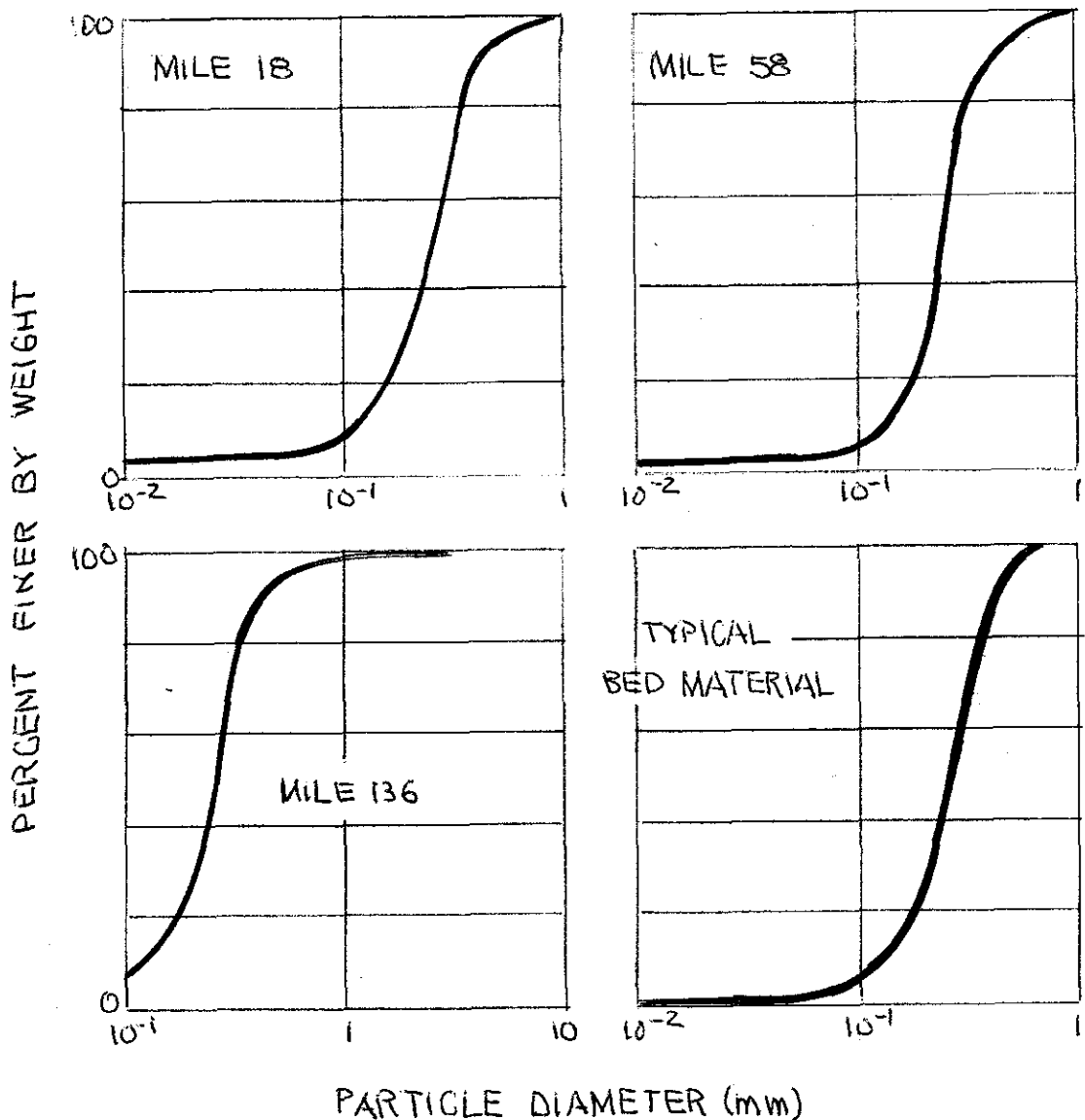


FIGURE 2. COMPOSITION OF BED MATERIAL

Straight lines were put through the computed beach-building sediment load points by eye in Figure 3, with some attention to the edge of the scatter band of the measured values. These relationships were then used to compute the annual beach-building sediment load past Lees Ferry and Grand Canyon as shown in Table III in which the annual load is the summation of the daily loads through the water year. In years without a large magnitude flood the load of beach-building material past Lees Ferry was greater than the load past Grand Canyon indicating that there was deposition between the two stations. However, over the 15 years used in the calculations, the amount of beach-building

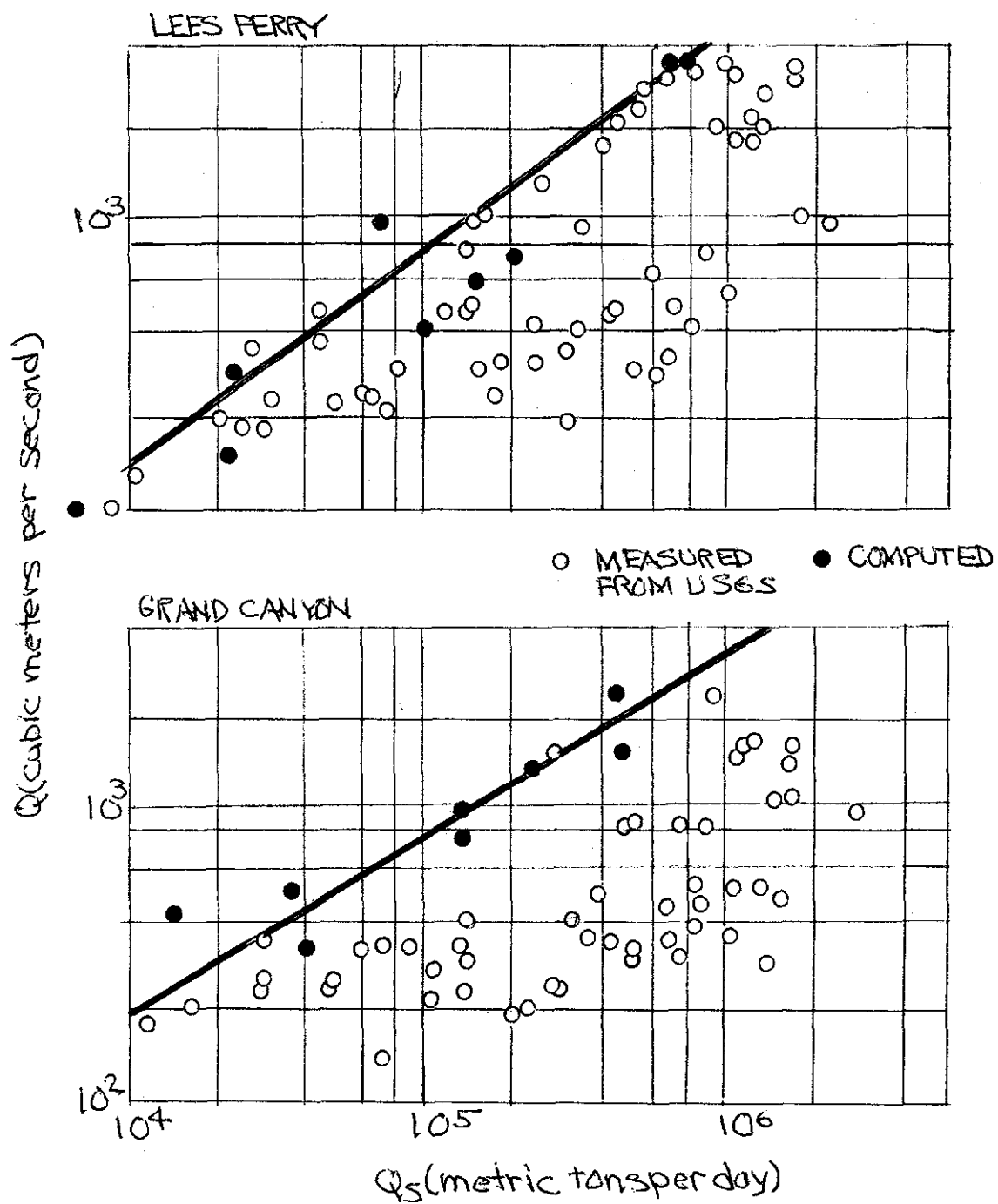


FIGURE 3. LOAD RELATIONSHIP FOR THE COLORADO RIVER

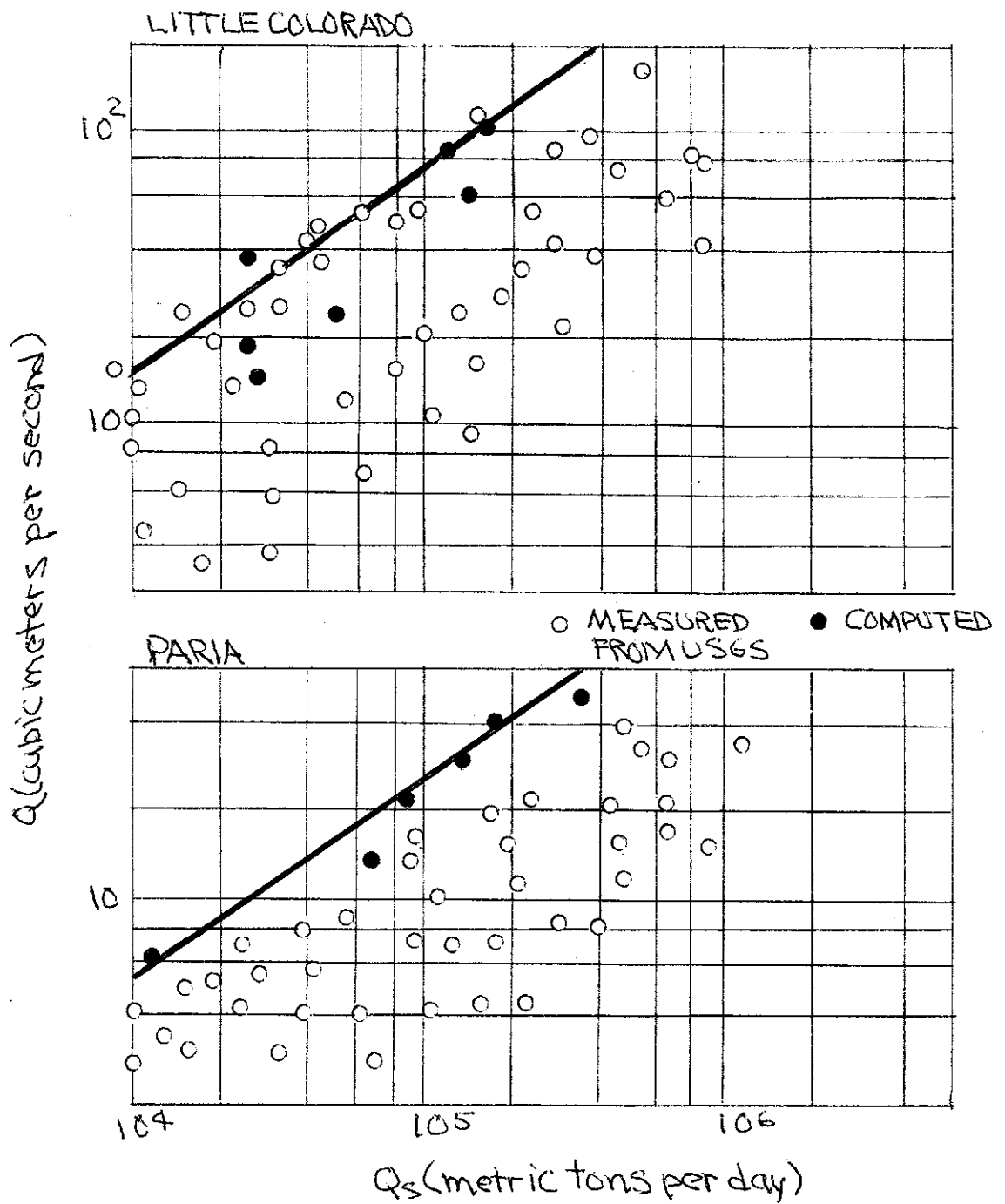


FIGURE 4. LOAD RELATIONSHIP FOR THE TRIBUTARIES

TABLE III

Computed Annual Beach-Building Sediment Load
for the Colorado River at Lees Ferry and Grand Canyon

Water Year	Lees Ferry Million Metric Tons	Grand Canyon Million Metric Tons
1948	29.3	32.1
1949	32.5	35.5
1950	21.2	19.9
1951	18.9	17.4
1952	45.3	55.5
1953	15.8	15.0
1954	8.8	6.8
1955	11.6	10.2
1956	16.5	16.2
1957	44.5	52.5
1958	32.3	38.0
1959	10.6	9.4
1960	16.5	16.0
1961	10.2	9.2
1962	32.1	35.6

material moving past Grand Canyon was greater than the amount moving past Lees Ferry. Presuming that over the 15 years the scour and fill compensate, the difference between the two stations would be the amount supplied by the tributaries in the reach between - especially the Little Colorado and the Paria. Similar computations were made for the Cameron station on the Little Colorado and the Paria station near the junction with the Colorado. Using the same bed material as before with slopes of 0.0049 at Cameron and 0.0042 at Paria, and the measured velocities and depths, the transport of beach-building material was computed. Computed and measured points are shown in Figure 4; straight lines were drawn in by eye through the computed points. These relationships for beach-building sediment load were used with the daily discharges for the Little Colorado and the Paria as shown in Table IV. The computed difference between Lees Ferry and Grand Canyon indicates that the tributaries supply about 1.8 million metric tons per year (2 million tons per year) of beach-building material; the computed loads for the Little Colorado and the Paria indicate that these two major tributaries add about 2.7 million metric tons per year (3 million tons per year). Further work is needed to refine these estimates, but for the moment it is sufficient to estimate that on the average the tributaries between Lees Ferry and Grand Canyon supply about 2.25 million metric tons per year (2.5 million tons per year) and that between Glen Canyon and Lake Mead the tributaries supply about 2.7 million metric tons per year (3.0 million tons per year) of beach-building material. Before Glen Canyon Dam much more fine material passed through the river, and even now somewhat more fine material is supplied; however, the

TABLE IV

Computed Annual Beach-Building Sediment Load
for the Little Colorado and Paria Rivers

Water Year	Paria Million Metric Tons	Little Colorado Million Metric Tons
1948	0.43	4.60
1949	0.35	3.24
1950	0.17	0.42
1951	0.28	0.93
1952	0.33	7.10
1953	0.43	0.61
1954	0.34	1.73
1955	0.37	3.80
1956	0.10	0.13
1957	0.33	2.58
1958	1.50	2.20
1959	0.26	0.58
1960	0.10	2.97
1961	1.42	0.32
1962	0.31	2.42

stability of the river is a matter not of the fine material but of the beach-building material.

Computations were made of the beach-building sediment load which could be carried by the regulated river for the three years 1967-68, 1968-69, 1969-70. Bi-hourly values were used to obtain a factor to account for the daily surge in the flow, and then daily values were used to obtain the yearly values. As shown in Table V the capacity for transport at Lees Ferry is slightly larger than for Grand Canyon (there are no more large floods), but they are both in the order of about 12 million metric tons per year (13 million tons per year). At Lees Ferry virtually all of this load must be made up from scour of the bed and banks between Glen Canyon Dam and Lees Ferry, if indeed there is sufficient supply left. Indications are, however, that very little beach-building material is moving past Lees Ferry. At Grand Canyon, some of the load is supplied by the tributaries - Paria and Little Colorado and a number of smaller ones - but about 9 million metric tons per year (10 million tons per year) must be obtained through scour of bed and/or banks.

As many of the banks of the Colorado River are characterized by shear rock walls 9 million metric tons per year represents a lot of material. There may be some self sorting so that the beaches gradually become paved with coarser material which cannot be moved as readily by the flow. However, there may also be a tendency for the pools to fill and for the flow to keep working on the beaches. The beaches are mostly in eddy zones before and after rock falls or large boulder deposits, and the erosive action of the flow on the beaches does not decrease a great deal as the beaches retreat.

TABLE V
Annual Beach-Building Sediment Load
of the Regulated Colorado River

Water Year	Lees Ferry Million Metric Tons	Grand Canyon Million Metric Tons
1968	12.6	11.2
1969	12.2	11.7
1970	13.2	11.4

THE PRELIMINARY CONCLUSIONS

Preliminary investigations show that the beaches of the Colorado River between Glen Canyon Dam and Lake Mead could be in danger of being washed away since the transport capacity of the regulated river is in excess of the amount of beach-building material being supplied from the tributaries in this 480 kilometer (300 mile) reach. How long they will last cannot as yet be estimated; certainly more than 10 years, probably less than 1000 years; but how much more or less than 100 years is a matter for continued study. However, it is possible to say that on the average the capacity of the Colorado River to transport beach-building material is about 12 million metric tons per year (13 million tons per year) at present, and the tributaries supply only about 2.7 million metric tons per year (3 million tons per year). Since the closure of the river by Glen Canyon Dam, no sediment is being supplied from the upper basin.

This general tendency for degradation may be alleviated to some degree if the river deposits (bed and beach) become self armored through coarsening by self sorting as the river degrades. However, there is very little coarse material in the beaches, so that they would probably continue eroding and become small vestiges of the present beaches which are now none too large.

On the other hand, with no floods in the regulated river there may be a tendency to fill the pools all along the reach. In that case the beaches would be in even a greater danger as all the capacity for transport would tend to be satisfied by erosion of the beaches. And as the pools fill, the velocities, and hence the shear on the banks, would tend to become greater.

POSTSCRIPT

During an inspection of the Colorado between Lees Ferry and Glen Canyon Dam in June of 1973 much evidence of bank erosion was apparent. Almost all of the talus slopes and beaches either had vertical slump faces or exposed rock protecting the underlying material. Some of the growth on the beaches under-

cut in the slumping of the banks was far older than Glen Canyon Dam. Most beaches left were in the lee of obstructions or other bank configurations giving rise to a lee eddy. Beaches upstream from the obstructions were either very small, and in an eddy zone or covered with self-sorted riprap. There were a few large sand bars off lee beaches which had reportedly been growing. The source of sand for these bars, however, was the lee beach itself which was eroding. Probably with the cutting back of the beach, the lee eddy became large enough so that a sand bar could build up in the central low velocity area. As the beach disappears, so will the sand bar. On the few sizable beaches left the "campsite" sign has had to be moved back several times as the beachline retreated.

The river bottom which was observable was mostly cobble and much of it was covered with moss. There were a few sand bars with ripples moving upstream on the bank half; these are probably temporary transitional features. In only one small area was there an active bed with dunes of coarse sand.

What has happened in this first 26 kilometer (15 mile) reach of the river below the dam is in conformity with the preliminary conclusions - that degradation can be expected through the Grand Canyon, that the beaches will gradually disappear, but that there may be some natural revetment by self-sorting in the erosional process and some vestiges of beaches may remain in the low velocity eddy in the lee of obstructions.

Since the closure of Glen Canyon Dam about 10 years ago, the degradation has progressed to the vicinity of Lees Ferry. Below Lees Ferry, the Paria and Little Colorado and other tributaries contribute their sediment load; therefore, the rate of degradation in the 480 kilometer (300 mile) reach to Lake Mead would be somewhat less than 26 kilometers (16 miles) per 10 years. Hence it may be somewhat more than 200 years before most of the beaches vanish.

ACKNOWLEDGEMENT

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SEDIMENT TRANSPORT STUDIES
IN THE DELTA-MENDOTA CANAL
AND THE CALIFORNIA AQUEDUCT^{1/}

James F. Arthur^{2/} and Norman W. Cederquist^{3/}

ABSTRACT

The Delta-Mendota Canal (DMC), completed in 1951, has experienced capacity problems, currently attributed to a combination of design deficiency and the accumulation of sediment and clam deposits. Investigations over a number of years have concluded that sediment in the water is bound by the excreta of the Asiatic clam (Corbicula manilensis). This accumulation of sediment-clam deposits causes a reduction in canal capacity, in addition to the design deficiency loss.

In the present investigation, sediment transport characteristics of the DMC and the California Aqueduct were compared. The investigation indicated that there were significant differences in the two intake facilities. Approximately 70 percent of the sediment entering the Aqueduct was deposited in Clifton Court Forebay and Bethany Reservoir, while only 10 percent was deposited in the DMC intake channel. The difference in sediment deposition was attributed to differences in design and operation of the intake facilities.

The concentration of total suspended solids entering the DMC was found to vary directly with total delta export and vary inversely with the solids concentration of the Sacramento River, the primary source of delta export water.

It appears that there is deposition of sediment in the channels leading to the export facilities during the winter and that resuspension of this sediment occurs in the summer during periods of high delta export.

OBJECTIVES OF REPORT

This report summarizes the findings of studies conducted in 1973-74 on the sediment-clam deposition problem in the Delta-Mendota Canal (DMC) and makes recommendations for potential alternatives for

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controlling the problem. The 1973-74 studies characterized sediment transport in the DMC and California State Aqueduct. The objectives of the studies were to determine the source(s) and quantities of suspended sediments entering the two conveyance systems, compare physical and operational differences, and determine the extent and location of sediment deposition.

INTRODUCTION

The Delta-Mendota Canal and the California Aqueduct are the major water conveyance facilities of the Federal Bureau of Reclamation's Central Valley Project and the California State Water Project. The DMC provides a major supply of water for agriculture on the west side of the San Joaquin Valley. The DMC conveys water from the Sacramento-San Joaquin Delta south 186 km (116 mi) to the Mendota Pool, on the San Joaquin River. Since 1968, water has been delivered from the DMC at km 111.5 (MP 69.30) to the San Luis Project through the O'Neill Pumping Plant. The canal is concrete lined for 153 km (95 mi), has a 14.65 m (48 ft) bottom, 1.5:1 side slope and an average water depth of 4.9 m (16 ft). The design capacity of the canal is 130 m³/s (4,600 ft³/s) and the design water velocity is 1.16 m/s (3.8 ft/s). The water flows through a 4.0 km (2.5 mi) intake channel to the Tracy pumps which lift the water 61.0 m (200 ft) into the concrete lined portion of the canal. The pumps operate continuously during the irrigation season which extends from January through October (figure 1).

Since the canal approached design capacity operation in the 1960's it has had continuing capacity problems. The canal has been dewatered for repairs and removal of sediment and biological growth accumulations on six different occasions. Furthermore, there are indications that the canal capacity problem is increasing. Removal of sediment-clam deposits averaged about 14,000 yards/year, prior to 1966, and 20,000 yards/year since 1968.

The severity of the canal capacity loss culminated in the Bureau raising the canal sidewall by 0.46 m (1.5 ft) in 1962, in order to provide 111 m³/s (4,200 ft³/s) canal capacity to O'Neill Pumping Plant. This temporarily alleviated water demand requirements but did not resolve the canal capacity loss problem related to sediment-clam deposition.

The costs for dewatering-cleaning operations are substantial. The physical removal of sediments from the canal has been estimated to cost from \$43,000 to \$100,000 per cleaning operation. Additional costs include the extra costs of pumping sediment-laden water and the undocumented but substantial physical damage to the canal each time it is dewatered.

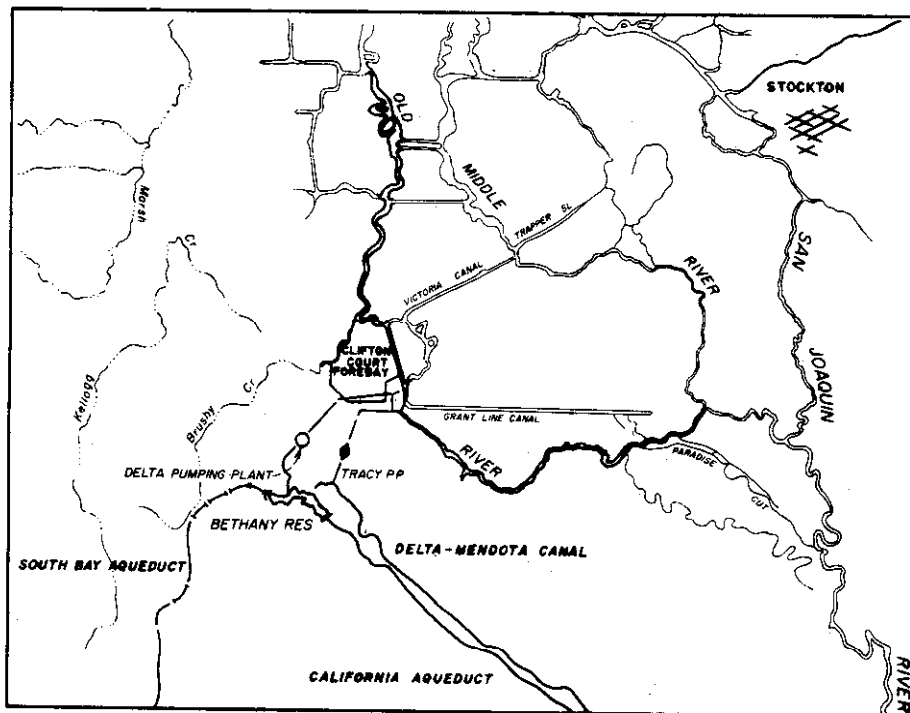
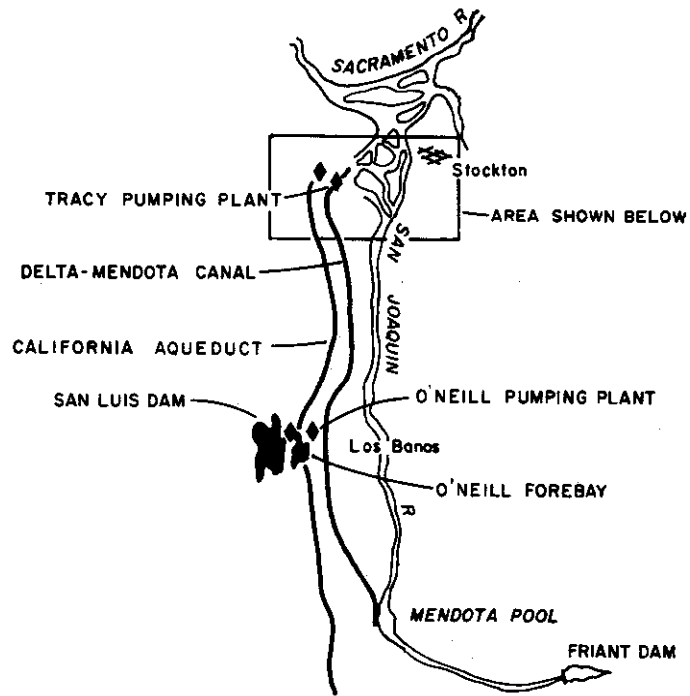


FIGURE 1—Sacramento-San Joaquin Delta illustrating Delta-Mendota Canal and State Aqueduct export facilities

Fortunately, in the past, water demands have been less than maximum canal capacity and water demands have been met. However, in the near future, water demands are expected to reach canal capacity. Reduction in canal capacity due to sediment-clam deposition and loss during the 8 to 10 weeks required for dewatering and cleaning operations will result in loss of water sales. The potential loss in water sales is projected to be from 400,000 to 450,000 acre-feet per year at an approximate cost of \$3.50 to \$7.50 per acre-foot.

A number of studies have been conducted to determine the cause(s) of canal capacity loss. Hebert (1958) reported that clean canal capacity was 15 percent less than original design capacity and that capacity was reduced by an additional 6.5 percent due to the accumulation of sediment and filter feeding organisms - the Asiatic clam, Corbicula, and an amphoid, Corophium. Prokopovich (1962) attributed buildup of clam sediments as having their origin from three possible sources: (1) inorganic and organic material pumped into the canal from the delta, (2) organic growth, including clams, amphoids, bryozoa, sponges, and other organisms transported into the canal from the delta, and (3) sand, clay, and organic sediment originated locally and transported into the canal through drain inlets, bank erosion, and wind. Subsequent dewatering and cleaning operations were investigated and reported on by Prokopovich (1964, 1965, 1967, 1969, 1973a, 1973b) Prokopovich and Hebert (1964) and Swain and Prokopovich (1969). They concluded that the Asiatic clam, Corbicula manilensis, binds sediments together in the form of excreta which prevents the sediments from passing freely down the canal. These studies culminated in a contract with the California Academy of Sciences, San Francisco, California, to conduct a 2-year biological investigation of the canal (Eng, 1975). In support of this biological study, the Water Quality Branch of the Bureau of Reclamation concurrently conducted a water quality investigation (Lentz, 1975). During the past 18 years, considerable speculation has been directed toward the possible effects the filter feeding organisms have on sedimentation, with little attention directed toward the source of sediments and the physical features of the DMC which are influencing sedimentation.

The California State Water Project was constructed during the 1960's to transport water from the Feather River in the Sacramento Valley, southward along the west side of the San Joaquin Valley, to provide supplemental water supply for the Southern San Joaquin Valley and Southern California. The delta facilities are adjacent to the Tracy Pumping Plant and intake channel and the Aqueduct parallels the DMC from near Tracy to the O'Neill Forebay (figure 1). The Aqueduct has a 14.65 m (48 ft) bottom, 1.25:1 side slope, 9.15 m (30 ft) water depth and a capacity of 283 m³/s (10,000 ft³/s). The delta facilities include Clifton Court Forebay (C.C.) which has a surface area of 810 ha (2,000 acres) and an average water depth of 3 to 3.6 m (10 to 12 ft), the Delta Pumping Plant and Bethany Reservoir

which is immediately down canal from the pumps, is about 1.6 km (1 mi) long, has a water depth varying from 5 to 20 m (16 to 65 ft) and a surface area of 80 ha (200 acres).

The State Water Project began operation in the fall of 1967. The Delta Pumping Plant operates during periods of off-peak power. The basic pumping schedule of operation in 1973 was from 10 p.m. to 7 a.m., Monday through Thursday, and from 10 p.m. Friday to 7 a.m. Monday, except for the period from 1:30 p.m. to 10 p.m. Saturday.

The intake channels of both the DMC and Clifton Court Forebay connect to Old River, the most westerly San Joaquin River branch in the delta. Inflow into the DMC intake channel is unregulated, the only physical structure being the Tracy Fish Screen Facility. A gated structure regulates the flow into Clifton Court Forebay. Old River is under tidal influence and the gates are opened only during high tide to permit water inflow. This operation takes advantage of the tidal current and provides a higher average water elevation in Clifton Court, thereby reducing the total pump lift into Bethany Reservoir.

While Old River is the major conveyance channel for export water supply, water flows through a number of delta channels leading to the two pumping plants. Essentially all of the flow of the San Joaquin River, in excess of local delta demands, is exported. Additional export water is obtained from the Sacramento River.

METHODS

A field study was conducted on July 30, 31, and August 1-3, 1973, to determine the extent of sediment and clam accumulation in the State system and develop methods of collecting sediment samples.

Dredge samples were collected from Clifton Court, Bethany Reservoir, and 64 km (40 mi) of the Aqueduct using the California Academy of Science's Lyman and Peterson dredges that were being used in their DMC Biological Study. A recording fathometer was also used to make bottom profiles of Clifton Court and Bethany Reservoir. These readings were compared to design specifications to determine the extent of sediment-clam buildup in the two reservoirs (Arthur and Cederquist, 1973a).

Sig-a-motor automatic time sequence composite water samplers were used to collect sediment-water samples of the Clifton Court inlet gates and the fish screen, on the DMC Intake Channel. Three samplers were located at the Clifton Court gates and were operated to collect water samples from 0.3 m (1 ft) beneath the surface, at mid-depth, and 0.3 m (1 ft) off the bottom when the gates were open. The timing mechanism of each sampler was programmed to collect water for 10 minutes, of each 44 minute interval. A fourth sampler was located at the fish screen on the DMC and collected water from mid-depth. Grab samples

were collected from near the top, mid-depth, and near the bottom at both locations at 3- to 4-hour intervals. Turbidity of the samples was measured using a Hach 2100 turbidity meter, and then the samples were stored on ice, in the dark, for return to the USBR laboratory where total and volatile suspended solids were analyzed according to Standard Methods (1971).

A second 14-day field study was conducted from September 12 to 26, 1973, (Arthur and Cederquist, 1973b), to collect water-sediment samples from three locations in the DMC and three locations in the Aqueduct. Samplers were located at the fish screen, the Tracy Pump discharge, km 5.6 (mi 3.50) and at check 12, km 102.94 (mi 63.98) on the DMC; and at Clifton Court Inlet, the Delta Pump discharge, km 4.83 (mi 3.00) at check 1, the outlet for Bethany Reservoir, km 9.49 (mi 5.90) on the Aqueduct. The samplers were programmed for 10 minutes of every 44-minute interval, collected water from mid-depth and were wired into the Aqueduct control system so as to operate only when water was flowing through the conveyance facilities. The preliminary study had shown sediment was completely mixed in passage through the control gates and the fish screen and similar turbulent mixed sites were selected for all six samplers.

As a result of this study, monthly total and volatile suspended solids analyses were added to the DMC Water Quality Investigation (Lentz, 1975) for the period from September 1973 to October 1974.

RESULTS AND DISCUSSION

The following sections summarize sediment transport studies conducted during 1973-74 on the DMC and Aqueduct.

Sediment Transport in the California Aqueduct

Dredge sampling was conducted in conjunction with fathometer profiling in both Clifton Court and Bethany Reservoir. Very few clams were found in Clifton Court and these were confined to hard bottom areas away from the inlet. Apparent sediment deposition was detected in a number of areas. Two shallow water areas, 1-1.5 m (3 to 5 ft) were found, one near the inlet and the other near the channel leading to the delta pumps. Asiatic clams and a number of shallow deposits of soft black mud were found in Bethany Reservoir. Bethany Reservoir was formed by damming a number of small valleys in the foothills of the Coast Range Mountains, creating a long narrow lake containing a series of shallow and deep water areas. Clams were confined primarily to shallow water areas and sediment deposition to the deeper areas.

The concrete lined portion of the Aqueduct was sampled at locations similar to where sediment-clam deposits were found in the

DMC using the Lyman dredge (an Ekman modified by the California Academy of Sciences for the DMC biological study). No deposits of sediment or clams were found in the first 21 km (13 mi) of the canal. However, a number of moderate sandy sediment and clam deposits were found between km 21 and 61 (mi 38).

The concentration of total suspended solids entering Clifton Court Forebay averaged 51 mg/l for the grab samples and 44 mg/l for the composite samples collected during the August 1-3 study. During the 14-day period in September, the average was 49 mg/l at the Clifton Court inlet, 17 mg/l at the Delta pump discharge and 14 mg/l at the Bethany Reservoir outlet at check 1 (table 1). This represented a 72 percent reduction in sediment load between the intake and check 1.

Table 1

California Aqueduct Sediment Loading
September 12 to 26, 1973

	<u>TSS</u>	<u>VSS</u>	<u>NVSS</u>
C.C.	49 mg/l	8 mg/l	41 mg/l
Inlet	133 lbs/acre-foot	22 lb/acre-foot	111 lbs/acre-foot
Delta	17 mg/l	5 mg/l	12 mg/l
Pump head	46 lbs/acre-foot	14 lbs/acre-foot	32 lbs/acre-foot
Check 1	14 mg/l	4.4 mg/l	9.6 mg/l
	38 lbs/acre-foot	12 lbs/acre-foot	26 lbs/acre-foot

There was also a change in composition of the solids, from 84 to 69 percent nonvolatile suspended solids indicating a higher percent loss of inorganic than organic solids in Clifton Court Forebay and Bethany Reservoir. This decrease in suspended solids load and inorganic content of the solids could have several effects. First of all, canal water velocity may be great enough to carry a higher percent of the remaining load through the canal. Secondly, the reduced turbidity, resulting from the reduction in sediment load, may increase the potential for attached and suspended algal growth, while at the same time reducing conditions necessary for clam growth. Since neither Corbicula nor Corophium were found in Clifton Court Forebay, it appears probable that the filter feeding organisms are unable to survive in the Forebay, presumably because of the sedimentation.

Sediment Transport in the Delta-Mendota Canal

The suspended solids concentration in the DMC during the 14-day study in September averaged 64 mg/l at the fish screen, 59 mg/l at the Tracy pump discharge, and 52 mg/l at check 12 (table 2). The August 1-3 samples collected at the fish screen averaged 66 mg/l, total suspended

solids. The difference indicated a 10 percent dropout in the intake channel and another 10 percent dropout between the Tracy pumps and check 12, with 80 percent passing on down the canal. The character of the solids changed little, from 86 to 83 percent nonvolatile solids.

Table 2

Delta-Mendota Canal Sediment Loading
September 12 to 26, 1973

	<u>TSS</u>	<u>VSS</u>	<u>NVSS</u>
DMC	64 mg/l	9 mg/l	55 mg/l
Intake	175 lbs/acre-foot	25 lbs/acre-foot	
Tracy	59 mg/l	8 mg/l	51 mg/l
Pump Head	156 lbs/acre-foot	22 lbs/acre-foot	134 lbs/acre-foot
MP 3.50			
MP 63.98	52 mg/l	9 mg/l	43 mg/l
	142 lbs/acre-foot	25 lbs/acre-foot	117 lbs/acre-foot

In summary, the amount of suspended solids entering the Aqueduct during the September study averaged 133 lbs/acre-foot, while the load entering the DMC averaged 175 lbs/acre-foot (tables 1 and 2). However, in the State system, solids load entering the canal at check 1 was reduced to 38 lbs/acre-foot, while the solids load entering the DMC at the pump discharge was 156 lbs/acre-foot.

Possible Sediment Sources

Total delta water export has increased greatly since 1966. DMC export has increased from an average of 1,650,000 in 1966 to 2,177,000 acre-feet in 1974, while the California Aqueduct export has increased from 910,000 acre-feet since the first year of operation in 1968 to 1,860,000 acre-feet in 1974. The result has been an increase in sediment removal during dewatering-cleaning operations to an average of 20,000 yd³ annually, compared to 14,000 yd³/yr during the 1956-1966 period, see table 3. The increased sedimentation rate in the DMC appeared to be directly proportional to the increase in delta export pumping.

Measurement of suspended solids at the Tracy pump discharge (September 1973 to September 1974) indicated that the concentration of solids was low during the winter when delta export was low, and high during the summer, when export was high (figure 2). The seasonal solids concentration variation in the DMC was similar to the seasonal variation in the San Joaquin River at Vernalis, but varied inversely with the concentration of suspended solids in the Sacramento River at Sacramento (figures 2 and 3). This would seem

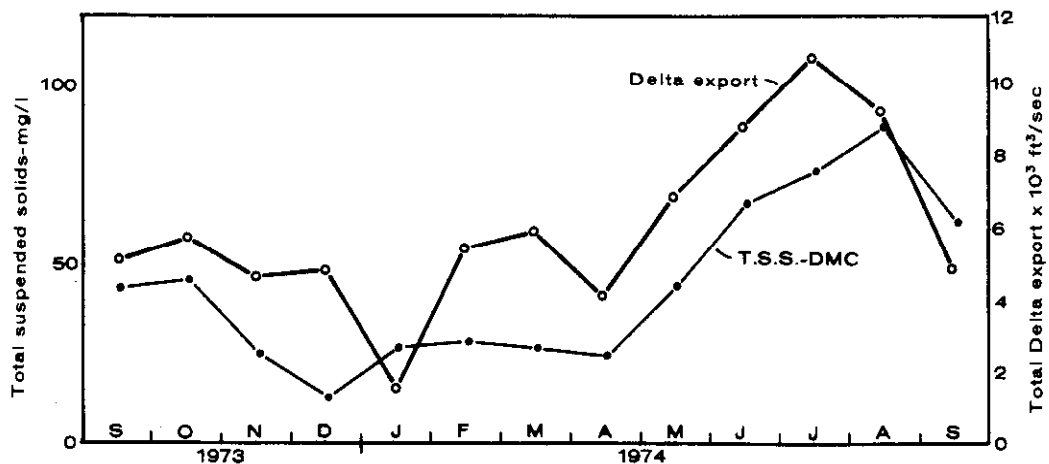


FIGURE 2—Comparison of average total delta export and average total suspended solids entering the Delta-Mendota Canal (USBR data)

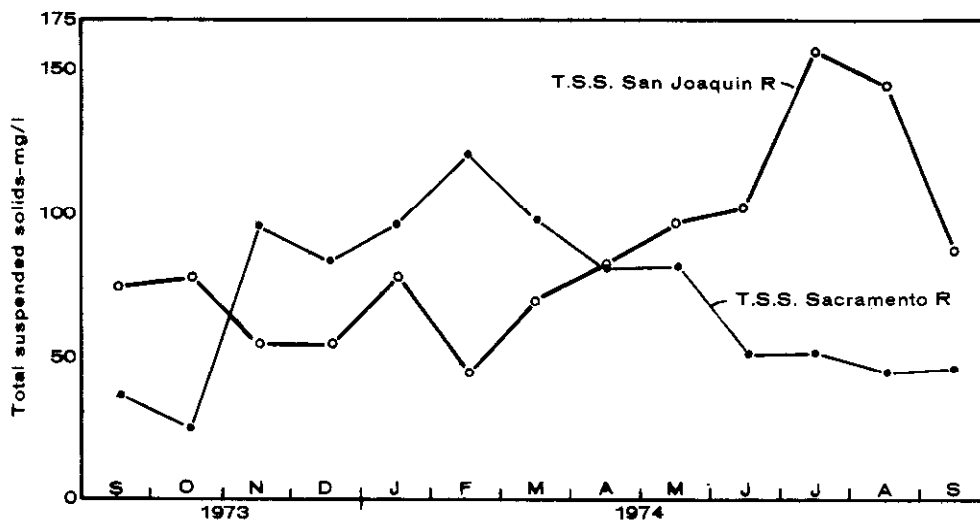


FIGURE 3—Average monthly total suspended solids in the San Joaquin River at Vernalis and Sacramento River at Sacramento (USGS data)

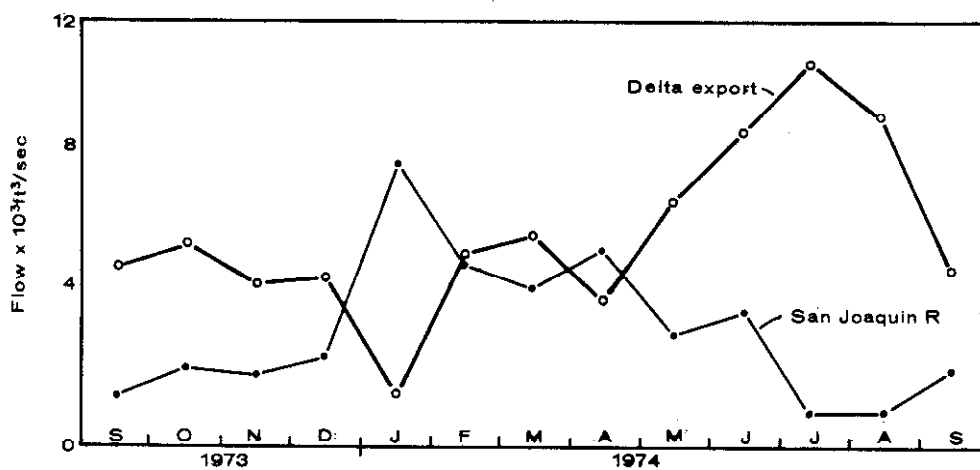


FIGURE 4—Comparison of average total delta export and average San Joaquin River flow at Vernalis (difference between export and river flow made up by Sacramento River flow)

to indicate that the San Joaquin River was the major source of suspended solids in the DMC. However, as illustrated in figure 4, during periods of high export, the flow in the San Joaquin River entering the delta, not allowing for delta consumption, was only 10 to 20 percent of total export and the remaining supply was from the Sacramento River.

Table 3

Wet volume of bottom sediments on the invert of the DMC
measured during dewaterings

Data are from Prokopovich (1965, 1967, 1973a, b)
and Prokopovich and Hebert (1964).

Units are cubic meters (cubic yards)

<u>Year of Dewatering</u>	<u>km 5.63-93.77 (MP 3.50-58.28)</u>	<u>km 93.77-112.65 (MP 58.28-70.01)</u>	<u>O'Neill Intake Channel km 111.50 (MP 69.30)</u>	<u>Total</u>
1955/56	-	-	-	-
1960/61	51,990 (68,000)	-	-	51,990 (68,000)
1962/63	7,800 (10,200)	13,685 (17,900)	-	21,485 (28,100)
1964/65	6,115 (8,000)	11,160 (14,600)	-	(22,600)
1965/66	12,540 (16,400)	11,850 (15,500)	-	24,390 (31,900)
1969/70	50,845 (66,500)	15,290 (20,000)	30,275 (39,600)	96,410 (126,100)
1972/73	29,590 (38,700)	10,015 (13,100)	19,115 (25,000)	58,720 (76,800)

During the winter, November to March, the suspended solids concentration of the water entering the DMC averaged less than 30 mg/l, but averaged 60 mg/l in the San Joaquin and 90-100 mg/l in the Sacramento River. During the high summer export season, April to September, suspended solids in the DMC averaged 60 to 70 mg/l, over 100 mg/l in the San Joaquin and less than 50 mg/l in the Sacramento River. This indicates a deposition of suspended solids during the winter as water is transported through the delta to the DMC intake and a resuspension of the deposited solids during the summer during periods of high delta export.

CONCLUSIONS

1. The concentration of suspended solids entering the Delta-Mendota Canal is directly related to delta export and inversely related to total sediment load in the source water.
2. The increase in suspended sediments concentration with increased delta export is probably the result of resuspension of materials in the channels leading to the DMC intake facilities.
3. Deposition of sediments in the DMC, during the study, was approximately 10 percent of the total entering the canal. However, the amount depositing in the canal probably depends upon sediment input and pumping rates.
4. The facility design and operation criteria of the Federal and State projects have a significant effect on sediment transport in the two systems. Clifton Court and Bethany Reservoirs serve as sedimentation basins for the State Aqueduct, reducing sediment-clam input to the Aqueduct.
5. Removal of sediments prior to the pumping facilities on the DMC should reduce sedimentation and concentration of filter feeders, Corbicula and Corophium, in the Delta-Mendota Canal. However, the reduced turbidity may increase attached algal growth.

RECOMMENDATIONS

1. A study should be conducted to determine the specific source(s) of sediments entering the Delta-Mendota Canal and California Aqueduct from the delta channels. A possible study might be to measure sediment transport in all of the delta channels leading into the intake facilities under different export regimes, tidal phases, and delta outflow conditions.
2. In order to prevent deposition in the conveyance systems and/or reduce the food supply for filter feeding organisms, sediment removal should be accomplished prior to entering the delta export facilities. In regards to this, further evaluation should be conducted to compare feasibility and costs of present methods of restoring the Delta-Mendota Canal capacity with alternative methods of sediment control. Possible alternatives might include: (1) utilization of an island adjacent to the intake facilities as a sediment basin, (2) deepening of the delta channels leading to the intake facilities with provisions for removing sediment on a periodic basis, (3) utilization of a mechanical device to dredge the canal while it is operating, and (4) dual utilization by USBR and DWR of Clifton Court Forebay with appropriate modifications for sediment removal.

Economic consideration of the above alternatives should be based on the time schedule for the proposed Peripheral Canal, which should reduce the sediment load reaching the export facilities.

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BEDLOAD TRANSPORT IN TWO LARGE, GRAVEL-BED RIVERS, IDAHO AND WASHINGTON

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ABSTRACT

Bedload transport in two gravel-bed rivers has been measured by direct sampling in the Snake and Clearwater Rivers in the vicinity of Lewiston, Idaho. These rivers are large and capable of high flows producing mean depths in excess of 6 metres (20 ft) and velocities in excess of 3 metres per second (10 ft/s). At high values of streamflow when the rivers are competent to move almost all sizes of particles on the streambed, bedload-transport rates are correlative with a predictable proportion of stream-power expenditure. As streamflow decreases and the river loses competence to transport the coarser bed particles, the channel bottom becomes armored and limits the availability of smaller sized material. As the channel becomes armored, the smaller material is transported at efficiencies that are greatly reduced from those predicted. Both rivers have a deficiency in bed material of intermediate sizes. Thus, median particle size of bedload shifts abruptly from very coarse gravel when all sizes of material are moving, to coarse sand when the channel bottom becomes armored.

INTRODUCTION

Development by the U.S. Army Corps of Engineers to convert the Columbia and Snake Rivers in the northwestern United States to a navigable channel for a distance as far inland as Lewiston, Idaho, is nearing completion with the construction of Lower Granite Dam. As shown in figure 1, Lower Granite Dam is being built across the Snake River about 50 kilometres (30 mi) below the confluence of the Snake and Clearwater Rivers at Lewiston, Idaho. Backwater on the Snake River and Clearwater River arms of the reservoir in back of Lower Granite Dam has necessitated construction of levees in the Lewiston area. Design height of the levees is based on the dual, but opposing, criteria of adequate protection and minimum cost. To define this height of levee, exacting hydrologic computations must be made, and these computations must be based on known sediment-transport characteristics and hydraulic- and channel-geometry data. Existing data prior to 1972, especially for sediment transport, were insufficient to allow the necessary precision in computations. Therefore, the U.S. Geological Survey and the U.S. Army Corps of Engineers began a cooperative study in March 1972 to collect the necessary data.

Tabulations of data collected through July 1974 have been published (Emmett and Seitz, 1973, 1974), and it is expected that annual supplements of data will continue to be published. A paper describing the

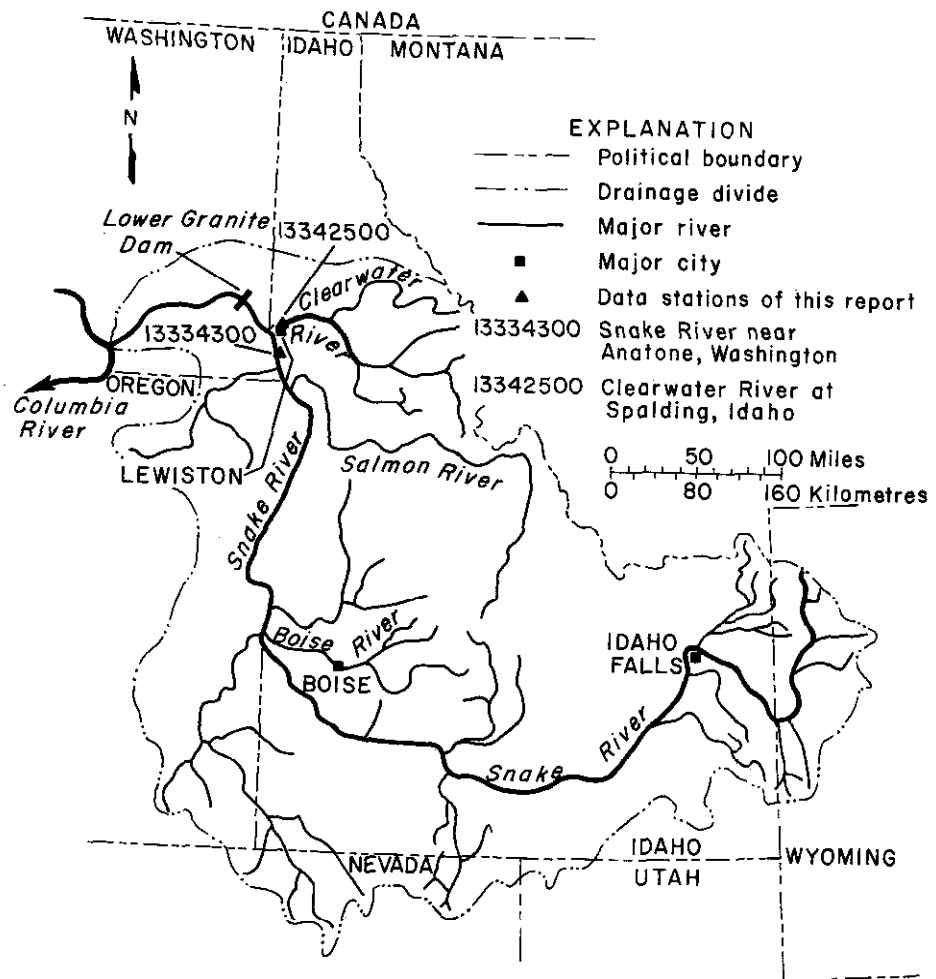


Figure 1.--Generalized map of Snake River basin showing location of two measuring sites near Lewiston, Idaho.

total loads transported by the two rivers is currently being prepared (Emmett, 1975). The purpose of this article is to explain the significance of the bedload-transport data already published.

HYDRAULIC- AND CHANNEL-GEOMETRY DESCRIPTION

The drainage area of the Snake River at Lower Granite Dam (see figure 1) is a little over 260,000 square kilometres (1.0×10^5 mi²). At the upstream end of the impoundment in the Lewiston area, U.S. Geological Survey gaging stations available as locations to measure the discharge of water and sediment into the reservoir area include station 13334300, Snake River near Anatone, Washington, and station 13342500, Clearwater River at Spalding, Idaho.

The Snake River station is located about 50 river kilometres (30 mi) above the confluence of the two rivers and has a drainage area of about 240,000 square kilometres (9.3×10^4 mi²), of which about 190,000 square kilometres (7.3×10^4 mi²) are upstream of flow-regulating dams. The Snake River station was established in 1958, and since that time, the maximum discharge that has occurred is 6,500 cubic metres per second (2.3×10^5 ft³/s) and the mean annual discharge is 1,000 cubic metres per second (3.6×10^4 ft³/s).

The Clearwater River station is located about 20 river kilometres (12 mi) above the confluence and has a drainage area of about 25,000 square kilometres (9.6×10^3 mi²). Since 1971, impoundment of water in back of Dworshak Dam on the North Fork has decreased the unregulated drainage area by about 6,200 square kilometres (2.4×10^3 mi²). The Clearwater River station has been in continuous operation since 1925, and since that time, the maximum flow that has occurred is 5,000 cubic metres per second (1.8×10^5 ft³/s), and the mean annual discharge is 450 cubic metres per second (1.6×10^4 ft³/s).

All measurements were obtained from cableway sections at the gaging-station locations. Tables 1 and 2 list for the Snake and Clearwater Rivers, respectively, values of streamflow at the times of bedload sampling, and the corresponding values of hydraulic and channel parameters. Values of width are at the water surface; depths are mean values (flow area divided by width); and velocities are mean values (discharge divided by flow area). Both cableway sections are located in relatively flat reaches of river. As streamflows increase, these flat reaches of river are progressively drowned and local water-surface slopes steepen.

Unit stream power, ω , is defined as the product of the unit weight of water, velocity, depth, and slope (γVDS) and is a parameter useful in presenting the transport rate of bedload sediment (Bagnold, 1966). Shear stress at the bed, τ , is defined as the product of the unit weight of water, depth, and slope ($\gamma DS = \omega/V$) and is a parameter useful in predicting the maximum size of bedload particles. Values of maximum particle size listed in tables 1 and 2 correspond to the mean values of shear stress listed (see, for example, Leopold, Wolman, and Miller, 1964, p. 170). However, at both river sections, maximum depth is about 1.5 to 2 times greater than mean depth, so that at the cross-channel point of maximum shear stress, maximum bedload-particle size may be on the order of about twice that listed in tables 1 and 2.

Table 1.--Snake River near Anatone, Washington
1972-74 Summary of Hydraulic Parameters

Date	Water discharge Q (m ³ /s)	Surface width W (m)	Mean depth D (m)	Mean velocity V (m/s)	Water- surface slope S (m/m)	Unit stream power ω (kg/m-s)	Mean shear stress τ (g/cm ²)	Maximum particle size associated with τ (mm)
5/10/72	2,190	181	4.60	2.62	0.00094	11.35	0.43	35.6
5/19/72	2,920	189	5.24	2.93	.00109	16.73	.57	46.8
6/02/72	3,770	197	6.04	3.17	.00124	23.73	.75	61.2
6/14/72	2,600	184	4.94	2.77	.00103	14.11	.51	41.6
4/18/73	780	155	3.26	1.51	.00056	2.76	.18	14.8
5/01/73	880	158	3.38	1.62	.00059	3.23	.20	16.4
5/15/73	1,130	165	3.72	1.83	.00067	4.56	.25	20.4
5/17/73	1,520	171	4.05	2.15	.00078	6.79	.32	26.0
5/21/73	1,610	172	4.18	2.23	.00080	7.43	.33	27.2
5/23/73	1,440	171	3.99	2.07	.00074	6.12	.29	24.0
6/06/73	1,020	162	3.60	1.74	.00064	4.01	.23	18.8
6/07/73	1,140	166	3.72	1.84	.00067	4.60	.25	20.4
6/11/73	1,050	162	3.63	1.77	.00065	4.17	.23	19.2
6/13/73	1,080	163	3.66	1.80	.00066	4.35	.24	19.6
6/18/73	1,020	162	3.60	1.74	.00064	4.01	.23	18.8
6/19/73	930	158	3.47	1.65	.00060	3.43	.21	17.2
2/06/74	1,090	165	3.66	1.81	.00067	4.45	.24	20.0
3/13/74	1,420	171	3.99	2.09	.00076	6.35	.30	24.8
3/26/74	1,840	177	4.36	2.38	.00087	9.03	.38	31.2
4/11/74	2,350	184	4.72	2.71	.00098	12.57	.46	38.0
4/24/74	2,410	186	4.75	2.74	.00100	13.05	.47	38.8
5/07/74	3,000	191	5.39	2.99	.00112	18.05	.61	49.6
5/14/74	2,660	189	5.06	2.86	.00105	15.22	.53	43.6
5/16/74	2,350	184	4.72	2.71	.00098	12.57	.46	38.0
5/22/74	1,950	180	4.42	2.47	.00090	9.83	.40	32.4
6/05/74	4,390	201	6.58	3.35	.00136	30.03	.89	73.2
6/11/74	3,090	192	5.46	3.02	.00113	18.43	.62	50.4
6/75/74	4,670	204	6.77	3.44	.00140	32.65	.95	77.6

Table 2.--Clearwater River at Spalding, Idaho
1972-74 Summary of Hydraulic Parameters

Date	Water discharge Q (m ³ /s)	Surface width W (m)	Mean depth D (m)	Mean velocity V (m/s)	Water- surface slope S (m/m)	Unit stream power ω (kg/m-s)	Mean shear stress τ (g/cm ²)	Maximum particle size associated with τ (mm)
5/10/72	1,590	140	5.00	2.26	0.00037	4.17	0.19	15.2
5/18/72	2,350	143	5.73	2.83	.00050	8.13	.29	23.6
6/01/72	2,740	146	6.06	3.11	.00056	10.56	.34	28.0
6/15/72	1,510	140	4.91	2.18	.00035	3.74	.17	14.0
4/17/73	289	125	3.29	.69	.00009	.21	.02	2.4
4/30/73	312	125	3.35	.73	.00010	.24	.03	2.8
5/14/73	419	128	3.54	.91	.00013	.40	.04	3.6
5/16/73	736	133	3.96	1.37	.00020	1.06	.08	6.4
5/22/73	680	133	3.93	1.30	.00019	.94	.07	6.0
5/24/73	680	133	3.93	1.30	.00019	.94	.07	6.0
6/05/73	388	128	3.51	.85	.00012	.36	.04	3.6
6/08/73	467	130	3.63	.98	.00014	.48	.05	4.0
6/12/73	382	128	3.47	.85	.00012	.34	.04	3.2
6/13/73	362	126	3.44	.81	.00011	.31	.04	3.2
6/19/73	496	130	3.66	1.02	.00014	.54	.05	4.4
6/20/73	365	126	3.44	.82	.00011	.31	.04	3.2
2/05/74	722	134	3.96	1.37	.00019	1.03	.07	6.0
3/13/74	892	136	4.18	1.58	.00023	1.52	.10	8.0
3/27/74	850	136	4.11	1.54	.00022	1.40	.09	7.6
4/10/74	1,200	139	4.54	1.94	.00029	2.55	.13	10.8
4/23/74	1,130	137	4.42	1.86	.00028	2.31	.12	10.0
5/08/74	1,810	143	5.24	2.45	.00040	5.15	.21	17.2
5/15/74	793	134	4.05	1.46	.00021	1.25	.09	7.2
5/23/74	623	133	3.84	1.22	.00017	.79	.06	5.2
6/04/74	1,730	142	5.15	2.38	.00039	4.78	.20	16.4
6/06/74	2,270	145	5.64	2.77	.00047	7.36	.26	21.6
6/12/74	1,700	142	5.12	2.35	.00038	4.57	.20	16.0
6/17/74	3,512	149	6.64	3.60	.00068	16.27	.45	37.2
6/20/74	3,120	149	6.34	3.35	.00062	13.18	.39	32.0

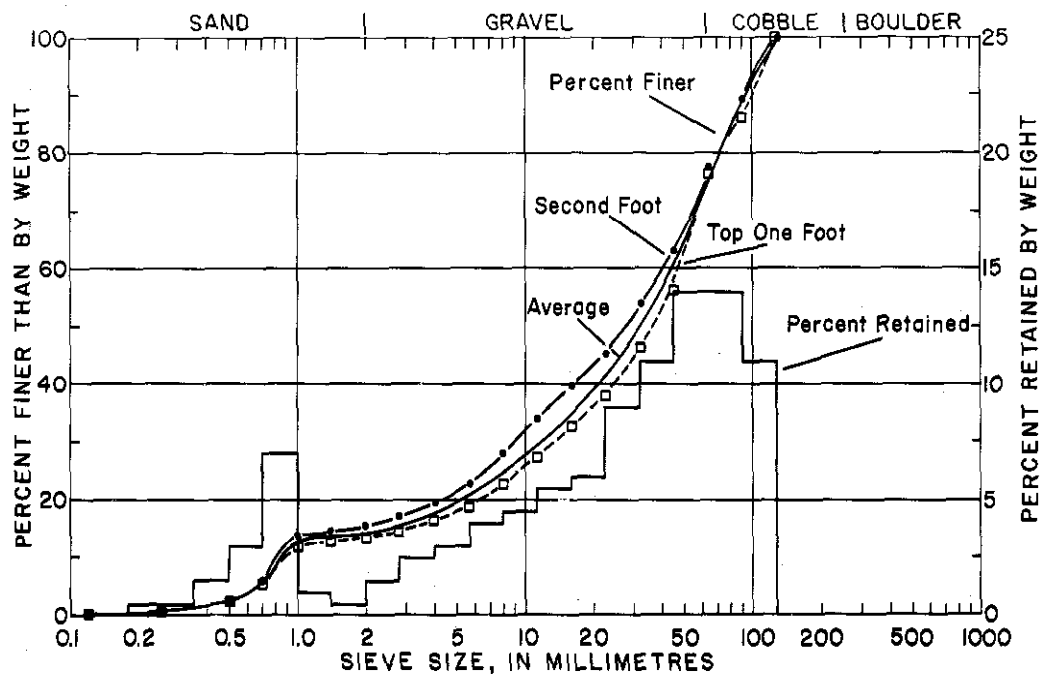


Figure 2.--Size distribution of bed-material sediment, confluence of Snake and Clearwater Rivers near Lewiston, Idaho.

The bed-material particle sizes were determined by sieve analyses of bulk samples and by the Wolman (1954) method of pebble count. Both rivers have poorly sorted bed material ranging in size from fine sand to large cobbles with a few boulders. The size distributions by sieve analysis presented in figure 2 and determined for samples obtained at the confluence of the two rivers are typical of either river. The median particle size, d_{50} , averages about a coarse gravel (32 mm) and shows a slight decrease in size with increasing depth below the streambed surface. This decrease in size is related to the formation of a cobble pavement at the streambed surface. Pebble counts of surficial bed material reflect this cobble pavement and show a median particle size of about 64 millimetres for either river. An analysis of dominant bed-material particle-size classes shows a bimodal distribution by weight with modes at the coarse-sand fraction and at the very-coarse-gravel to small-cobble fraction.

COLLECTION OF BEDLOAD-TRANSPORT DATA

Bedload samplers were of the Helley-Smith type (Helley and Smith, 1971). The bedload sampler in use during 1972 was of conventional design with a 76-millimetre (3-in) square orifice, but externally weighted to about 70 kilograms (150 lb) to enable use in deep, fast rivers. The bedload sampler used in 1973-74 incorporated a 152-millimetre (6-in) square orifice in a geometric scale-up of the standard Helley-Smith design. It was internally weighted to about 75 kilograms (165 lb) by a lead-filled frame, which allowed a streamlined design and increased stability. For each date of bedload-data collection, samples were collected at each of about 20 equally spaced cross-channel locations and for two traverses of the stream. Sampling duration was either 30 or 60 seconds at each location. Bedload-sediment samples were individually tagged, air dried, and sieved. The size distribution was determined by the weight percentage of each size fraction retained on sieves that had incremental mesh-size openings differing by a factor of $\sqrt{2}$, or 1.414.

It is emphasized that the Helley-Smith bedload sampler has not yet been fully calibrated, and the data of bedload transport presented in this report may have to be modified. However, field calibration of the sediment-trapping characteristics of the bedload sampler (Emmett, 1975) tentatively indicates that the sampler efficiency through the range of usually encountered bedload-transport rates may be near 100 percent.

DISCUSSION OF BEDLOAD-TRANSPORT DATA

Tables 3 and 4 for the Snake and Clearwater Rivers, respectively, list the bedload-transport data collected through 1974. The discharge listed is the mean during the several hours of data collection each day. Stream power has been previously defined. Bedload transport is expressed in terms of unit transport rate and load for the day. The median bedload-particle size, d_{50} and the sieve size that retained the maximum-size particle, were determined from the composite of sieve analyses for a given day. Stream efficiency is the unit bedload-transport rate (expressed as immersed rather than dry weight) divided by unit stream power, and will be discussed in more detail later.

The only researcher known to theoretically describe bedload transport without recourse to experimentally determined coefficients is Bagnold (1966). Bagnold's description is derived from general physics and, briefly, his derivation shows that the rate of bedload transport should be a function of the rate of stream-power expenditure. He later suggested (Bagnold, 1973) that at low values of stream power (low stream-flow), transport rate increases rapidly with increase of stream power, but at some higher value of stream power, the further increases of transport become a direct and linear function of stream power.

Using direct measurements of bedload-transport rates from a river in Wyoming, Leopold and Emmett (1975) confirmed Bagnold's hypothesis.

Table 3.--Snake River near Anatone, Washington
1972-74 Summary of Bedload-Transport Parameters

Date	Water discharge Q (m^3/s)	Unit stream power ω ($kg/m-s$)	Bedload Rate I_B ($kg/m-s$)	Bedload transport Load G_B (t/day)	Median bedload particle size d_{50} (mm)	Maximum bedload particle retained on sieve size (mm)	Stream efficiency (percent)
5/10/72	2,190	11.35	0.00589	92.5	0.63	16	0.03
5/19/72	2,920	16.73	.01440	234	.48	16	.05
6/02/72	3,770	23.73	.02366	402	.50	45	.06
6/14/72	2,600	14.11	.00832	132	.49	5.7	.04
4/18/73	780	2.76	.00004	.59	.49	1.0	.00
5/01/73	880	3.23	.00038	5.33	.45	4.0	.01
5/15/73	1,130	4.56	.00303	43.2	.50	16	.04
5/17/73	1,520	6.79	.01811	267	.57	45	.17
5/21/73	1,610	7.43	.02708	403	.50	45	.23
5/23/73	1,440	6.12	.01474	217	.46	5.7	.15
6/06/73	1,020	4.01	.00713	78.7	.70	32	.11
6/07/73	1,140	4.60	.01772	251	.92	45	.24
6/11/73	3,050	4.17	.00692	96.2	.75	32	.10
6/13/73	1,080	4.35	.00496	69.9	.90	32	.07
6/18/73	1,020	4.01	.00415	82.0	.60	11.3	.06
6/19/73	930	3.43	.00503	68.9	.95	45	.09
2/06/74	1,090	4.45	.01613	230	66	64	.22
3/13/74	1,420	6.35	.05761	849	36	64	.57
3/26/74	1,840	9.03	.07711	1,180	43	64	.53
4/11/74	2,350	12.57	.15308	2,440	42	90	.76
4/24/74	2,410	13.05	.09478	1,520	35	90	.45
5/07/74	3,000	18.05	.09315	1,530	57	90	.32
5/14/74 ^{1/}	2,660	15.22	.10477	1,710	28	90	.43
5/16/74	2,350	12.57	.05912	943	.65	45	.29
5/22/74	1,950	9.83	.00995	154	.70	32	.06
6/05/74	4,390	30.03	.01109	193	.34	22.6	.02
6/11/74	3,090	18.43	.03525	584	.62	45	.09
6/24/74 ^{1/}	4,670	32.65	.13419	2,370	.37	11.3	.26

^{1/} Less than two traverses of stream completed on this date.

Table 4.—Clearwater River at Spalding, Idaho
1972-74 Summary of Bedload-Transport Parameters

Date	Water discharge Q (m^3/s)	Unit stream power ω ($kg/m-s$)	Bedload transport Rate I_B ($kg/m-s$)	Load G_B (t/day)	Median bedload particle size d_{50} (mm)	Maximum bedload particle retained on sieve size (mm)	Stream efficiency (percent)
5/10/72	1,590	4.17	0.01724	209	0.47	22.6	0.26
5/18/72	2,350	8.13	.07337	907	35	45	.56
6/01/72	2,740	10.56	.05929	748	30	45	.35
6/15/72	1,510	3.74	.02055	249	.53	2.0	.34
4/17/73	289	.21	.00078	8.56	.49	1.0	.24
4/30/73	312	.24	.00886	95.2	.64	2.0	2.33
5/14/73	419	.40	.00190	21.0	.48	1.4	.29
5/16/73	736	1.06	.01999	229	.49	1.4	1.18
5/22/73	680	.94	.00518	59.3	.50	2.8	.34
5/24/73	680	.94	.00439	50.3	.50	2.0	.29
6/05/73	388	.36	.00008	.94	.46	1.4	.01
6/08/73	467	.48	.00055	6.21	.52	2.8	.07
6/12/73	382	.34	.00022	2.40	.48	4.0	.04
6/13/73	362	.31	.00010	1.22	.51	1.4	.02
6/19/73	496	.54	.00147	16.5	.52	1.4	.17
6/20/73	365	.31	.00005	.60	.48	1.4	.01
2/05/74	722	1.03	.00303	35.1	.50	2.0	.18
3/13/74	892	1.52	.00755	97.1	.68	45	.31
3/27/74	850	1.40	.00137	16.1	.52	5.7	.06
4/10/74	1,200	2.55	.00417	49.9	.52	32	.10
4/23/74	1,130	2.31	.00732	90.4	.54	45	.20
5/08/74	1,810	5.15	.03855	476	52	64	.47
5/15/74	793	1.25	.00291	33.7	.53	11.3	.15
5/23/74	623	.79	.00049	5.67	.45	1.0	.04
6/04/74 ^{1/}	1,730	4.78	.09577	1,210	.53	16	1.24
6/06/74	2,270	7.36	.01386	173	.57	64	.12
6/12/74	1,700	4.57	.02641	323	.50	45	.36
6/17/74 ^{1/}	3,512	16.27	.07993	1,030	.47	32	.31
6/20/74	3,120	13.18	.25815	3,330	61	90	1.22

^{1/} Less than two traverses of stream completed on this date.

Combined with meager but increasing amounts of bedload-sampling data from other rivers, Leopold and Emmett generalized what they think represent the relations of bedload transport to stream power for many rivers, if an adequate amount of detritus is available in the channel ready for movement. These relations are shown as the dashed lines in figure 3. The family of dashed-line curves is labeled for values of median size of transported material, d_{50} , in millimetres. At high values of stream power when bedload-transport rates become directly proportional to stream power, stream efficiency, or the proportion of stream power utilized for bedload transport, becomes constant. Values of the constant are suggested as about 30 percent for medium sand (0.3 mm), 15 percent for coarse sand (1-2 mm), 5 percent for medium gravel (8-16 mm), and 2-3 percent for very coarse gravel (32-64 mm).

Data from the Snake and Clearwater Rivers (tables 3 and 4) are also shown on figure 3. The solid symbols represent bedload-transport rates having maximum-size bedload particles of very coarse gravel or larger (>32 mm), the open symbols represent data with maximum-size bedload particles of fine to coarse gravels (4-32 mm), and the lined symbols represent data with maximum-size bedload particles of very fine gravel or smaller (≤ 4 mm). The number beside each data point represents the median particle size of the approximately 40 individual bedload samples which compose each piece of data. Using the interrelations of stream power, mean shear stress, and particle size (tables 1 and 2), the approximate maximum particle size expected in motion for a given value of stream power is shown at the top of figure 3. But, as previously mentioned, maximum shear stress may be about twice mean shear stress, and thus maximum particle sizes in motion may be about twice those shown at the top of figure 3.

At higher values of stream power (approximately 5 to 25 kg/m-s), comparison of values of particle size from the top axis of figure 3 to the bed-material size shown in figure 2 indicates that the river is competent to transport almost all sizes of bed material available in the river. Data points in this range of stream power, generally represented by the solid symbols in figure 3, tend to verify the Leopold-Emmett relation for rivers with beds of very coarse gravel (50 mm).

At the highest values of bedload-transport rate, median particle size of bedload reflects median particle size of bed material. As stream power and bedload-transport rate decrease, the stream loses competence to transport the coarser bed particles, and the median particle size of bedload decreases. The strongly bimodal size distribution of bed material (figure 2) becomes prominent, and the median particle size of bedload generally reflects one or the other of the two modes.

Because the volume of bedload moved at the higher transport rates is considerably greater than that at lower transport rates, the size distribution of the total quantity of bedload moved during a period of time corresponds more closely to the size distribution of bedload at high transport rates. This is shown in figure 4 by the size-distribution

APPROXIMATE MAXIMUM PARTICLE SIZE, IN MILLIMETRES,
IN MOTION USING MEAN SHEAR STRESS - PARTICLE SIZE RELATIONS

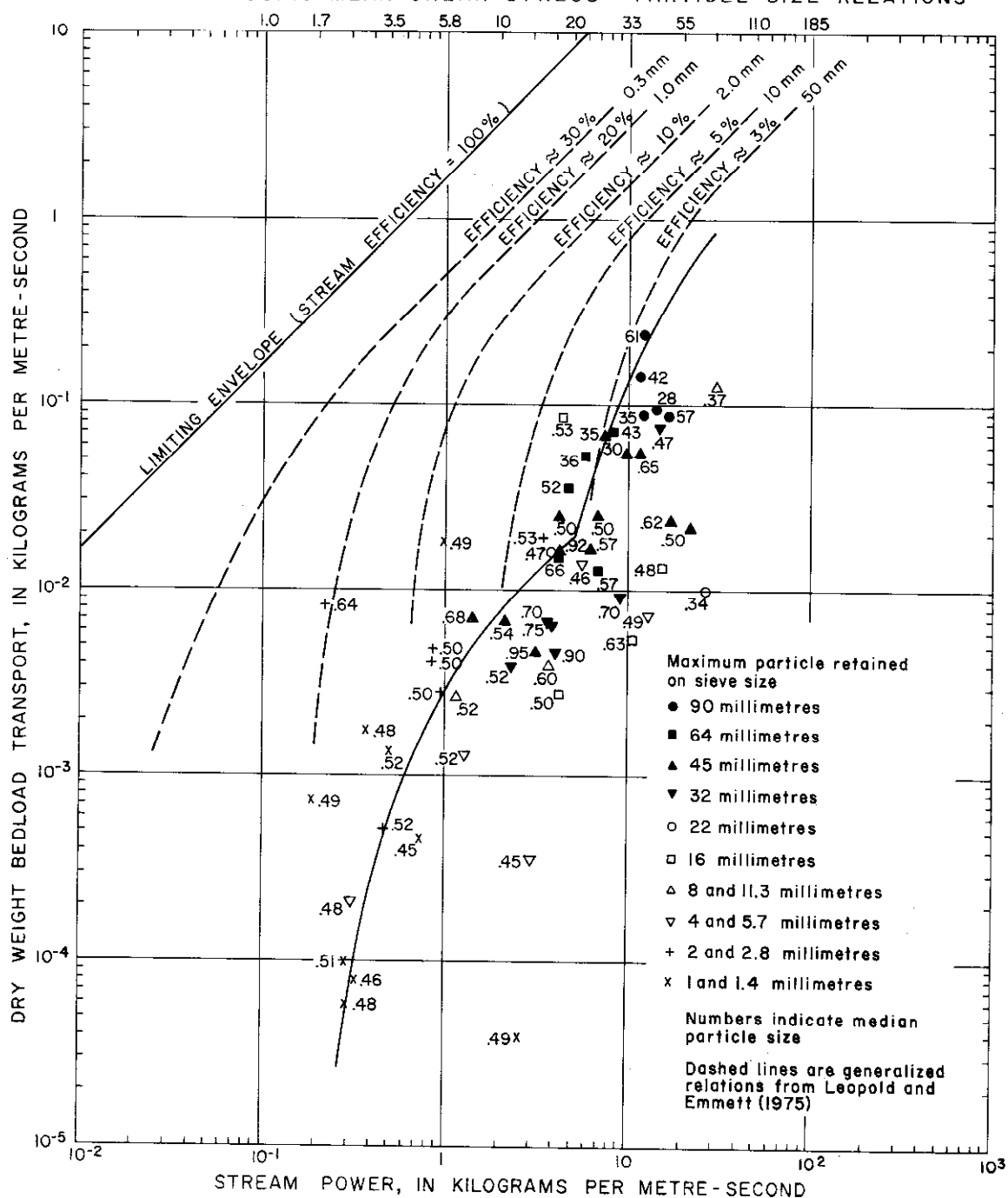


Figure 3.--Bedload transport as a function of stream power.

curves of the collective material for each river during the 1972-74 period. The bimodal size distribution of bedload sediment is very pronounced; the graph clearly illustrates that when particles of size greater than about 32 millimetres cease to move, median particle size of bedload material abruptly shifts to the coarse-sand mode.

As coarse particles cease to move, an increasingly larger percentage of the streambed becomes armored and limits the availability of smaller material. In effect, actual quantities of bedload transported are less than the stream is competent to transport.

The net effect of limiting the availability of material is shown in figure 3. The lined symbols, representing bedload with maximum particle sizes of less than 4 millimetres and with a median particle size of about 0.5 millimetre, lie well below the Leopold-Emmett generalized relation for the corresponding size. Stream efficiency is greatly reduced. At a unit stream power of about 1 kilogram per metre-second and utilizing the generalized relation for 1.0 millimetre material, stream efficiency is reduced about 200-fold.

The open symbols in figure 3 generally represent bedload transport that is intermediate between the large decrease in predicted stream efficiency when channel armoring severely limits availability of material and the nearly negligible decrease in predicted stream efficiency when the river is competent to move all sizes of bed material. For the open-symbol data, the decrease in stream efficiency under that predicted is about 50-fold, but is limited in application to actual data because of the shortage of bed material of intermediate sizes.

SUMMARY STATEMENT

For the Snake and Clearwater Rivers near Lewiston, Idaho, two relations are necessary to describe bedload-transport characteristics. One relation is applicable to low values of stream power when coarse particles are not moving and the availability of fine material is limited; the other is applicable to higher values of stream power when almost all material is moving. These two relations are shown as the solid line in figure 3. Both relations have the same form as the dashed-line relations proposed by Leopold and Emmett (1975). The relation for the Snake and Clearwater Rivers at low stream powers is probably analogous to the Leopold-Emmett relations but shows the effect of decreased stream efficiency as the channel becomes armored and protects the supply of smaller bed material. The Snake-Clearwater River relation at higher stream powers is nearly identical to the Leopold-Emmett relation that was generalized for very-coarse-gravel rivers. And, the break in the Snake-Clearwater River relation at intermediate values of stream power is due to the bimodal size distribution of bed material; that is, because there is an absence of intermediate sizes of gravel, the channel bed is either armored or moving.

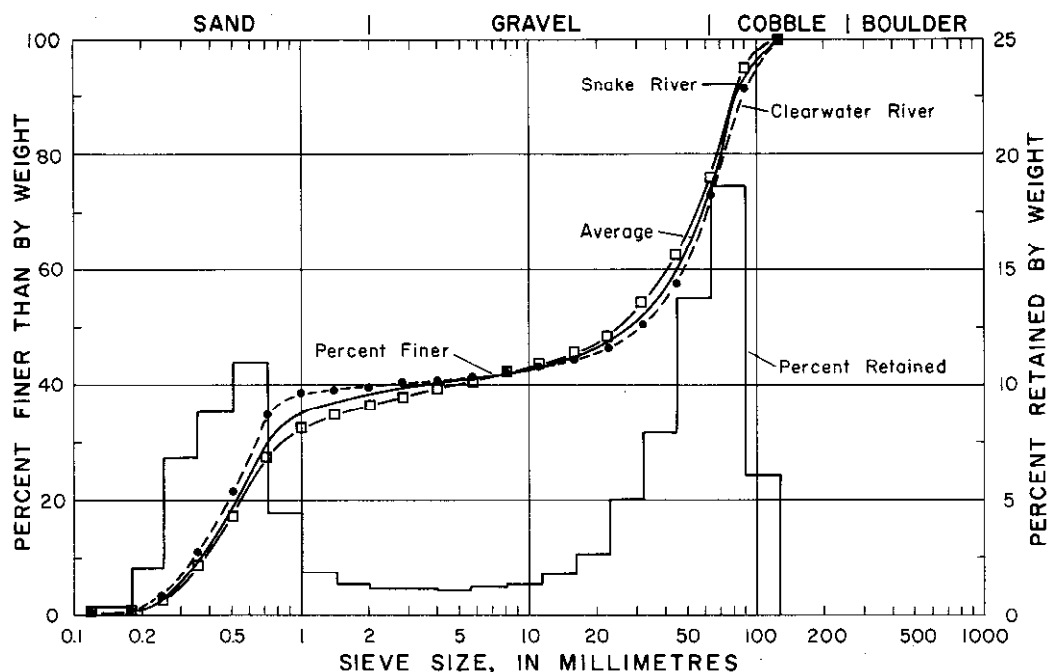


Figure 4.--Size distribution of bedload sediment, Snake and Clearwater Rivers near Lewiston, Idaho.

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SEDIMENT STORAGE IN CHANNELS DRAINING
SMALL FORESTED WATERSHEDS IN THE
MOUNTAINS OF CENTRAL IDAHO

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ABSTRACT

The role of sediment storage was studied in channels draining seven small forested watersheds in central Idaho. Data collection included an inventory of sediment deposited behind channel obstructions, detailed surveys of channel cross sections, and measurements of annual sediment yields in sediment detention reservoirs.

Extremely large volumes of sediment were trapped behind channel obstructions in the 1973 and 1974 sample years; only about 10 percent of the sediment stored appeared as sediment yield. Most obstructions shifted from one year to the next; so the stability of the remaining sediment stored in the channels was questionable.

The series of detailed channel cross sections was used in evaluating annual changes in the total volume of sediment stored on channel bottoms. Considerable aggregation occurred during the low flow year (1972-73), whereas degradation occurred during the high flow year (1973-74). Annual watershed erosion was determined by entering the annual sediment yields and the annual change in stored sediment into a continuity equation.

INTRODUCTION

The importance of channel sediment storage can be illustrated with a continuity equation for watershed erosion-sediment yield as follows:

$$\begin{array}{l} \text{Outflow (sediment yield)} = \text{inflow (watershed erosion)} - \Delta \text{ storage} \\ \text{(sediment stored in channels)} \end{array} \quad (1)$$

As used here, watershed erosion refers to the amount of material from watershed slopes and delivered to the stream channel or eroded from streambanks, or both, and sediment storage refers to material deposited on the streambed. Usually, the change in the sediment storage term of the equation is ignored and sediment yield for a given period is assumed to equal erosion for that period. This stance is tantamount to assuming that sediment storage is in a steady state, a situation that seldom occurs

(Trimble, 1975; Platts and Megahan, 1975). The sediment storage term can be particularly significant when it is large in relation to the other terms of the equation. This can occur where channel conditions are conducive to storing large amounts of sediment, where a high proportion of the total sediment yield consists of bedload, or where both conditions prevail.

The potential for proportionally large channel sediment storage increases as one progresses upstream to lower order streams. This is particularly true of forested mountainous areas where there is a greater probability of slope profile irregularities and channel obstructions, such as logs and branches. The combination creates a stair-step effect; so deposition behind obstructions must be considered in addition to the usual concerns with degradation and aggradation of the channel bottom.

To date, most research dealing with the effects of obstructions in mountain streams has been related to the potential for biological impacts on fishery resources (Froelich, 1973; Meehan et al., 1969). Heede (1972) studied the effect on stream equilibrium of steps created by obstructions in two Colorado streams. He found that average step length between obstructions decreased as channel gradient and size of bed material increased, thus providing effective adjustment for increased flow energies.

In the present investigation, the role of sediment storage was studied in headwater channels draining small, undisturbed watersheds in the mountains of central Idaho. Study objectives were to find out: (1) how much sediment storage occurs, (2) how storage varies over time, and (3) how sediment storage relates to sediment yields. Studies will be continued to determine how logging and road construction affect sedimentation processes in the study watersheds.

DESCRIPTION OF THE STUDY AREA

The seven watersheds are on the Silver Creek study area in the Middle Fork of the Payette River drainage near Crouch, Idaho. The area is representative of conditions in the Idaho Batholith, a 41,400 km² (16,000 mi²) expanse of granitic bedrock in central Idaho (Figure 1). This is a mountainous area characterized by steep slopes; shallow, extremely erodible, and coarse-textured soils; and high climatic hazards resulting from rainfall, snowmelt, or both. Erosion hazards are high in this area, and soil disturbances, natural or man-made, can greatly accelerate erosion and consequent sedimentation (Megahan, 1975). Much sedimentation occurs as bedload because of the extremely coarse texture of the soils. Streamflows and sediment yields exhibit marked seasonal variation in response to large winter storms, spring snowmelt, and long, dry, summer periods. Descriptive data for the study watersheds are presented in Table 1. All but one of the study watersheds are undisturbed; the Ditch Creek drainage contains a low-standard road constructed during the 1930's.

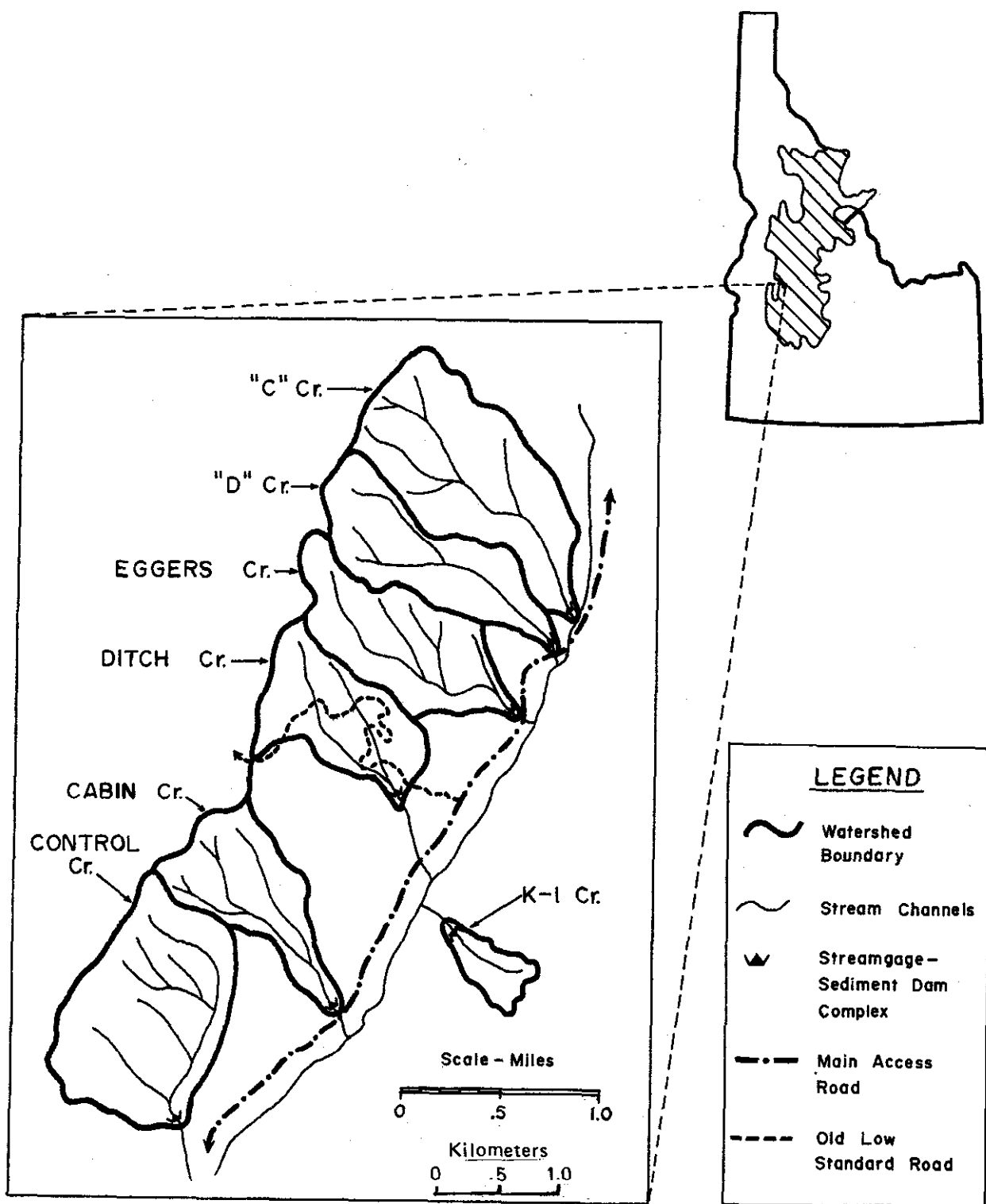


Figure 1.--Location map of the study area in the Idaho Batholith.

Table 1. Descriptive data for seven study watersheds on the Silver Creek study area

Drainage name	Area	Mid-elevation ^{a/}	Dominant aspect	Mean channel gradient ^{b/}	Total channel length ^{c/}	Stream order ^{c/}	Average bank width
	km ² (mi ²)	m (ft)		(percent)	km (mi)		m (ft)
C	1.94 (0.75)	1,779 (5,835)	SE	21.5	9.5 (5.9)	3	1.5 (5.0)
D	1.22 (0.47)	1,765 (5,790)	SE	24.2	5.5 (3.4)	2	1.1 (3.6)
Eggers	1.29 (0.50)	1,733 (5,685)	SE	22.0	5.8 (3.6)	3	1.1 (3.5)
Ditch	1.06 (0.41)	1,631 (5,350)	SE	20.8	2.9 (1.8)	2	0.8 (2.7)
Cabin	1.04 (0.40)	1,533 (5,030)	SE	14.9	5.1 (3.2)	3	1.0 (3.3)
Control	2.02 (0.78)	1,597 (5,240)	SE	15.2	6.8 (4.2)	3	1.1 (3.5)
K-1	0.26 (0.10)	1,623 (5,325)	NW	31.5	1.0 (0.6)	1	1.0 (3.3)
Average	1.27 (0.49)	1,666 (5,465)	--	21.4	5.1 (3.2)	--	1.1 (3.6)

^{a/} (Maximum elevation - minimum elevation)/2.

^{b/} (Total relief/length main channel to the upper ridge) × 100.

^{c/} Taken from a 3.1 cm per km (2 inches to the mile) planimetric map.

METHODS

Channel sediment storage is determined from samples taken at stations located every 152 meters (500 feet) along the dominant channel in each study watershed. The first station is 30 meters (100 feet) above a sediment basin at the mouth of each drainage. Each sample station consists of five permanently marked cross sections 3 meters (10 feet) apart and a 15-meter (50-foot) section of channel on both sides of the cross sections (Figure 2). Annual data collection includes detailed engineering surveys of each cross section; measurements of sediment deposited behind obstructions within each 43-meter (140-foot) sample station; and evaluation of channel properties that characterize the aquatic environment, such as bank conditions, size of streambed materials, properties of pools, and type and amount of bank vegetation. An average

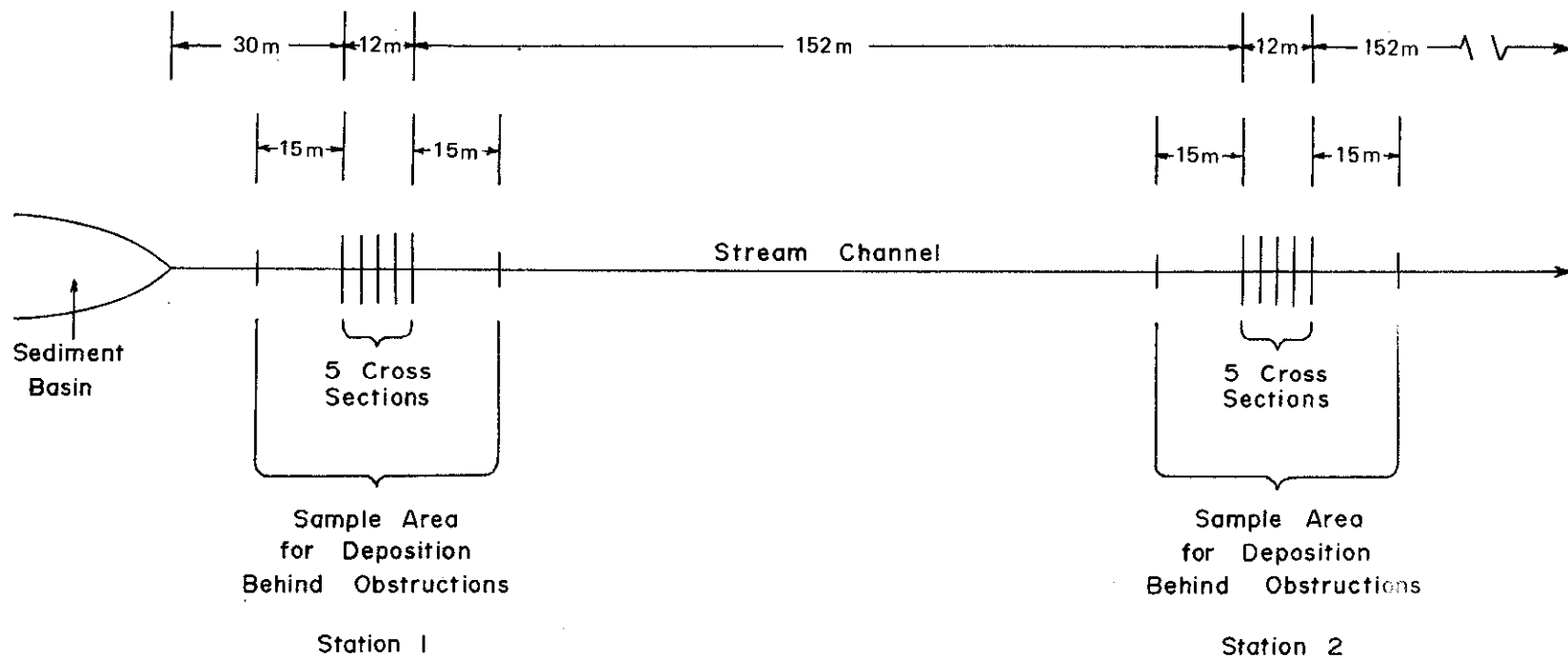


Figure 2.--Schematic of stream sampling design.

of 10 sample stations on each study channel provides an average of 28 percent of the study channel sampled for sediment deposition behind obstructions. Sample stations are located only on the dominant channel in each watershed; so only about 8.8 percent of the total channel length in each watershed is sampled (Table 2).

Data are collected annually during low flow periods in late July and August. Data for surveyed cross sections and aquatic habitat conditions are available for 3 years (1972-74); data for sediment deposited behind obstructions are available for only 2 years (1973 and 1974). In 1973, all discernible obstructions and associated sediment accumulations were measured, a procedure that proved to be too time consuming. The following year, sampling was restricted to sediment accumulations with the following minimum dimensions: height, 0.2 meter (0.6 foot); width, 0.3 meter (1.0 foot); and length, 0.6 meter (2.0 feet).

Table 2. Frequency of sampling on study watersheds

Stream	Number of stations	Number of cross sections	Length of study channel m (ft)	Amount of study channel sampled for obstructions (percent)	Amount of total channel length sampled for obstruction (percent)
C	14	70	2,198 (7,210)	27.2	6.3
D	8	40	1,210 (3,970)	28.2	6.2
Eggers	12	60	1,868 (6,130)	27.4	8.8
Ditch	11	55	1,704 (5,590)	27.5	16.1
Cabin	10	50	1,539 (5,050)	27.7	8.2
Control	11	55	1,704 (5,590)	27.5	6.9
K-1	4	20	552 (1,810)	30.9	17.7
Average	10	350	1,539 (5,050)	28.0	8.8

A computer program was developed to plot channel cross section surveys directly from field data and locate such key channel characteristics as size and type of bottom materials. Annual degradation or aggradation of the channel bottom is determined by overlaying plots of successive channel cross section surveys. Annual obstruction effects are presented on the basis of total number and type and the volume of sediment deposition.

A continuously recording stream gage and a sediment basin for determining annual sediment yields are operated at the mouth of each watershed. Sediment basins are surveyed twice a year in June and October by using a network of closely spaced cross sections. Generally, more than 95 percent of the total sediment yield for the year is measured during the June survey. Trap efficiencies of the detention reservoirs are estimated to average more than 75 percent because of the coarse soils on the study watersheds (Megahan, 1975).

RESULTS

According to data from cross section surveys, channel bottoms aggraded an average of 0.3 m^3 (11.9 ft^3) per 30 meters (100 feet) of channel length from 1972 to 1973. This increase was negated somewhat by channel bottom degradation that averaged 0.1 m^3 (3.3 ft^3) per 30 meters (100 feet) of length from 1973 to 1974. When the width of channel affected is taken into consideration, an average 1.55 cm (0.61 inch) of aggradation occurred from 1972 to 1973 and an average 0.58 cm (0.23 inch) of degradation from 1973 to 1974. Differences in sediment storage varied considerably by stream, although general trends by year were consistent in most cases. The changes in sediment storage in the channels were reflected in the sediment yields for the 2 years studied; yields were low in 1973 and high in 1974 (Table 3).

The change in sediment storage was most likely caused by unusual hydrologic conditions during the 1973 and 1974 water years. Precipitation and flows were unusually low in 1973 and unusually high in 1974 (Table 4). The low flows in 1973 were not capable of transporting all sediment supplied to the channel system; so considerable aggradation of bed material occurred. In contrast, high flows in 1974 caused degradation of the channel streambed.

Obstructions were an important factor regulating the amount of sediment storage in stream channels. Although the data presented (Table 5) for individual years are not comparable because the definition of obstruction was modified, the 2 years of data are still informative. In 1973 and 1974, respectively, there was an average of 3.9 obstructions detaining an average 0.28 m^3 (10 ft^3) of sediment and 3.0 obstructions detaining an average 0.34 m^3 (12 ft^3) of sediment per 100 feet of channel. This amounted to 1.1 m^3 (40.0 ft^3) of sediment per 30 meters (100 feet) of channel in 1973 and 1.0 m^3 (36.7 ft^3) of sediment per 30 meters (100 feet) of channel in 1974. Effects of obstructions on sediment storage varied considerably by individual channel; sediment storage increased in some channels from 1973 to 1974, but decreased in others (Table 5).

Table 3. Change in volume sediment storage in stream channel bottoms and sediment yields, 1972-73 and 1973-74

Stream	Average volume change per 100 feet of channel bottom		Average depth change		Annual sediment yields ^{a/}	
	1972-73	1973-74	1972-73	1973-74	1973	1974
	m ³ (ft ³)		cm (inches)		m ³ (ft ³)	
C	+0.65 (+23)	-0.08 (- 3)	+1.93 (+0.76)	-0.30 (-0.12)	6 (203)	29 (1,038)
D	+0.42 (+15)	-0.54 (-19)	+1.57 (+0.62)	-2.87 (-1.13)	2 (70)	26 (917)
Eggers	+0.20 (+ 7)	-0.06 (- 2)	+1.04 (+0.41)	-0.43 (-0.17)	4 (152)	13 (450)
Ditch	+0.28 (+10)	-0.31 (-11)	+1.91 (+0.75)	-2.16 (-0.85)	4 (151)	39 (1,392)
Cabin	+0.42 (+15)	-0.20 (- 7)	+2.39 (+0.94)	-0.94 (-0.37)	7 (238)	22 (769)
Control	+0.31 (+11)	+0.51 (+18)	+1.63 (+0.64)	+2.44 (+0.96)	2 (83)	20 (703)
K-1	+0.06 (+ 2)	+0.03 (+ 1)	+0.33 (+0.13)	+0.13 (+0.05)	1 (52)	7 (239)
Average	+0.34 (+11.9)	-0.09 (-3.3)	+1.55 (+0.61)	-0.58 (-0.23)	4 (136)	22 (787)

^{a/} Actual values uncorrected for trap efficiency.

A total of 680 obstructions were sampled during the 2 years of data collection. Logs were the most common obstructions, averaging about 38 percent of the total. Organic material, classified as debris (branches, bark, leaves), accounted for an additional 34 percent of the obstructions. The remaining 28 percent was made up of less mobile materials, rocks, roots, and stumps (Table 6).

Obstructions appeared to be randomly located along the channels. Only about 25 percent of the obstructions noted in 1973 remained in place in 1974 of these, about two-thirds were logs. Usually, locations of obstructions varied considerably as obstruction materials decomposed, were washed downstream, or both.

Table 4. Hydrologic data from seven study watersheds, 1965-74

Water year	Mean annual precipitation	Average instantaneous peak flow
	cm (inches)	m ³ /sec/km ² (ft ³ /sec/mi ²)
1965	108.0 (42.5)	0.24 (22.3)
1966	54.4 (21.4)	0.05 (4.3)
1967	81.8 (32.2)	0.08 (7.5)
1968	78.7 (31.0)	0.06 (5.1)
1969	82.0 (32.3)	0.11 (10.1)
1970	104.4 (41.1)	0.12 (10.8)
1971	105.4 (41.5)	0.18 (16.2)
1972	95.8 (37.7)	0.11 (10.5)
1973	67.1 (26.4)	0.05 (5.0)
1974	110.7 (43.6)	0.19 (17.8)

Table 5. Volume of sediment stored behind obstructions, 1973 and 1974

Drainage name	Number of obstructions per 30 meters (100 feet) of channel		Average sediment volume/ obstruction		Average sedi- ment volume/ 30 meters (100 feet) of channels		Total sediment behind obstruc- tions/total channel length	
	1973	1974	1973	1974	1973	1974	1973	1974
			m ³ (ft ³)		m ³ (ft ³)		m ³ (ft ³)	
C	4.5	3.2	0.52 (18.2)	0.60 (21.2)	2.34 (82.7)	1.93 (68.1)	729 (25,762)	600 (21,215)
D	6.4	3.6	0.36 (12.7)	0.46 (16.1)	2.31 (81.7)	1.62 (57.4)	415 (14,667)	292 (10,304)
Eggers	3.2	2.5	0.20 (7.0)	0.20 (7.0)	0.62 (22.0)	0.55 (19.4)	118 (4,182)	104 (3,688)
Ditch	3.8	3.1	0.18 (6.5)	0.25 (8.7)	0.70 (24.9)	0.78 (27.7)	67 (2,366)	75 (2,633)
Cabin	3.5	2.5	0.18 (6.2)	0.25 (8.9)	0.62 (21.8)	0.63 (22.3)	104 (3,683)	107 (3,768)
Control	2.9	2.8	0.12 (4.4)	0.20 (7.2)	0.35 (12.5)	0.57 (20.2)	78 (2,772)	127 (4,480)
K-1	3.6	4.3	0.18 (6.5)	0.29 (10.1)	0.66 (23.3)	1.22 (43.1)	21 (738)	39 (1,365)
Average	3.9	3.0	0.29 (10.2)	0.35 (12.2)	1.13 (40.0)	1.04 (36.7)	219 (7,739)	192 (6,779)

Table 6. Frequency of occurrence of obstructions by type, 1973 and 1974

Obstructions	Total number		Percent	
	1973	1974	1973	1974
Logs	133	123	34.5	41.8
Debris ^{a/}	107	121	27.8	41.1
Rocks	94	46	24.4	15.6
Roots	49	2	12.7	0.7
Stumps	2	2	0.6	0.8
Total	386	294	100.0	100.0

^{a/} Organic material other than logs, such as branches, bark, and leaves.

DISCUSSION AND CONCLUSIONS

Surprisingly large volumes of stored channel sediments were measured on the study watersheds, considering they represent undisturbed conditions for the most part. To illustrate, an average of less than 10 percent of sediment stored behind obstructions appeared as annual sediment yield for the 2 years of data collected. Many obstructions were transitory from one year to the next; so the stability of the remaining 90 percent of channel sediment in storage is questionable, especially under extreme flow conditions.

Sediment stored on the channel bottom changed considerably between years. During a low flow year, bottom sediment showed an average of 1.55 cm (0.61 inch) of aggradation and during a high flow year, an average of 0.58 cm (0.23 inch) degradation. Accumulations were largest immediately upstream from obstructions; depths of 0.3 meter (1.0 foot) or more were common. The biological implications of such extreme annual variations in bottom materials on small, undisturbed watersheds are significant; data suggest that aquatic ecosystems in these areas are frequently subjected to relatively large stresses even without the disturbing effects of man's activities. To further understand the influence of sediment storage on aquatic life, we are cooperating in a research effort with the University of Idaho, Department of Entomology. The responses of aquatic insects are being evaluated in the study streams, both before and after logging and road construction.

Using annual sediment yields from sediment basins and data from the cross section surveys, we can use the continuity equation (1) to determine average annual watershed erosion for the seven study watersheds (Table 7).

Table 7. Average annual watershed erosion calculations for study watersheds, 1972-73 and 1973-74

Period sampled	Sediment yield ^{a/}	Change in sediment storage on channel bottoms	Watershed erosion
		m ³ (ft ³)	
1972-73	5 (181)	+72 (+2,454)	75 (2,635)
1973-74	30 (1,049)	-12 (-419)	18 (630)

^{a/} Corrected for 75 percent trap efficiency.

An apparent inconsistency exists between high watershed erosion during the low flow year of 1973 and low watershed erosion during the high flow year of 1974. We have no data to verify the actual amount of material eroded from the watersheds and delivered to the channel during this period. However, we do have some past on-site erosion data that help explain the inconsistencies. From 1968 to 1972, data were collected from twenty-eight 40-m² (1/100-acre) erosion plots scattered throughout the study watersheds. During the 5-year sampling period, no correlation existed between on-site erosion and streamflow rates. Most annual on-site erosion (averaging about 75 percent) occurs from June through September as the result of dry creep and raindrop splash during small summer storms. Erosion of this type is completely unrelated to high streamflows that are caused by large volume inputs from spring snowmelt or winter rainstorms often coupled with snowmelt.

The continuity equation data in Table 7 allow us to compare the amount of channel erosion to erosion delivered to the stream channel from the remainder of the watersheds. Channel erosion is composed of both bank erosion and bottom erosion; bottom erosion was directly measured in this study, and the results are shown in Table 7. For the 1972-73 period, no channel bottom erosion occurred; instead, a net accumulation of 69 m³ (2452 ft³) of sediment was measured on the channel bottoms. However, channel bottom erosion resulted in a net loss of 12 m³ (419 ft³), from 1973 to 1974. About 40 percent of the total erosion occurring that year could be attributed to channel bottom erosion.

The question arises as to how much of the watershed erosion in Table 7 can be attributed to bank erosion because no precise measurements of bank erosion were made during the 1972-74 sample periods. Fortunately, various descriptive data were obtained for streambanks during the 1972-74 surveys.

Data included type and condition of bank vegetation, a qualitative rating of the severity of bank erosion, and an estimate of the predominant size of bank material. Only about 4 percent of the streambanks sampled at all cross sections were classified as actively eroding during the 3 sample years. To account for the volume of watershed erosion during the 2 years, erosion at these sites would have had to average 64 cm (25 inches) and 15 cm (6 inches) for 1973 and 1974, respectively. Actual bank recession at these sites could be a few inches at the most; by far the largest percentage of watershed erosion occurred on watershed slopes outside stream channels.

Actual measurements of soil erosion on a watershed scale are fraught with sampling and measurement difficulties that have been insurmountable to date. Determining delivery of eroded material to the stream channel creates an additional measurement problem. The approach used in this study provides an indirect measure of soil erosion that may provide a practical solution to this problem.

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THE SEDIMENTARY INFLUENCE OF A TRIBUTARY STREAM

GROWTH OF THE NIOBRARA DELTA

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ABSTRACT

There is evidence that a dynamic balance has been achieved between the sediment transport capabilities of two alluvial streams when graded conditions develop at their confluence. This quasi-equilibrium state implies a dual adjustability in the channel dimensions or slope of both streams as the individual hydrologic trends of the separate drainages vary discharges and sediment loads. When a sudden reduction in the frequency, magnitude and duration of flows on the main stem occurs, due to upstream regulation, this balance is interrupted and the tributary sediment contribution assumes controlling proportions. Such was the case at the mouth of the Niobrara River after the Missouri River reservoir system commenced operation in 1953. This paper discusses the growth of the Niobrara delta in the Missouri River channel over a 20-year period. Preliminary results of study investigations are summarized and prevailing trends identified.

INTRODUCTION

The geometry of flow channels at the confluence of two alluvial streams represents some quasi-equilibrium state that is dependent upon certain interrelationships between water and sediment discharges. E. W. Lane, in 1955, presented a qualitative assessment of this interrelationship by explaining that such a dynamic equilibrium is dependent upon a balance in stream discharge and slope with changes in the volume and grain size of the transported bed material load. Many empirical correlations have since been derived in attempts to provide quantitative or predictive relationships. This study effort related to the growth of the Niobrara delta will add to these attempts; however, the conclusions will hopefully be more definitive because of the quality and quantity of documented field measurements.

It is not intended that this paper shall present final study conclusions. That objective is some distance away in time because of the volume of necessary study assessments. Rather, the purpose of this paper is to present a brief summation of the physical characteristics of the Missouri and Niobrara rivers near their confluence and report on certain variations that have been identified in stream discharge, channel dimensions, sediment transport and delta deposition over a twenty-year period. Examples identify the variety of study data available. Although the conclusions of this preview paper are mostly qualitative, the trends that are identified certainly establish the growth rate of the Niobrara delta and its equilibrium sensitivity.

NATURAL STREAM CHARACTERISTICS

Deltaic type conditions of various magnitudes have undoubtedly existed at the confluence of many Missouri River tributary streams for centuries. The concentration of sandbar formations in the flow channels at the junction of alluvial streams is a common geologic occurrence. Early evidence of such deltaic conditions exists in the Lewis and Clark Journals. In particular, they described in September 1804, that the Niobrara River, like the Platte, "throws out into the Missouri River great quantities of sand which form sandbars and shores near its mouth." A comprehensive Missouri River survey by the Corps of Engineers in 1892 provides the first hydrographic evidence as to the extent of such conditions. It is apparent that the heavy sediment loads contributed by these tributaries, plus an abrupt and major change in river slope as they enter the relatively flat, sluggish Missouri, is conducive to the deposition of sediments and formation of deltaic sandbar conditions. However, the extent or degree of such concentrations was naturally controlled by the periodic flushing or redistribution of such deposits further downstream during flood periods. Missouri River floods of sufficient magnitude generally occurred twice each year; in early Spring due to snow melt over the upstream Great Plains and in late June or early July due to mountain snow melt runoff. It is probable that when drought periods curtailed runoff from the upper basin, the growth of deposits at the mouth of the Niobrara or Platte River was greater than normal. But the periodic swing of the hydrologic pendulum toward excessive runoff conditions would effectively flush the Missouri River channel clean and start a new cycle. It was this condition that existed at the mouth of the Niobrara River in 1953 when the effective regulation of the Missouri River flows started. The maximum flood of record on the Missouri River occurred in April 1952 to set the stage for the subsequent delta growth.

The natural state of the Missouri River near its confluence with the Niobrara is best described by its serpentine channel alignment that meanders at random across an alluvial flood plain that is entrenched between steep valley bluffs. The bank-full flow channel averaged about 2,000 feet in width but was generally divided into several channels by large islands or lower elevation sandbars. See Figure 1. Its general shape was rectangular. Flood plain bank heights averaged between 12 and 15 feet in height with a channel capacity of about 150,000 cfs. At normal stages, water surface slopes generally varied from 0.8 to 1.0 feet per mile; channel widths from 700 to 1,500 feet; and mean depths between 4 and 11 feet. The greater widths and shallower depths occurred in the diagonal crossings where the channel meandered between bluff contacts. Channel bed forms were constantly in a state of change due to the shifting movements of ripples, dunes and bars. Although this movement would produce characteristic, seasonal shifts in the stage-discharge relationship, the long-term rating at station locations remained relatively consistent. The natural sediment transport capability of Missouri River flows was both great and variable. Annual sediment load volumes for the Missouri River near Niobrara probably ranged between 80 and 240 million tons with an average near 148 million

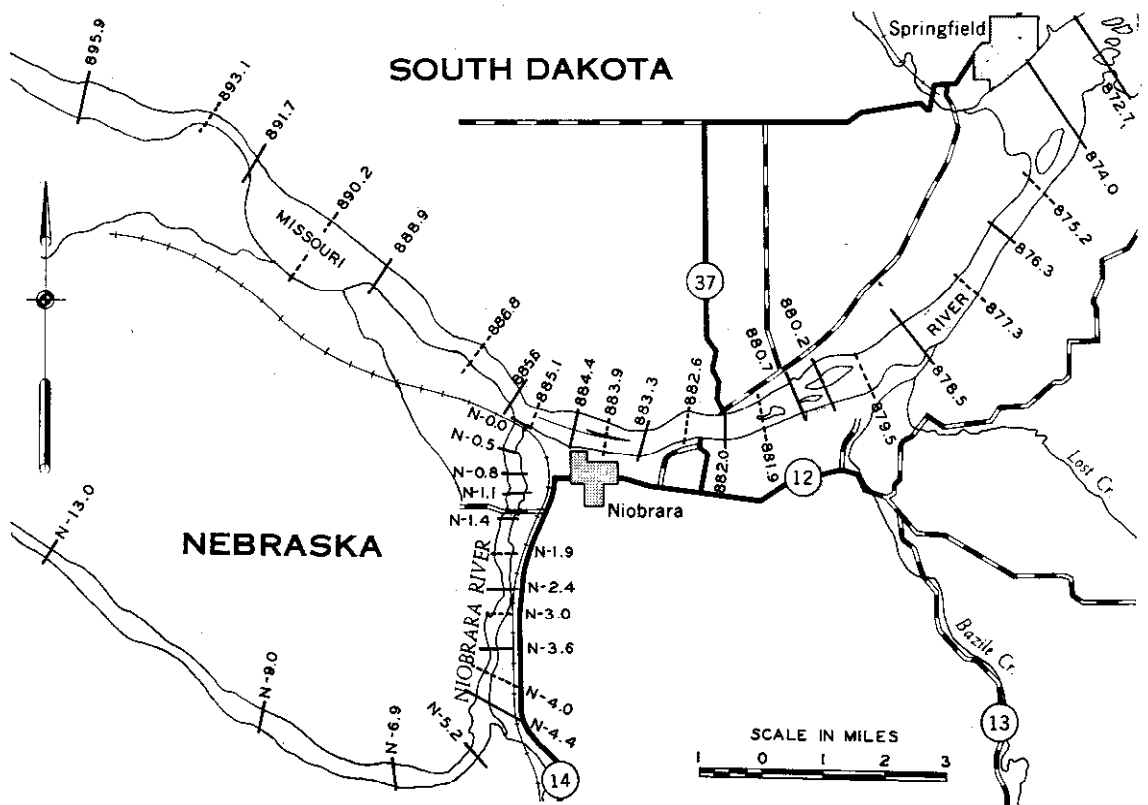


Figure 1. Confluence of Missouri and Niobrara Rivers.

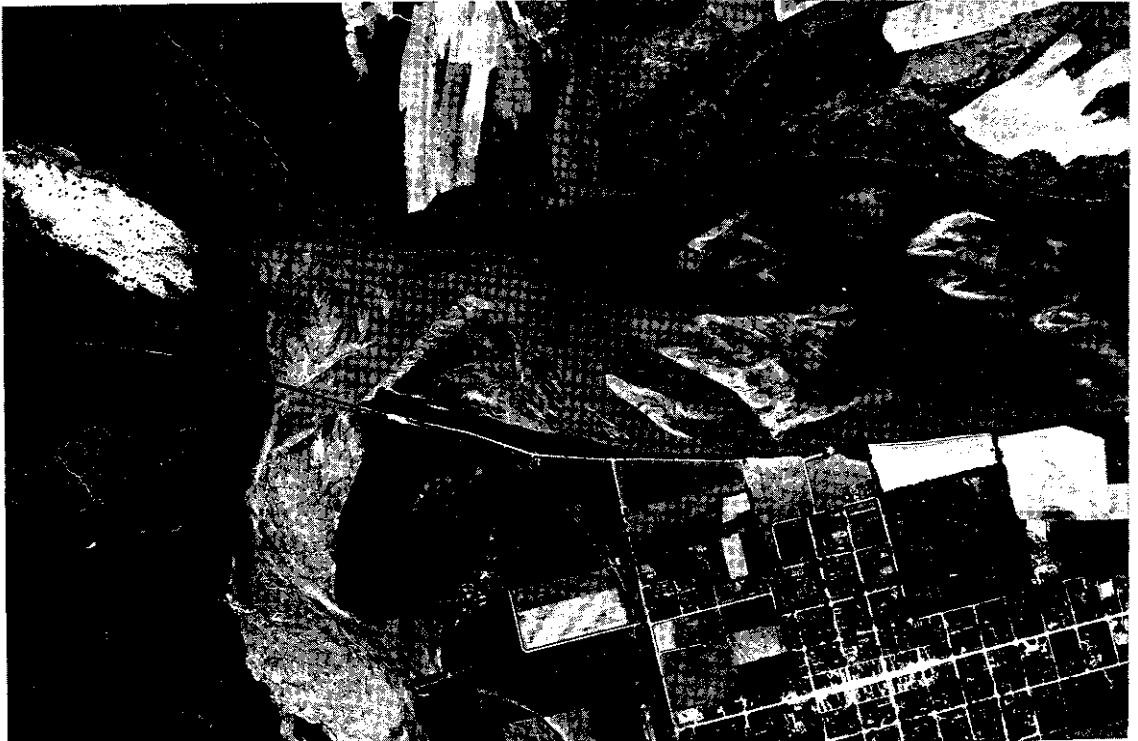


Figure 2. An example of active delta growth - June 1961.

tons. The particle size distribution of the suspended sediments averaged about 30 percent sand, 30 percent silts and 40 percent clays. The bed material sediments generally ranged within the very fine to fine sand sizes. A normal D_{50} size would be 0.20 mm with the percent finer at D_{10} and D_{90} being 0.08 and 0.30 mm, respectively.

The Niobrara River is, in contrast, a shallow braided stream. The total channel width generally varies from 1,000 to 1,500 feet but the flow is divided into numerous separate channels by barren or sparsely vegetated sandbars and higher elevation islands. Note the braided conditions in Figure 2. The heights of the sandbars above low flow stages are usually less than 12 to 18 inches. The height of the channel bank, flood plain and island elevations range between 4 to 6 feet above low flow stages; however, the flood plain contains many remnants of old flow channel chutes which are, at times, subject to inundation at less than bankfull stages. Natural levees, of slightly higher elevation, along both the active and older chutes are also common. The average slope of the lower Niobrara River over long reaches appears to be quite uniform at about 7 feet per mile. But short reach slopes can vary quickly over a wide range as shifts in the flow channel shape or alignment occur.

The sands which comprise the streambed of the Niobrara are, like the Missouri River, quite uniform in size but slightly coarser. Their range from fine to coarse sand makes them easily transportable even under the energy gradient of low flows. This characteristic is, of course, associated with a braided stream and accounts for the lack of any stable stage-discharge relationship over the range of base flows. Year round, this base varies between 1,000 and 2,000 cfs. Spring snowmelt runoff in March, combined with ice jams, accounts for maximum stages and peak flows but the greater volume is sometimes produced during late spring and early summer. Table 1 presents a summary of experienced discharge variations for the Niobrara River near Verdel, Nebraska for the 20-year period 1953-1973. (The time increments shown will remain consistent in later tables to permit a comparative assessment of variations in stream and sediment discharge, channel dimensions and deposition volumes).

TABLE 1
VARIATIONS IN STREAM DISCHARGE

<u>Stream</u>	<u>Location</u>	<u>Period</u> (Water Year)	<u>Total Runoff</u> <u>Volume</u> (AF)	<u>Mean Annual</u> <u>Discharge</u> (CFS)	<u>Maximum Mean</u> <u>Daily Discharge</u> (CFS)
Missouri	Below Fort Randall Dam	1953-1955	49,750,000	22,900	105,000
		1956-1960	65,390,000	18,000	50,500
		1961-1965	61,168,000	16,900	38,400
		1966-1970	95,030,000	26,250	52,000
		1971-1973	69,330,000	31,900	50,500
Niobrara	Verdel	1953-1955	4,084,000	1,710	15,700
		1956-1960	6,168,000	1,730	25,100
		1961-1965	6,436,000	1,780	16,600
		1966-1970	5,407,000	1,490	10,000
		1971-1973	3,345,000	1,540	6,500

The long-term, average-annual sediment load of the Niobrara River is estimated at 2,900,000 tons. This estimate is based upon a limited record of sampling measurements where total suspended sediment load computations provided a relationship with annual runoff volumes and permitted an extrapolation for a 41-year flow record. It correlates very well with the volume of deposited sediments measured by hydrographic surveys in the Missouri River channel and Lewis and Clark Lake. Unlike the relatively constant annual runoff volumes, the annual sediment volumes vary by a factor of 2 or more. The peak flood discharges account for some of this variance but high transport rates during periods when moderate flows produce sustaining scour capability, may be an equal or greater influence. During normal flow periods, the transport rate of sand is near 75-80 percent, silt ranges from 10 to 20 percent and clays usually less than 10 percent. However, during peak flows the wash load will at times dominate with the sand contribution dropping to 25 percent or less. The bed material sizes probably increase with discharge but the average D₅₀ size is about 0.25 mm with the D₁₀ and D₉₀ percent finer being 0.15 and 0.4 mm, respectively. Gravel size particles or larger are very uncommon. Table 2 presents a summary of estimated suspended sediment discharges for the Niobrara River near Verdel, Nebraska for the period 1953-1973.

TABLE 2
ESTIMATED VARIATIONS IN SUSPENDED SEDIMENT DISCHARGE

<u>Stream</u>	<u>Location</u>	<u>Period</u> (Water Yr.)	<u>Total</u> <u>Suspended</u> <u>Sediment</u> <u>Discharge</u> (Tons)	<u>Annual Variations (Tons)</u>		
				<u>Mean</u>	<u>Maximum</u>	<u>Minimum</u>
Missouri	Range 893.1	1953-1955	3,008,000	1,000,000	1,606,000	701,000
		1956-1960	2,390,000	480,000	916,000	219,000
		1961-1965	1,967,000	390,000	507,000	106,000
		1966-1970	7,811,000	1,560,000	2,263,000	1,022,000
		1971-1973	8,979,000	3,000,000	4,015,000	1,570,000
Niobrara	Verdel	1953-1955	8,950,000	2,980,000	3,700,000	2,000,000
		1956-1960	12,560,000	2,510,000	3,350,000	1,610,000
		1961-1965	14,920,000	2,980,000	5,500,000	1,850,000
		1966-1970	10,250,000	2,050,000	2,320,000	1,800,000
		1971-1973	6,610,000	2,200,000	2,640,000	1,900,000

NOTE: The values shown in this table should be considered preliminary and subject to change. Some were developed by sediment rating curves in order to provide an approximation for this paper.

STREAMFLOW REGULATION

The first step toward regulation of the Missouri River started with construction and closure of Fort Peck Dam in Montana in 1937. Although it provided regulation of the extreme upper basin, its influence on the flow duration trends of the Missouri near Niobrara were insignificant. The next two quick steps, with closure of Fort Randall in 1952 and Garrison in 1953, produced a decisive and positive change in the flow regime to complete regulation. Later, closure of Oahe in 1958 and Big Bend in 1963 completed a regulation capability for storage of over

76,000,000 million acre feet of water. With the closure of Fort Randall and Garrison, an immediate need developed for the initial filling of almost 22 million acre feet of operational storage from an anticipated mean annual supply of about 24 million acre feet. Under normal hydrologic conditions this filling aspect would induce some minor downstream flow restrictions but an orderly accumulation of operational storage in the system was probable within a few years. But coincident with this need, in 1954 there began a seven-year drought over the upper basin which effectively extended the filling period to 1968. Abnormally low inflow rates occurred in 1954, 1955, 1958 and 1961.

Releases from the reservoir system, via Fort Randall, were held to a minimum during these years. Peak discharges ranged up to 45,000 cfs during periods of maximum power demands but night-time releases were frequently zero. During the March to November open water season, the mean daily flow of the Missouri River passing the confluence of the

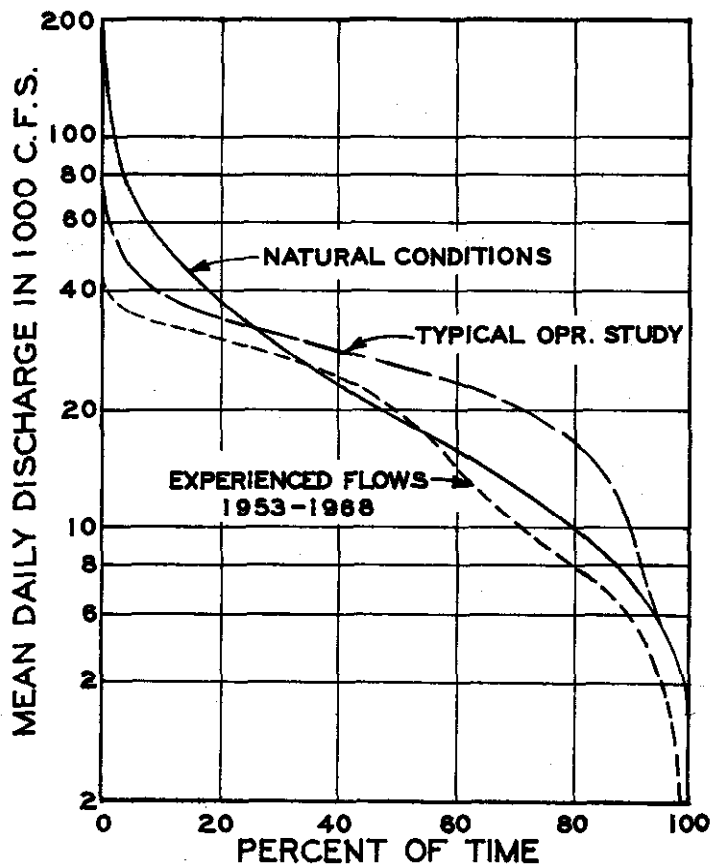


Figure 3. Missouri River flow duration curves.

Niobrara River seldom exceeded 35,000 cfs. Winter flows were consistently less than 10,000 cfs. For illustrative purposes, the flow durations experienced during the period from 1953-1968 are compared in Figure 3 with natural conditions prior to 1953 and a synthetic operational plan which assumed repetition of an experienced long-term flow record. It was not until 1969, when it became necessary to evacuate

flood control storage in the upstream system, that releases exceeded the 50,000 cfs level, or about an expected 5-year flow event. The magnitude of these discharges, plus the need to release similar flows again in 1971 and 1972, effectively demonstrated the influence of the Niobrara delta growth on the main stem channel capacity.

DELTA GROWTH

The regulated flow regime of the upstream reservoir system and the start of the Niobrara delta growth began one year after the historic 1952 flood of record flushed the Missouri River channel clean at the confluence. For the next 17 years, until 1969, the mean daily flow of the Missouri passing Niobrara was usually 30,000 cfs or less. However, during the first decade of this period, above normal flows from the Niobrara basin, including the flood of record, continued to add sediments to the delta at an average growth rate of about 450 acre feet per year. This produced a deltaic blockage of the Missouri River channel for several miles downstream from the Missouri-Niobrara confluence. The Figure 2 photo depicts typical conditions during an active delta growth period. The resultant effect of the shallow, weir-like delta produced (1) a progressive upward shift of at least five feet in the Missouri River rating curve at Niobrara; (2) a reduction in the channel capacity from about 120,000 cfs to 60,000 cfs at this same point; (3) significant backwater influences at upstream points along both the Missouri and Niobrara Rivers. Note Figures 4 and 5. The first consequence of this delta became apparent after the mid-1960's when adverse groundwater levels were experienced within the town of Niobrara and at the nearby Niobrara State Park. The next impact occurred during the late summer and through the fall of 1969 when it became necessary, for the first time, to evacuate water from upstream reservoir flood control storage zones, prior to winter, in preparation for the next flood seasons runoff. Sustained releases in the 50,000 cfs range produced near bank-full stages and the inundation of some 1,750 acres of lower elevation flood plain lands. Similar inundations occurred again in 1971 and 1972 for the same reasons.

The most sensitive indicator for assessing the delta growth has been shifts in the Missouri River stage-discharge relationship at gage 884.2. This gage is strategically located just downstream of the Missouri-Niobrara confluence near the apex of any delta growth. Stage trends for flows of 30,000 cfs at this gage are presented in Figure 4. Note the continuous annual rise during the early growth period, a leveling off between 1963 and 1968 as some equilibrium developed and the distinct sag after 1969 which reflects high flow scour of delta deposits. Although not distinctly apparent in Figure 4 because of their transient nature on an annual basis, stage records can identify short-term shifts which seem to correlate with high sediment load contributions from the Niobrara basin.

The changes in channel dimensions which account for such stage-discharge trends have been documented by periodic surveys of channel ranges. These cross-sectional measurements provide the basic means for assessing variations in hydraulic parameters and sediment deposition volumes. Tables 3 and 4 present comparisons of such measurements and

are used to illustrate, by the intervals identified in Tables 1 and 2, the variations occurring during the delta growth. A comprehensive analysis of these variations will be possible when similar measurement records at other channel ranges are compiled.

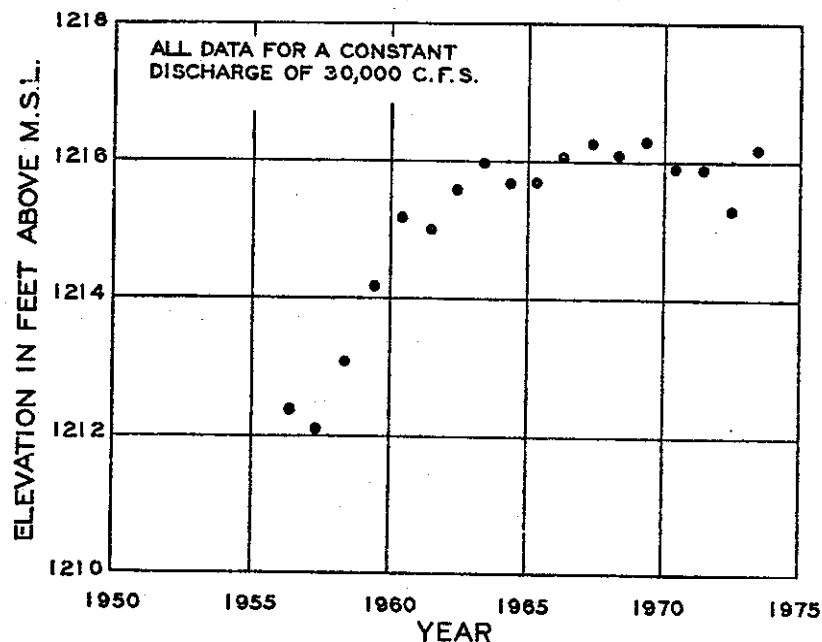


Figure 4. Missouri River stage trends at gage 884.2.

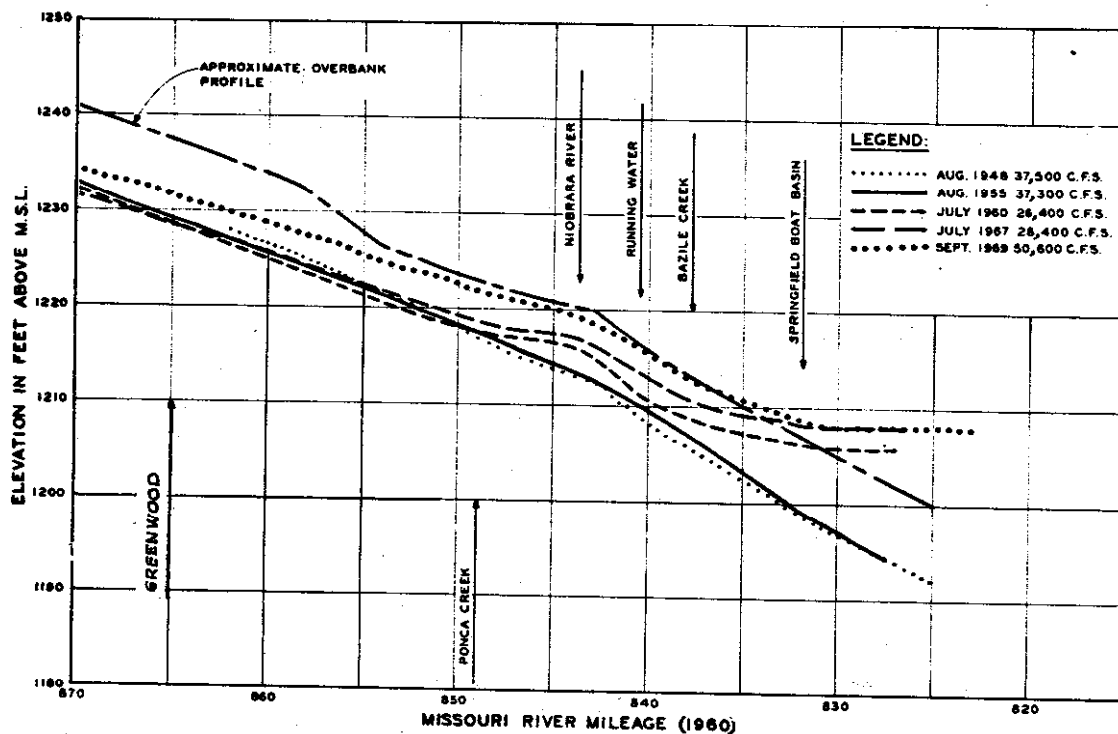


Figure 5. Missouri River water surface profiles.

TABLE 3

VARIATIONS IN CHANNEL DIMENSIONS

<u>Range</u>	<u>Date</u>	Bank Top Elev. (MSL)	<u>Cross Section</u>			<u>Width Depth Ratio</u>
			<u>Area</u> (Sq.Ft.)	<u>Width</u> (Ft.)	<u>Depth</u> (Ft.)	
M-885.6	Oct 1955	1220	29,048	2,740	10.60	258
	Jul 1960		26,658	2,784	9.58	291
	Aug 1965		25,155	2,788	9.02	309
	May 1970		25,245	2,818	8.96	314
	Sep 1973		24,871	2,919	8.52	343
M-884.4	May 1955	1220	36,872	2,952	12.46	237
	Jul 1960		30,148	3,103	9.72	319
	Aug 1965		24,592	3,208	7.67	418
	Aug 1970		23,513	3,236	7.27	445
	Sep 1973		21,366	3,227	6.62	487
M-882.0	May 1955	1215	23,902	2,349	10.18	231
	Jul 1960		21,174	2,355	8.99	262
	Aug 1965		14,410	2,350	5.12	385
	Aug 1970		14,725	2,363	6.23	379
	Sep 1973		14,539	2,353	6.18	381
N-0.0	Sep 1956	1224	11,684	1,272	9.18	138
	Jul 1960		10,061	1,276	7.88	162
	Aug 1965		9,015	1,277	7.06	181
	Aug 1970		7,822	1,278	6.12	209
N-0.8	Sep 1956	1225	17,816	2,303	7.74	298
	Jul 1960		15,997	2,319	6.90	336
	Aug 1965		12,988	2,347	5.53	424
	Aug 1970		11,049	2,355	4.69	502

TABLE 4

VARIATIONS IN CHANNEL DEPOSITION

<u>Stream Reach</u>	<u>Period</u>	<u>Deposition Volume (AF)</u>	<u>Deposition Rate (AF/YR)</u>	<u>Bed Material Size</u>
				<u>D₅₀ (mm)</u>
M-885.6 to M-884.4	1955-1960	664	133	0.26
	1960-1965	513	103	0.27
	1965-1970	72	14	0.20
	1970-1973	183	61	0.26
M-884.4 to M-882.0	1955-1960	1375	275	0.31
	1960-1965	1792	358	0.31
	1965-1970	110	22	0.30
	1970-1973	340	113	0.32
N-0.0 to N-0.8	1955-1960	167	42	0.27
	1960-1965	196	39	0.25
	1965-1970	154	31	0.29
	1970-1973	90*	30*	0.23

* Estimate

Assessment of the data presented in Tables 1 through 4 provides several interesting insights related to the rate growth, attainment of a stability level and the sensitivity of this equilibrium to change. Reference is made to a composite summary on Table 5 for the following interim evaluation on these items. It is intended that final study

TABLE 5
RELATIONSHIP OF STREAMFLOW AND SEDIMENT DISCHARGES
TO DELTA GROWTH

<u>Period</u>	<u>Average Annual Missouri River Discharge (CFS)</u>	<u>Total Sediment Discharge* (AF)</u>	<u>Delta Growth</u>		<u>Percent Sediment Inflow Deposited (%)</u>
			<u>Volume Change (AF)</u>	<u>Deposition Rate (AF/YR)</u>	
1953-1955	24,600	6,780	800**	400	11.8
1955-1960	19,700	8,475	2210	442	26.0
1960-1965	18,700	9,575	2500	500	26.1
1965-1970	27,700	10,240	336	67	3.3
1970-1973	33,400	8,840	613	204	6.9
TOTAL	24,100	43,910	6459	323	14.7

* Assumed 5 percent bed load factor and average deposited unit weight of 85 PCF.

** Estimated

assessments, to be presented in a comprehensive manner in a future paper, will amplify this discussion and produce quantitative conclusions.

(1) The combination of greater than normal sediment contributions from the Niobrara drainage and less than normal releases from the Missouri reservoir system strongly influenced the rate of early delta growth. During the periods when the Niobrara produced from 85 to 90 percent of the total sediment inflow, the growth rate of the delta remained relatively constant at about 450 acre feet per year. However, this rate accounts for only 25 percent of the available sediment supply. These values probably establish near maximum rates for expected delta growth and its trap efficiency.

(2) The relative stable channel widths observed on both the Missouri and Niobrara, while deposition depleted the channel capacity, was not anticipated. A much greater degree of change was expected. A cursory review of additional data indicates that the average channel-forming velocities immediately downstream of the confluence were significantly higher.

This could account for the coarsening of the D₅₀ bed material sizes noted at range 884.4 in Table 4. The higher width-depth ratios indicate an expected increase in bed material transport but total load computations are not sufficiently complete to isolate the significance of this bed coarsening.

(3) The delta equilibrium appears to be most sensitive to variances in Missouri River discharges. Stage-discharge trends during the higher flow periods after 1969 indicate the possibility of delta scour or at least more efficient channel conditions. Lower deposition rates also tend to confirm this even though the upstream Missouri River channel is transporting significantly greater sediment loads. Defining the hydraulic combinations that trigger a shift in the bed material transport capability from deposition to scour will provide the much needed key to predicting the degree of future delta growth.

CONCLUSIONS

The dynamic balance in sediment transport that occurred under a natural regime at the confluence of the Missouri and Niobrara Rivers was dependent on the frequency, magnitude and duration of main stem flows. Regulation of these flows after 1952 by a system of upstream reservoirs interrupted this equilibrium and permitted the tributary sediment contributions to accumulate in the Missouri River channel. The growth of this Niobrara delta was accelerated due to two concurrent factors; first, below normal runoff from the upper Missouri River basin during the "filling" of operational reservoir storage and second, above normal sediment yield from the Niobrara drainage. The resultant effect has been a significant reduction in channel capacity with attendant consequences of operational restrictions, frequent low land inundation and higher groundwater levels.

Field measurements of sediment and hydraulic parameters have documented the growth of this delta over a 20-year period. Study assessments of this data are not complete but preliminary results indicate (1) the probable maximum growth rate to be expected would be 450 acre feet per year with a trap efficiency factor of 25 percent; (2) surprisingly, relatively stable channel widths have prevailed while the channel capacity has been reduced by at least 50 percent; and (3) delta scour has probably occurred due to high releases from the reservoir system but the accompanying variances in bed material transport have not, as yet, been quantified. It is planned that a comprehensive report of study findings, including the complete documentation of field data, will appear in a future addition to the Missouri River Division Sediment Series publications.

DISTRIBUTION OF RESERVOIR SEDIMENT--IOWA AND
MISSOURI DEEP LOESS HILLS^{1/}

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USDA ARS, Columbia, Missouri.

ABSTRACT

The designers of small reservoirs need to estimate the sediment distribution in a proposed reservoir to best determine (1) the capacity of the structure needed for sediment deposition below the principal spillway and (2) the elevation of the principal spillway. These two items are very crucial to the proper performance of the reservoir and its useful life.

The senior author reported on the sediment distribution in 23 small reservoirs in the Iowa and Missouri Deep Loess Hills Major Land Resource Area in 1961. He found that the distribution of sediment could be estimated quite well; the coefficient of determination for estimating the elevation of deposited sediment at the spillway was 0.91. The most important variables were storage depletion, reservoir topography, remaining capacity, and sediment volume-weight.

Since 1961, five of the 23 small reservoirs were resurveyed, and information on the actual distribution of sediment in other small reservoirs has become available. These new data have been analyzed separately and also in conjunction with the findings of the first study.

The new study shows that storage depletion, reservoir topography, length, and original depth are the most important variables in predicting depth of sediment at the dam. The last two variables differ from the first study, primarily because the reservoirs surveyed since 1961 were selected to include the greatest diversity of reservoir variables in the resource area. The authors concluded that sediment distribution can be predicted for this resource area.

INTRODUCTION

The distribution of deposited sediment in small reservoirs is becoming increasingly important because of the increased cost of construction and more costly developments near the water's edge. Furthermore, good reservoir sites are a valuable natural resource, and the number is limited; we must optimize the design and utility of each reservoir.

^{1/} Contribution from the Watershed Research Unit, North Central Region, USDA ARS, in cooperation with the Missouri Agricultural Experiment Station, Columbia, Missouri, and the Iowa Agriculture and Home Economics Experiment Station, Ames, Iowa.

The distribution of sediment that will be deposited in a proposed reservoir has long been a problem in designing such structures. Estimates of the amount and elevation of sediment deposits are used in determining the elevation of the principal spillway and capacities of the various storage pools. Furthermore, information on sediment distribution is needed in connection with planned recreation sites, homes, and other installations in or near the water. Costly structures should be protected from uncontrolled sediment deposits.

The Bureau of Reclamation, U.S. Department of the Interior, has made several studies of sediment distribution in large reservoirs (Sedimentation Section, 1954; Borland, 1958; Lara, 1962), and one study was made by the Army Corps of Engineers (Hobbs, 1967). However, very little research pertains to sediment distribution in small reservoirs (Heinemann, 1961; Stall, 1964).

This report discusses recent research in sediment distribution utilizing the data from 23 small reservoirs included in the first study (Heinemann, 1961) and more recent sedimentation data from 18 additional reservoirs surveyed by C. W. Farnham in connection with special work at Iowa State University for the Soil Conservation Service. Other studies of the data from these latter surveys were reported earlier (Beer, 1966; Farnham, 1966).

Reported here are the multiple regression analyses of the reservoir characteristics and measurements of sediment deposited in these reservoirs. These analyses were successful since they permit a designer to predict how sediment will be distributed in a proposed reservoir during any period of time after construction. Many combinations of important reservoir and structural parameters that affect sediment deposition were included in this effort.

Some of the parameters not included in the data table herein and those not appearing statistically significant in this particular Land Resource Area may be highly significant in other areas. The findings of this study apply only to small reservoirs in the Iowa and Missouri Deep Loess Hills (Austin, 1965). Furthermore, these findings should not be used to attempt to predict reservoir sediment distribution when the proposed reservoir will have values for parameters highly different from those found significant in this study.

STUDY LOCATION

All reservoirs included in this study, except two, were located in the Iowa and Missouri Deep Loess Hills Major Land Resource Area 107 (Austin, 1965) of western Iowa (primarily), the extreme eastern edge of Nebraska, and a narrow band along the Missouri River to central Missouri. This area (Figure 1) comprises about 5 million hectares (12 million acres) and is covered with a blanket of loess ranging in thickness from one meter to over 60 meters (200 ft.). The loess is characterized by relatively homogeneous particle-sized distribution of 70 percent silt, 25 percent clay, and 5 percent sand. It is underlain chiefly by Kansan till and, in some small areas, by Iowan till or bedrock.

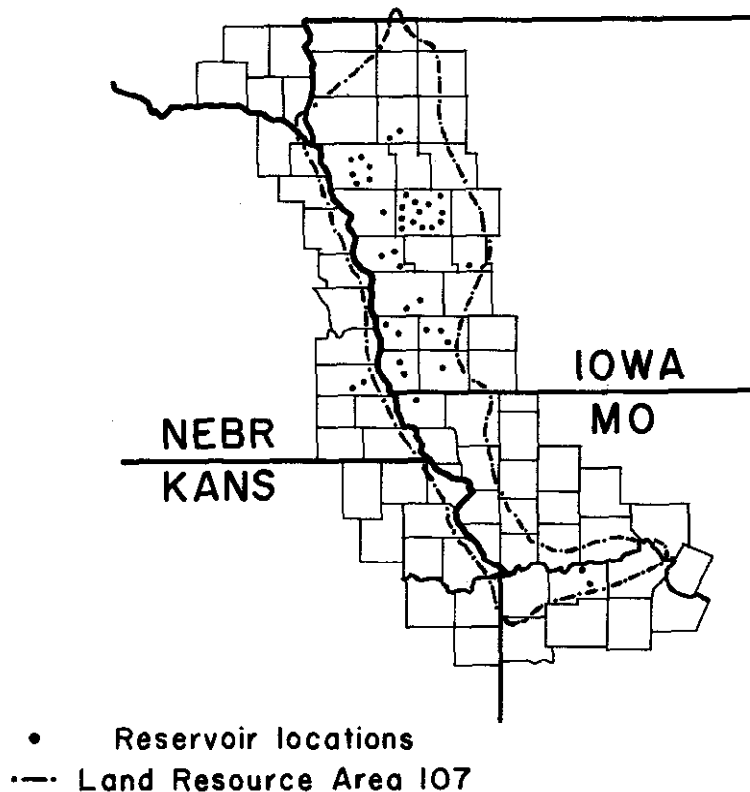


Figure 1.--Location of Survey Reservoirs.

The topography varies from many rugged bluffs along the Missouri River to almost level land in the northern portion. Although there are several different upland soil types, differing primarily in thickness of the loess deposit, degree of slope and erosion, and vegetation under which they were formed (timber or prairie), all are very similar with respect to the erosion hazard when cultivated.

This "area produces more sediment per square mile from small watersheds than any other area of equal size in the country" (Gottschalk, 1950). Sheet and rill erosion are severe in this area with its frequent intense rains on steeply sloping topography, particularly when clean-tilled or small grain crops are farmed up and down the slope. In addition, the rolling to hilly loess-mantled plain is intricately dissected by deep, upland gullies that have developed in the last 65 years because of intense land use, hard rainstorms, and changes in downstream channel gradients. Many gullies advance headward at a rate of about 30 meters (100 ft.) annually and contribute high concentrations of sediment (Gottschalk, 1950). Because the deep loess is inherently very fertile throughout the soil profile, it is difficult to induce farm operators to initiate conservation and management practices to reduce erosion from their land.

Average annual precipitation varies from approximately 64 cm (25 in.) in the north to 91 cm (36 in.) in the southeast. Mean annual runoff, determined from U.S. Geological Survey gaging records (Gottschalk, 1950) of

10 years or longer, for drainage areas from 520 to 7,250 km² (200 to 2,800 square miles), varied from about 4 cm (1.5 in.) in the north to over 15 cm (6 in.) in the southeast.

This Land Resource Area is characterized by about 75 percent cropland, less than 15 percent pasture and grassland, and little forest and woodland.

BASIC DATA, DATA PROCESSING AND COMPILATION, AND APPROACH

Reservoir sedimentation data were obtained from two sources. Field data on the 23 small reservoirs included in the Heinemann (1961) study were obtained from the Soil Conservation Service, USDA; Iowa State University; and personal measurements. On some of the 23 reservoirs, data were available from more than one sedimentation survey and, when adequate, it was included in the study. This resulted in having 34 sets of sedimentation data for the 23 reservoirs. Reservoir capacities were calculated by the range and prismoidal methods, and differences were adjusted to the range method (Heinemann, 1965). Additional details on data processing were given by Heinemann (1961).

A combination of the contour and range methods was used in making the additional 24 reservoir sedimentation surveys in Farnham's study. Remaining capacities and sediment volumes were determined by using the stage-area curve method (Rausch, 1968). In all, data were used from 58 surveys of 41 reservoirs.

The reservoir sedimentation survey methods, as well as the reservoir capacity determinations, were different for the two sets of data. It would have been better if all had been surveyed and computed using the procedures of the latter set of reservoir data. However, the differences in capacities, as determined by the two methods, should not exceed about 7 percent.

The sedimentation information curves approach was followed in this study. This involves converting reservoir depths, capacities, sediment volumes, and storage depletion data to percentages, so that information on one reservoir is easily compared with other reservoirs. Dimensionless capacity-sediment-depth curves provide a great deal of sedimentation history and sediment distribution information on each reservoir. Reservoir #35-4 is used as an example (Figure 2).

The original stage-capacity curve (CO/CT) shows the percentage of total original capacity below a given elevation (D/DT). For example, in Figure 2, with CO/CT = 20 percent (6,975 m³) of the original capacity was located in the bottom 50 percent (2.3 m) of the reservoir. Of course, the more horizontal a segment of this curve is, the more capacity there is available within the given segment of depth.

The capacity-replaced-by-sediment curve (CREPS) shows the percentage of storage depletion below a given elevation. In this reservoir, 100 percent of the original capacity in the bottom 41 percent of the reservoir depth (Point Y) was filled with sediment as of the survey date.

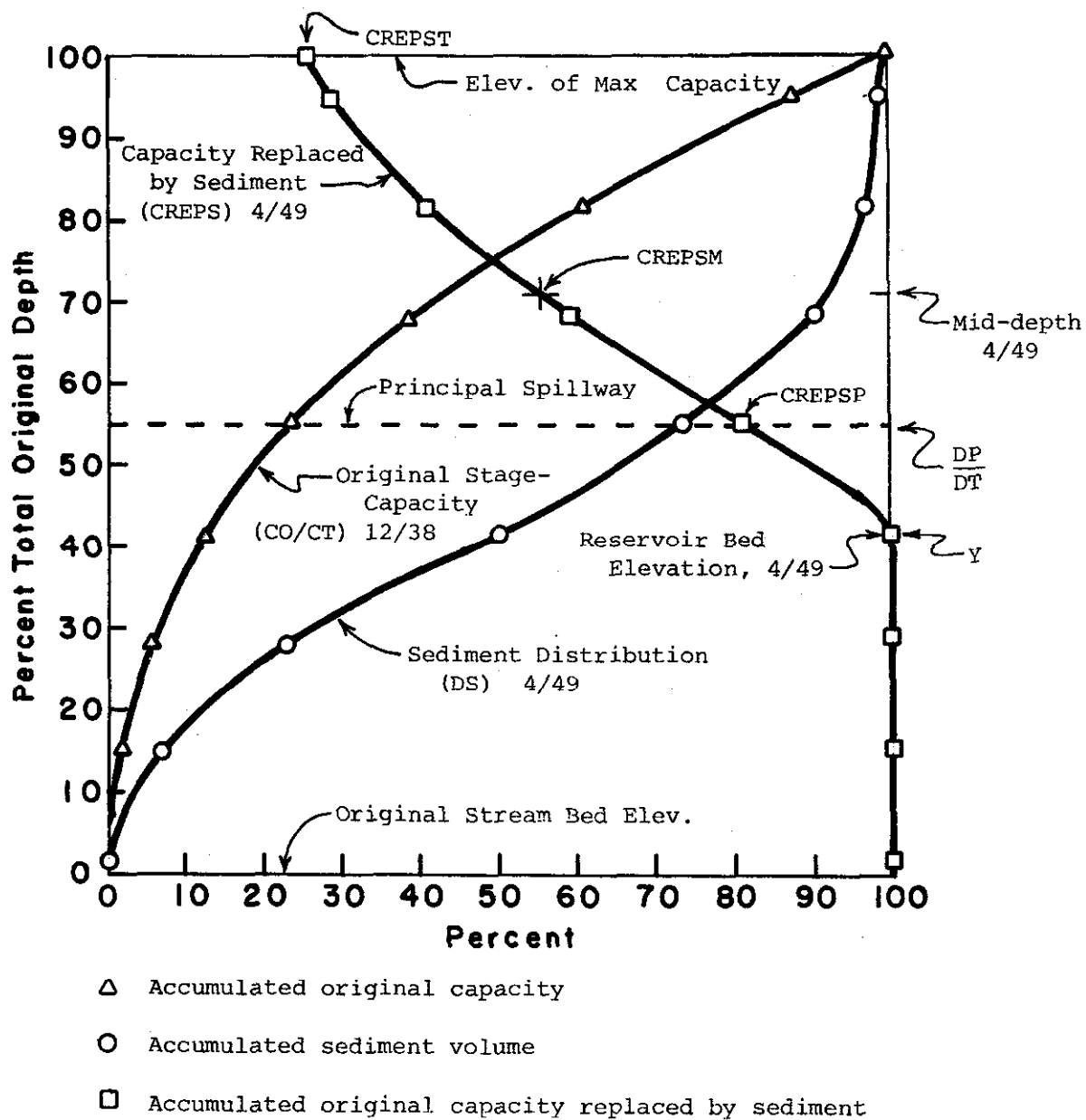


Figure 2.--Sedimentation information curves for Evers Lower Reservoir, 35-4. Total original capacity 34888m^3 (28.3 A.F.); total original depth 4.6m (15.0 ft).

This means that the bottom of the reservoir was (in April, 1949) 41 percent x 4.6 m or over 1.8 m higher than it was originally. The total capacity replaced by sediment to the elevation of maximum capacity (CREPST) is about 26 percent.

The sediment distribution curve (DS) shows the vertical location of deposited sediment in the reservoir as of the date of the sedimentation survey--April 1949 (Figure 2). This shows, for example, that about 73 percent of the sediment is located below the principal spillway. The flatter segments indicate the elevations between which exists the highest percentages of the deposited sediment. Only 10 percent of the sediment is located in the upper 32.5 percent of the reservoir depth.

If the original stage-capacity and capacity-replaced-by-sediment data are available, the sediment distribution (DS) can be computed. If the original stage-capacity and sediment distribution curves plus the total sediment volume are available, the capacity replaced by sediment can be computed. To determine a point on one curve at a given elevation when information is available on the two other curves, use the equation

$$DS = \frac{CO}{CT} \times \frac{CREPS}{CREPST} \quad (1)$$

Pertinent data for reservoirs studied are tabulated in Table 1.

ANALYSIS AND RESULTS

The analysis of sediment distribution in these reservoirs listed in Table 1 was divided into three interrelated segments; (1) the percentage of total depth filled with sediment after an accumulation of a given amount of sediment; (2) the percentage of the remaining capacity below the principal spillway; and (3) the percentage of capacity replaced by sediment at an elevation mid-depth between the bottom and emergency spillway.

The percentage of depth filled with sediment (Y) was found by regression analysis to be dependent on (1) the percentage of total capacity replaced by sediment (CREPST), (2) the length of reservoir in meters (L), (3) the original total depth in meters (OD), and (4) the slope ("N" value) of the plottings on log-log paper of original depth versus capacity (CO). Original "N" may be computed:

$$NO = \frac{\log D/DT}{\log CO/CT} \quad (2)$$

where CO is the original capacity to a selected depth (D) in the reservoir, and CT is the total capacity in the reservoir at maximum depth DT. Other variables that were tested are: slope of upstream channel and reservoir bottom, capacity of reservoir, capacity/watershed ratio, volume-weight of sediment, length/capacity ratio, capacity/watershed area x OD ratio, and capacity/watershed area x length ratio. The best prediction equation is

$$Y = 39.5 + .919 \text{ CREPST} - .023 L + 1.41 \text{ OD} - 55.8 \text{ NO} \quad (3)$$

TABLE 1.--RESERVOIR SEDIMENTATION DATA

Data Summary Sheet	Name	Date of Survey (Mo./Yr.)	Total Drainage Area (ha)	Sed. Vol. Wt. (G/cm ³)	---RESERVOIR STORAGE---			Reser. Length (m)	Orig. "N"	Total Depth (m)	Prin. Spwy. Depth (m)
					Total (m ³)	Below Prin. Spwy. (m ³)	Total Depletion (m ³ & %)				
1/	Anthon	5/54 6/64	46.10	1.19	50,765 31,173	9,222 0	19,592 38.6	175	0.41	6.5 2.8	3.5 0.0
35-1	Baak	9/44 4/49	41.18	0.88	27,716 20,753	8,531 2,815	6,963 25.1	213	.38	6.0 3.6	3.7 1.3
35-2	Brown	6/41 5/49	25.90	1.02	28,309 18,667	14,679 5,691	9,642 34.1	201	.49	6.0 2.8	4.5 1.3
31-2	Chinqueet	6/38 5/49	42.99	0.80	20,333 10,519	11,679 3,198	9,814 48.3	206	.47	5.2 2.3	4.2 1.3
	Davis	10/57 5/64	69.93	0.97	61,136 53,803	10,580 3,432	7,333 12.0	244	.53	6.3 4.3	2.8 0.8
35-3	Esbeck	5/40 5/49	53.87	0.90	24,531 13,815	15,556 5,568	10,716 43.7	116	.49	4.8 2.2	4.0 1.4
35-4	Evers (Lower)	12/38 4/49	48.43	1.10	34,889 25,877	8,185 1,556	9,012 25.8	267	.41	4.6 2.6	2.5 0.5
35-5	Evers (Upper)	3/39 4/49	11.66	1.15	4,025 1,469	2,346 272	2,555 63.5	49	.46	4.5 1.2	3.8 0.5
35-6	Fienhold	5/45 4/49	110.85	1.01	33,704 21,543	15,593 4,296	12,161 36.1	428	.47	6.6 3.2	4.7 1.3
31-3	Fuelling	7/39 5/49	271.95	1.01	84,654 41,358	39,259 2,383	43,296 51.1	994	.37	6.4 3.1	4.5 1.2
35-7A	Gadd	12/40 5/49 6/52	20.98	1.01 0.92	17,877 15,827 14,790	9,173 7,247 6,481	2,050 11.5 3,087 17.3	168	.49	4.7 3.3 3.1	3.4 2.0 1.8
	Glenwood	7/51 7/64	128.21	1.00	94,272 77,568	41,642 24,889	16,704 17.7	161	.39	9.3 6.4	7.1 4.2
35-8	Gosler	5/40 3/49	23.05	1.11	16,901 14,568	12,568 10,235	2,333 13.8	143	.41	5.1 3.5	4.6 3.0
	Harmony #5	10/57 6/64	143.23	0.83	99,457 81,679	38,963 24,346	17,778 17.9	183	.58	6.7 3.1	5.2 1.6
31-39	Higginsville Uppersite	9/24 7/64	706.56	0.90	556,124 296,396	2/----- -----	259,728 46.7	708	.29	7.0 3.0	--- ---
35-10	Hollrah	8/44 3/49	56.20	0.92	46,333 38,445	23,432 15,753	7,888 17.0	357	.42	7.7 5.5	6.1 3.9
31-40	Huscher	10/50 8/64	86.51	0.94	40,086 31,716	25,049 19,235	8,370 20.9	444	.47	6.3 7.5	5.3 4.6
	Isaacson	10/58 7/64	31.86	0.88	11,605 10,519	6,136 5,185	1,086 9.4	380	.48	4.5 3.9	3.5 2.9
35-13	Jensen-O' Neil	11/36 11/48	51.54	0.88	50,840 38,494	29,235 20,728	12,346 24.3	213	.40	6.9 3.9	5.9 2.9
35-12A	LaFrontz	5/42 4/49 7/53 5/64	39.63	0.90 0.99 1.00	18,864 12,815 11,691 10,877	15,667 9,704 8,556 7,728	6,049 32.1 7,173 38.0 7,987 42.3	128	.44	5.9 3.9 2.7 2.5	5.5 3.5 2.3 2.1
35-15A	Lage, A.	7/41 4/49 6/52	47.66	0.86 0.85	13,864 10,568 8,815	7,037 4,086 2,753	3,296 23.8 5,049 36.4	262	.42	4.1 2.4 1.9	3.2 1.5 1.0

1/ Data summary sheet number assigned by Sedimentation Section of Water Resources Council for river basin and reservoir survey reported. Some summaries are not available.

2/ Total storage is at original principal spillway.

Conversion factors: Acre-feet = m³ x 0.00081
Acre - Hectares x 2.471
Feet = m x 0.3048
lb./ft³ = g/cc x 62.43

TABLE 1.--RESERVOIR SEDIMENTATION DATA (CONT.)

-----RESERVOIR STORAGE-----												
Data Summary Sheet	Name	Date of Survey (Mo./Yr.)	Total Drainage Area (ha)	Sed. Vol. Wt. (G/cm ³)	Total Prin. Spwy. (m ³)	Total Depletion (m ³ & %)	Reser. Length (m)	Orig. "H"	Total Depth (m)	Prin. Spwy. Depth (m)		
35-16A	Lage, H.	7/41	9.84		14,901	10,173		128	.62	4.7	3.8	
		4/49		0.88	13,346	8,247	1,555 10.4			4.2	3.3	
		6/52		0.79	11,062	6,358	3,839 25.8			3.8	2.9	
	Lake's Nursery	6/56	105.67		46,062	16,025		312	.44	5.9	4.0	
		7/64		1.03	36,889	7,926	9,173 19.9			3.6	1.7	
	Little Beaver	10/56	174.31		133,062	27,679		396	.50	8.0	4.3	
		5/64		0.89	112,062	7,123	21,000 15.8			4.5	0.8	
	Lum Hollow	9/58	198.66		165,556	63,309		413	.31	13.8	10.7	
		5/64		0.95	141,235	36,161	24,321 14.7			6.1	3.0	
	36-6	Masters	11/49	39.63		47,383	12,753		230	.51	7.2	4.1
			10/57		1.08	40,494	6,889	6,889 14.5			4.8	1.7
			5/64		1.24	39,210	4,938	8,173 17.2			4.5	1.4
35-17A	Mattson	7/44	25.90		20,296	4,840		122	.48	3.9	2.1	
		4/49		1.11	15,506	2,358	4,790 23.6			2.9	1.1	
		7/53		1.40	11,000	0	9,296 45.8			1.6	0.0	
35-18A	Meyer	11/54	76.92		54,074	13,630		396	.43	6.0	3.6	
		4/49		0.93	46,086	7,901	7,988 14.8			4.4	2.0	
		6/52		1.02	42,753	4,667	11,321 20.9			3.5	1.1	
		5/64		1.00	37,148	667	16,926 31.3			2.7	0.3	
35-19A	Miller #1	11/41	57.76		60,284	11,728		236	.51	4.1	1.8	
		5/49		1.04	44,135	1,667	16,149 26.8			2.7	0.4	
		6/52		1.11	39,630	148	20,654 34.3			2.4	0.1	
35-20A	Miller #5	11/41	63.97		65,667	20,111		200	.42	6.4	3.9	
		5/49		1.11	51,778	9,198	13,889 21.2			3.7	1.2	
		6/52		1.17	50,691	8,272	14,976 22.8			3.5	1.0	
		6/64		1.21	48,420	3,654	17,247 26.3			3.5	1.0	
	Morgan	10/57	67.08		41,803	11,840		270	.44	5.2	3.0	
		6/64		0.92	36,309	6,259	5,494 13.1			3.7	1.5	
36-12A	Mortenson	8/49	356.65		282,988	78,778		831	.35	11.7	7.8	
		6/64		0.87	270,605	66,432	12,383 4.4			9.7	5.7	
	Mule Creek "H"	11/54	201.51		160,395	28,407		549	.34	6.2	3.3	
7/64	0.87	144,148		10,488	16,197 10.1	3.9	1.0					
35-21A	Mundt	10/44	87.03		56,753	15,975		262	.36	6.0	4.2	
		4/49		0.86	40,840	9,938	15,913 28.0			3.5	1.7	
		6/52		0.85	36,074	5,914	20,679 36.4			2.7	0.9	
		5/64		0.99	31,938	8,049	24,815 43.7			2.1	0.3	
35-22A	North	11/39	63.46		60,049	45,185		302	.41	6.7	6.0	
		3/49		0.84	49,432	35,000	10,617 17.7			4.8	4.0	
		7/53		0.91	46,185	32,012	13,864 23.1			4.2	3.5	
	Ogden	9/54	84.18		50,173	22,062		415	.48	6.4	4.7	
		6/64		0.76	47,543	19,630	2,630 5.2			5.8	4.1	
35-14	Peterson	10/36	19.43		4,778	2,543		84	.47	3.4	2.5	
		11/48		0.98	2,148	198	2,630 55.0			1.1	0.2	
	Skalla	10/55	103.08		43,124	19,728		203	.49	6.2	4.7	
		6/64		1.30	21,346	346	21,778 50.5			2.1	0.6	
36-2A	Stiles	12/40	153.85		96,296	56,790		445	.43	4.5	3.8	
		9/50		0.76	85,309	54,938	10,987 11.4			4.0	3.3	
		2/53		0.93	83,580	53,457	12,716 13.2			3.9	3.2	
		6/64		0.93	78,593	48,593	17,703 18.4			3.5	2.8	
36-11	Theobald, "D"	8/48	25.38		34,926	23,210		201	.46	5.2	4.3	
		6/64		1.10	27,284	11,012	7,642 21.9			2.9	2.0	
36-9	Theobald, "Main"	8/48	125.10		130,988	4,198		494	.51	5.6	1.3	
		6/64		1.22	97,185	0	33,803 25.8			4.3	0.0	

The $R^2 = .75$ and the Standard Error of Estimate (S.E.E.) is ± 8.2 percent as determined by regression analysis. This is point "Y" on Figure 2. This equation can be used to determine the minimum elevation of the principal spillway, if CREPST is estimated for the proposed design life of the reservoir--based on sediment yield and reservoir trap efficiency.

The remaining capacity in the reservoir, which lies below the principal spillway (CPR), can be derived from Equation 2.

$$CO/CT = (D/DT)^{1/NO} \quad (4)$$

Let CO = CPR and D be the remaining depth at the principal spillway. Since the "N" value in the future is unknown and cannot be predicted when predicting CPR, the original NO was substituted. Additional terms were added to the equation to improve the prediction accuracy. The final equation found by nonlinear regression analysis was

$$CPR = .96 (D/DT) .73/NO (CPO/CTO) .23 CTR \quad (5)$$

where D/DT is the remaining depth ratio, CPO/CTO is the original capacity below the principal spillway/total capacity ratio, and CTR is the remaining capacity. The $R^2 = .96$ and the S.E.E. of CPR/CTR = ± 5.1 percent.

The percentage of original capacity replaced by sediment below the principal spillway elevation (CREPSP) can be calculated using CPR from Equation 5:

$$CREPSP = 100 \left[1 - \frac{CPR}{CPO} \right] \quad (6)$$

where CPO is the original capacity below the principal spillway. This point is shown on Figure 2.

The third predictable value in sediment distribution determination is the percentage of capacity replaced by sediment at the remaining mid-depth elevation (CREPSM) (Figure 2). This factor is dependent on the percentage of total capacity replaced by sediment (CREPST). The prediction equation is

$$CREPSM = 28.77 + .86 \cdot CREPST \quad (7)$$

The $R^2 = .68$ and the S.E.E. is ± 8.1 percent as determined by linear regression analysis.

APPLICATION AND DISCUSSION

From the three calculated values (Y, CREPSP, and CREPSM) and the given total capacity depletion (CREPST), the capacity replaced by the sediment curve can be sketched. If the points CREPSM (from Equation 7) and CREPSP (from Equation 6) conflict in the drawing of this curve, the latter should be given more weight since it is more accurate.

If the total capacity replaced by sediment (CREPST) is >20 percent, the shape of this sketched curve can be checked by planimetering the total area above it and the area above it and below the principal spillway. The ratio of these two areas should be approximately equal to the ratio of total remaining capacity (CTR) and remaining capacity below the principal spillway (CPR) determined in Equation 5. Adjust the sketched curve if the ratios are far apart.

Points for plotting the sediment distribution curve (DS) can be computed from capacity replaced by sediment curve (CREPS) and the original stage-capacity curve (CO/CTO). Equation 1 is used for computing the points at various elevations, which gives the percentage of sediment located below a given elevation. This can usually be associated with a given location in a reservoir and can be used for predicting sediment accumulation in that area. The flattest portion of this sediment distribution curve is the area of greatest sediment deposition.

SUMMARY

Sedimentation data from 58 surveys of 41 reservoirs were analyzed to determine sediment distribution of deposited sediment. All reservoirs, except two, were located in the Iowa and Missouri Deep Loess Hills Major Land Resource Area. The sedimentation information curves approach was used in this study.

Linear and nonlinear regression analyses were used to develop prediction equations for various values of the capacity replaced by sediment curve. The minimum design spillway elevation can be predicted if the following are known: (1) total capacity to be replaced by sediment at the end of the reservoir design life, (2) reservoir original "N" value, (3) length, and (4) original depth. The percentage of the remaining capacity that lies below the principal spillway is dependent upon the (1) remaining depth ratio, (2) the original "N" value, and (3) percentage of the total original capacity that was below the principal spillway.

Having the capacity replaced by sediment curve and the original stage-capacity curve, points can be computed for drawing the sediment distribution curve. The capacity replaced by sediment curve and its corresponding sediment distribution curve can be predicted for any estimated reservoir storage depletion in the Iowa and Missouri Deep Loess Hills.

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The Role of Sediment Problems in Hydroelectric Development

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(Note: The views and opinions in this paper are those of the author and do not necessarily reflect those of the Federal Power Commission.)

ABSTRACT

When an artificial interference, such as a dam, is built on a graded river, the streamflow characteristics and the sediment carrying capacity of the river change. These modifications upset the equilibrium already established in that river. In turn, this unbalanced situation sometimes has a significant and unexpected impact on the environments in the vicinity of and, possibly, far away from the dam structure.

This paper briefly defines the sedimentation problems involved and discuss the factors of concern in the design and planning of a hydroelectric project. Considerations from the engineering, environmental, and economic point of view will be included. Also, deficiencies in the present technology and necessary research are discussed.

Purpose & Scope

Changes in the process of sedimentation -- erosion, transport, and deposition of sediments -- due to dams can cause engineering, economic, and environmental impacts. A knowledge of the extent of these impacts must be brought to the attention of the decision-makers who approve the construction and operation of dams. This paper outlines, therefore, the kinds of information which engineers associates with planning, designing, and evaluation of water resource projects (particularly hydroelectric) must provide before any action is taken by the decision-makers. All of these

problems seem obvious, in the past, however, many projects were undertaken without giving prior considerations to the impacts of sediment on the environment downstream, upstream, and sometimes, of even the reservoir itself. In fact, many old dams with sediment behind them have been removed without considering the possible and serious consequences of filling downstream channels prior to the removal of such dams.

Whereas a layman may view sedimentation as a local problem, which might cause only some loss of reservoir capacity, engineers have a wider knowledge. This paper is based on this engineering vantage point and outlines the kinds of problems which may occur. The process of sedimentation is considered from a river system point of view; the problems are discussed qualitatively without providing extensive details.

Sediment Problems in Normal Rivers

In an unobstructed normal river, sediments carried by the flow usually consist of two major parts: (1) the suspended load which is kept in suspension by flow turbulence; and (2) the bed load which is rolled along the river bed by some other forces. Quite often, suspended loads come from sheet erosion of the upstream land, but bed loads are the consequences of local channel scouring.

Flow turbulence and the forces moving the bed load are the functions of the velocity which in turn, depends on the flow velocity. When the velocity is high, the flow will have excess energy to carry a greater amount of sediment with it and consequently, it can cause scouring. On the other hand, when the velocity drops, deposition might occur. As the velocity of the flow varies, scouring and deposition may occur from time to time. However, it has long been recognized by scientists that all rivers tend to be graded unless there are external interruptions. This means that both scouring and deposition are only temporal and local phenomena. In other words, they are intermittent and cyclic in both the time and the space domain.

During low flow and even during the initial stages of a flood period most graded rivers have practically no sediment transport and the water is almost clear. However, when the flow rises to a certain stage, the flood gains such momentum so that in addition to the fine materials, the coarser particles will bounce and roll near the bed.

Usually, after the flood has receded, one may see once again a low river stage with almost clear and no sediment transport. When local adjustments of the river bed are not taken into consideration it is possible that the river bed can maintain approximately the same elevation during the passage of the flood. The erosion of the bed material due to increased turbulence of the river is balanced by the deposition of other bed material that is carried from upstream.

In an ungraded river the supply of sediment from the watershed is not always equal to the sediment that the river can carry with its flow characteristics. However, the erosion or deposition process will be restricted mainly to the location where the deficit or excess of sediment load originates.

Sediment Problem in Dammed Rivers

When a dam is built across a river, an artificial interference is imposed on the flow of a river. This disturbs the balance of the regime already established for a river system. Generally, this imbalance will initially be only a local problem, in the vicinity of the dam, and as time goes by, it will gradually propagate upstream and downstream. Sometimes, this problem could even extend to the whole river basin as well.

Sediment Problems in Reservoirs

Practically, the sedimentation of a reservoir is a three dimensional process. When a river enters a relatively large reservoir, the flow velocity becomes only a fraction of what it was originally. Consequently, it will drop the coarse material first and progressively finer fractions further downstream. As a

result of this sediment deposition, a triangular shaped delta in both longitudinal profile and plan view is usually formed.

Along the longitudinal profile, three distinct regions exist according to their pattern of deposit, namely: top-set bed, foreset bed, and bottom-set bed. The top-set bed is located in the upper part of the reservoir where back-water may occur. It has a slope approximately parallel to the water surface profile. The deposits of this region are largely composed of coarse material or bed load and usually, have little effects on the storage capacity of the reservoir. There may be, however, detrimental effects to upstream riparian interests.

The foreset bed region is in the middle of the three regions. It slopes downstream at approximately the angle of the repose of the sediment deposit, the majority of which is the wash load. Usually, this region plays a significant role in the reduction of the live-storage capacity of a reservoir.

The bottom-set region is located immediately upstream of the dam and is the lower part of the deposits in a reservoir. It is primarily composed of fine suspended sediments brought from upstream by density currents. It is situated in that part of a reservoir which is usually considered as dead storage. Since the pre-allocated dead storage can sometimes be very large and this portion can possibly be flushed out of a reservoir if the density current arrives at the sluice gate at the right time, then this portion of the deposits does not create a serious storage reduction problem.

The rate of reservoir silting depends primarily upon four factors, e. g.

1. The rate of sediment flow into the reservoir

In turn, the sediment flow rate depends on: (a) the degree and distribution of various types of erosion, such as sheet erosion; (b) soil types; (c) flow

characteristics; (d) topographic form of the watershed.

2. Trap efficiency of the reservoir

Trap efficiency is defined as the ratio of the volume of sediment trapped in the reservoir to the total volume of sediment flowing into the reservoir. It depends upon: (a) the reservoir storage capacity per square mile of drainage area; (b) the range of particle size of the sediment delivered; (c) the shape of the reservoir; (d) the manner of sedimentation, such as the formation of density currents and flocculation of the sediment load.

3. The purpose of the reservoir

Some flood control reservoirs, sometimes have sluice gates at the stream beds which are closed only during flood periods. At all other times the flow through these gates is unimpeded, sluicing out a considerable part of the sediment deposited during flood storage.

4. Reservoir Regulation & Silting Process

The primary function of a reservoir is the management of water flow; mainly storage of water during periods of excess flow and release of water during times of low flow. To perform this function, the reservoir level will necessarily fluctuate. The size of the fluctuations could range from a few feet to the entire depth of a reservoir depending on the purpose of that reservoir.

Generally, a substantial range of fluctuation in pool level will further complicate the complex sediment deposition pattern as discussed previously. During the high flow seasons the reservoirs will be at or near full pool stage and sediment will be deposited in the upstream part of the reservoir. Then, during the subsequent low flow period, part of the newly deposited sediment will be eroded away due to the drawdown process and redistributed to a downstream, lower portion of the reservoir. In the following high flow season

the previously eroded portions of the reservoir will be filled up again and a new cycle will start.

Upstream Sediment Problems

Upstream from the reservoir, there will be aggradation of the river bed due to deposition in the backwater reaches of the reservoir. This process will extend, in the course of time, further and further upstream. Since the relationship between sediment transport capacity and water surface profile can become highly obscure, the process is rather complicated.

In the upper reaches of the backwater curve the coarsest particles will be deposited and in the lower reaches the finer particles will be deposited. Consequently, the sediment composition of the river bed changes from place to place, thus disturbing the sediment transport capacity versus slope relationship that might have been originally established in the river.

Downstream from the Dam

The construction of a dam will usually produce its greatest benefit in the river valley downstream through the generation of electricity, flood control and a more dependable water supply. Hence, the river valley downstream must be expected to develop economically at a much greater rate. Usually, below the dam the sediment supply is reduced more than the reduction of flow carrying capacity, causing the channel to erode. While a river is in a process of degradation, it not only remove sediment from the river bed, but, after a while, the sediment comes sliding down the banks. Eventually, unless unusual amounts of sediment is discharged from the reservoir, the river downstream will be stabilized at a flatter profile.

Sediment Control Measures and Problem in Evaluation

To control and alleviate a sediment problem, in my opinion, there can be two important types of measures, i.e. temporal (or local) and system approaches.

1. Temporal or Local Measures

This type of measure is easy to design and implement. Due to its simplicity it has been widely used.

This measure can typically be represented by:

- (a) Dredging
- (b) Sediment Diversion, or Trapping
- (c) Bank Protection
- (d) Channelization
- (e) Others

Superficially, this type of measure can sometimes be effective; however, as implied by its name, its effects can only be temporal or local. In spite of this, the operation can still be very expensive.

For instance, the dredging of a channel is not only very costly at the present time, but it would also change the hydraulic gradient above and below the section. Consequently, it would probably cause erosion upstream and induce aggradation downstream. As a result, an expensive dredging process only serves the purpose temporarily and can not solve the problem permanently.

2. System Approach

With this approach, a sediment problem should not only be evaluated and solved from the standpoint of the temporal impacts of fluvial mechanics, engineering design, economics, and environment of a particular site or location. Rather, the long term impacts on a given river basin as a whole should be examined thoroughly before any design is actually implemented.

Ultimate Effect of Dam Construction

In the future, probably, all of our reservoirs will ultimately be filled up with sediments. By that time,

the new river bed, upstream of the dam, would be parallel to the old one (before the dam was constructed) and raised by an amount equal to the height of the dam, provided that important parameters such as river flow (Q), total sediment transport (Q_s), effective sediment diameter (d_s) remains unchanged.

After the reservoir has filled up, some sediments will pass over the dam and thereby slowing the downstream degradation process. Ultimately, the original river profile will be restored if the former parameters, Q , Q_s , d_s , etc. are also restored.

The above discussions are probably more academic than realistic since Q , Q_s , and d_s will never remain constant. However, before a dam is constructed there should be an advanced plan concerning what action(s) should be taken when sediments fill up the reservoir.

The Economic Consequences of Sedimentation

The economic factor is the most important factor that has to be considered by a dam builder. In an economic analysis the major factors to be considered are as follows:

(a) Damage Losses

The major damages caused by sedimentation could be one more of the following items;

- (i) Loss of storage capacity resulting in the loss of electricity
- (ii) Deterioration of equipments and installations, such as turbine blades, pipes, and etc.
- (iii) Flood damage because of backwater effect due to silting up
- (iv) Loss of navigation capacity due to channel modification
- (v) Loss of land due to erosion
- (vi) Loss of recreation facilities
- (vii) Loss of fish production
- (viii) Others

(b) Benefits

Sediments are not always harmful or detrimental. Sometimes, they can also be beneficial. For example:

(1) Some sediments, in particular, fine sediments can serve as fertilizer for agricultural lands. This is why the agricultural productions of some farms are much higher during the year after a flood; (2) As already discussed previously, sedimentation can also serve as agents in stabilizing a fluvial river or sandy beaches. The recently built Aswan Reservoir on the Nile River in Egypt is a good example. Because enormous amounts of sediments were trapped by the Aswan, the river downstream and also the beaches near the mouth of the Nile have had serious erosion problems. One leading Egyptian engineer has suggested that it might have been much wiser if the Aswan Reservoir was built beside the Nile River instead of across it; and (3) Sediment may possibly be an indication of a suitable environment for formation of oil. This possibility is suggested because every large oil field in the world is located underneath a very thick sediment deposit right off-shore the mouth of an existing or a past alluvial river. Even though this process may require many years, if this theory can be verified and some close relationship can be found, it is possible that some time in the future our scientists may find a means to shorten and accelerate this process.

Impacts of Fish

Sediment in water may adversely affect fish in a number of ways: (1) When sediments settle to form deposits, they may (a) silt up spawning beds and (b) fill crevices where invertebrates eaten by fish can breed; (2) When sediments are in suspension, they may cut down the penetration of light. As a result, (a) the productivity of the water could be reduced so that the food supply of certain species of fish is diminished, and (b) the reduction in visibility can make food more difficult to obtain for fishes in the reservoir; and (3) A thin deposit of fine sediment or sludge could seal the surface of reservoir beds and thereby inhibiting the circulation

of water and oxygen, both of which are essential to aquatic life.

In most cases, biological productivity increases during the first few years of a new reservoir, due largely to nutrients from decaying, submerged land vegetation. After a few years the productivity generally stabilizes at a lower level. Biological effects of sedimentation may also contribute to decline of fish populations in older reservoirs.

Impacts on Recreation

In reservoirs and streams, outdoor recreation involves primarily fishing, hunting, boating, and swimming. The development of such activities requires conditions conducive to the development of (a) food and shelters for wildlife, (b) beaches for swimming (c) facilities for launching and berthing of boats, (d) open water areas free of obstructions for pleasure boating and water skiing, and (e) suitable quality of water for swimming and growth of aquatic life.

However, sedimentation may seriously affect all of these activities and reduce the life of such facilities or progressively inflate the maintenance problems beyond rational limits. For instance, silting may cause locks and ramps to become impassable and turn them into monuments on a mud flat. On the other hand, in the absence of this process beaches may be washed away by wave action and become a sandy cliff.

Eutrophication

Eutrophication, the process of nutrient enrichment of water, is usually accompanied by a depletion of oxygen. It often results in symptomatic changes in lakes and reservoirs, including increased production of algae and other aquatic plants, deterioration of fish life, and other responses that impair water uses.

Sedimentation plays a predominate role in the eutrophication process of a reservoir. But the type of sediment which is influential to an eutrophication process

is still unclear and needs to be better defined.

The eutrophication process in a reservoir is initiated when nutrients transported by inflowing sediments are concentrated in bottom deposits and released to the overlying water. This results in an acceleration of biological activity and thereby producing a large proliferation of both plant and animal life within the photosynthetic region which in time dies off and accumulate with the bottom sediments.

The ensuing decomposition process releases the nutrients to start the cycle once again. But the rate of this process is influenced by the quantity and quality of additional sediment inflow.

Sediment Yield and Soil Conservation

As discussed earlier, most fine sediments are derived from sheet erosion on agricultural, forest, or range lands of the river basin. Therefore, certain carefully planned soil conservation practices would effectively reduce the sediment sources and yields. The following are some of the practices currently being adopted in this country:

1. Contour cultivation and strip cropping
2. Converting crop land into good grasslands or pasture
3. Rotation of crops to include meadow in the cropping sequence
4. Changing cultivated fields from row crops to small grain.

Deficiency in Sediment Technology and Research

Although many scientists and engineers have made a great deal of valuable contributions and progress in this field, our current knowledge of sediment problems, including basic fluvial mechanics and engineering applications, is still rather limited. Presently, we are probably still in a qualitative stage. In fact, we are still relying heavily on practical experience and professional judgement.

Tremendous research opportunities still lie ahead of us. The following are some of the questions that still need to be answered.

1. Fluvial Mechanics: (a) basic mechanics and analytical solution, if any, of erosion, transport, deposition, channel stabilization, and geomorphic processes; (b) mechanics of reservoir sediment deposition, including the distribution pattern, density trap efficiency, and methods of control; (c) land erosion mechanics and quantifications of sediment yield in a basin; and (d) interrelation of hydrology, hydraulics, and fluvial mechanics.
2. Environmental Impact: (a) effects of sediment upon water quality, aquatic biota, and reservoir eutrophication process; (b) impact of various sediment control measures, such as dredging, diversion, and etc.; and (c) socioeconomic impact of sediment on an environment.
3. Data Sampling: (a) optimal data length and time interval required; (b) optimal location and spatial density of station; (c) method of analyzing and interpreting data; (d) sampling instruments; and (e) data base.
4. Modelling: (a) physical models (scaling, and construction) and (b) digital simulation and prediction.

Summary

When an engineer is planning or designing a hydroelectric project one of the most important factors to be considered is the sedimentation problem. Unfortunately, the sedimentation problem is very complicated and vague. In many areas of sedimentation questions still remain unresolved.

Nonetheless, the engineer or scientist in charge should utilize all the available technology to study, thoroughly, the problem from a river system point of view. Also, under certain circumstances judgements based on one's past professional experience must be made. Therefore, in the final analysis, it is an engineer's responsibility to provide a decision-maker with the long term engineering, economic, and environmental impacts of a project.

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