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SYMPOSIUM 5
CHANNEL ADJUSTMENTS

SCOUR AND FILL IN AN EPHEMERAL STREAM

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ABSTRACT

The classical concept that mean bed elevation over an entire stream reach is lowered by scour during flood-wave passage and is restored by deposition in the waning flood phase (mean-bed scour and fill) can be challenged. The alternative that both scour and fill occur simultaneously at different migrating loci within a reach (local scour and fill) is more consistent with published field data. The field investigation reported herein suggests that mean-bed scour and fill in a natural uniform channel is minor compared to local scour and fill caused by bedform migration.

This experiment, utilizing a rectilinear array of buried maximum-scour indicators (scour-cords), produced data for contouring of maximum scour and fill in an ephemeral streambed during two floods. In the first flood, 24 cm (9.5 in) of scour and fill was measured for a bankfull flow depth of 23 cm (9 in). In the second, maximum scour and fill was at least 66 cm (26 in) for a bankfull flow depth of 34 cm (13 in).

Estimates of antidune amplitudes for the two floods, based on theoretical models and laboratory and field observations, are 28 to 64 cm (11 to 25 in) and 48 to 97 cm (19 to 38 in), respectively. This suggests that all scour and fill measured by the scour-cord array was caused by antidune migration.

INTRODUCTION

Bed behavior of steep, sand-bed streams during floods is poorly understood because large amounts of suspended sediment make direct bed observations impossible. Bed-elevation soundings from bridges or gaging station cable-crossings (e.g., Pierce, 1916) indicate that the beds of these streams are scoured during flood crest passage and raised to approximately their former elevation on the waning flood. This behavior of the bed during flood passage is called scour and fill. In discussions to follow, reference will be made to scour and fill, mean-bed scour and fill, and local scour and fill. Mean-bed scour and fill is defined as a lowering of mean bed elevation by scour during flood crest passage followed by a return to approximately the initial mean bed elevation caused by deposition on the waning flood. As defined, this scour occurs simultaneously over a stream segment length comparable to the length of the flood crest and the subsequent filling does likewise. Local scour and fill is limited to a reach whose length is comparable to or less than stream width. Local scour and fill can be caused by flow disturbances in narrow gorges or near bridge abutments and other engineering structures, or by normal bedform development and migration. Local scour and fill is not necessarily simultaneous over an entire transverse section across the stream channel and may occur several times

at the same location during a single flood. "Scour and fill" may refer to either mean-bed or local scour and fill, and will be used interchangeably with "bed reworking" where the specific cause is unknown.

The classical interpretation of gaging station scour and fill measurements has been that the entire flooded stream reach experiences scour and fill comparable to that in the measured cross-sections. However, Mitchell (in Leopold and Maddock, 1952) suggests that measurements at gaging stations may not be representative of the whole flooded reach. Lane and Borland (1954) note that gaging stations are typically in narrows and hypothesize that deposition occurs immediately downstream where the stream widens. They propose that, during waning phases of the flood, the pre-flood bottom is approximately restored by deposition in scoured sections by material removed from accumulation sections. In point measurements along extensive reaches of both ephemeral and perennial streams, Emmett and Leopold (1963) find that scour with respect to the pre-flood bed occurs at some time during flood passage at all sections of these reaches.

Interpretations of these field observations generally treat scour of all or part of the streambed as a continuous event, when in fact scour and fill may alternate locally several times during a flood with only a small percentage of the entire reach experiencing scour or fill at any one time (Culbertson and Dawdy, 1964). This suggests that the scour and fill process may be related to movement of bed forms such as dunes during flood passage. Colby (1964) suggests that the maximum scour measured at a point is simply the elevation of the lowest interdune trough to pass that point during a flood, and that net scour is zero except in channel constrictions.

EXPERIMENTAL PROGRAM

A field experiment on scour and fill was conducted in an ephemeral stream in northwest Ventura County, 135 km (84 mi) northwest of Los Angeles in the winters of 1973-74 and 1974-75. The purpose was to apply a previously developed scour and fill recording technique (Miller and Leopold, 1963) more intensively to a limited reach of an ephemeral stream so the pattern and magnitude of total bed reworking within that reach could be determined. Conceptually, mean-bed scour and fill occurring simultaneously over the entire study reach could be approximated by subtracting local scour and fill from total scour and fill.

Study Area. Quatal Creek, an east side tributary of the Cuyama River (Figure 1), heads on the west flank of Cerro Noroeste (2526 m) (8286 ft) and drains approximately 111 square kilometers (43 mi²) in the Los Padres National Forest in Kern, Ventura, and Santa Barbara counties, California (Figure 2). Details of the drainage basin are shown on U. S. Geological Survey, 7.5 minute series topographic quadrangles: Sawmill Mountain, Apache Canyon, and Cuyama Peak. The study-area lies in the Apache Canyon quadrangle. Rocks outcropping in the drainage upstream of the study area provide sediment from clay to cobbles, augmented with boulders from pediment gravels. Geometric mean sediment size for the first 10 km (6 mi) of stream-bed above the study area is 1.3 mm (.05 in),

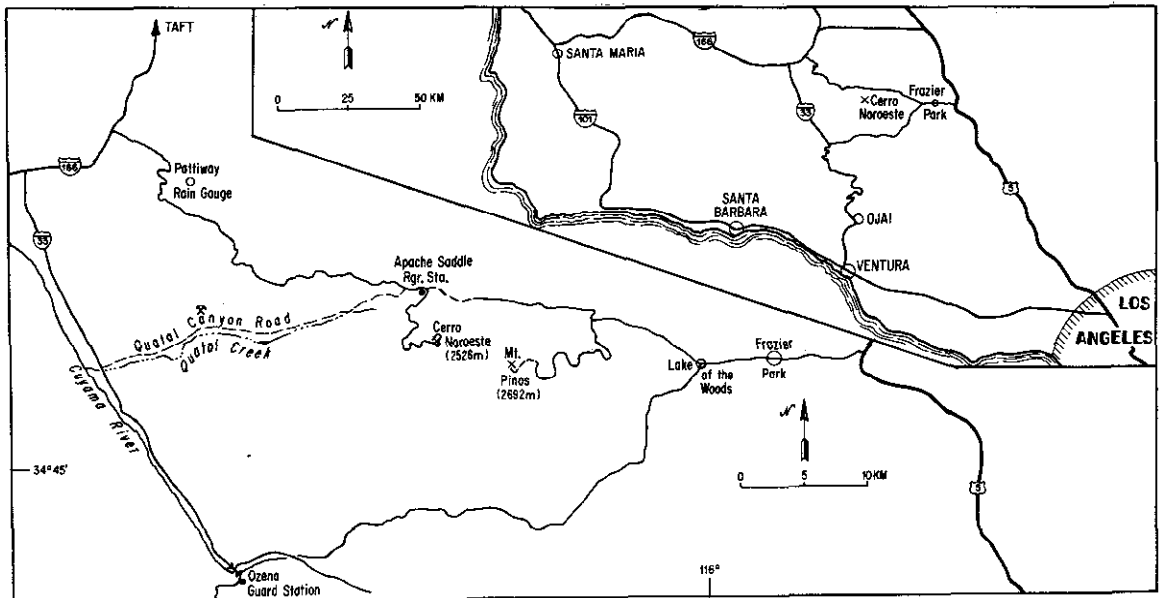


Figure 1. Index map showing location of field area.

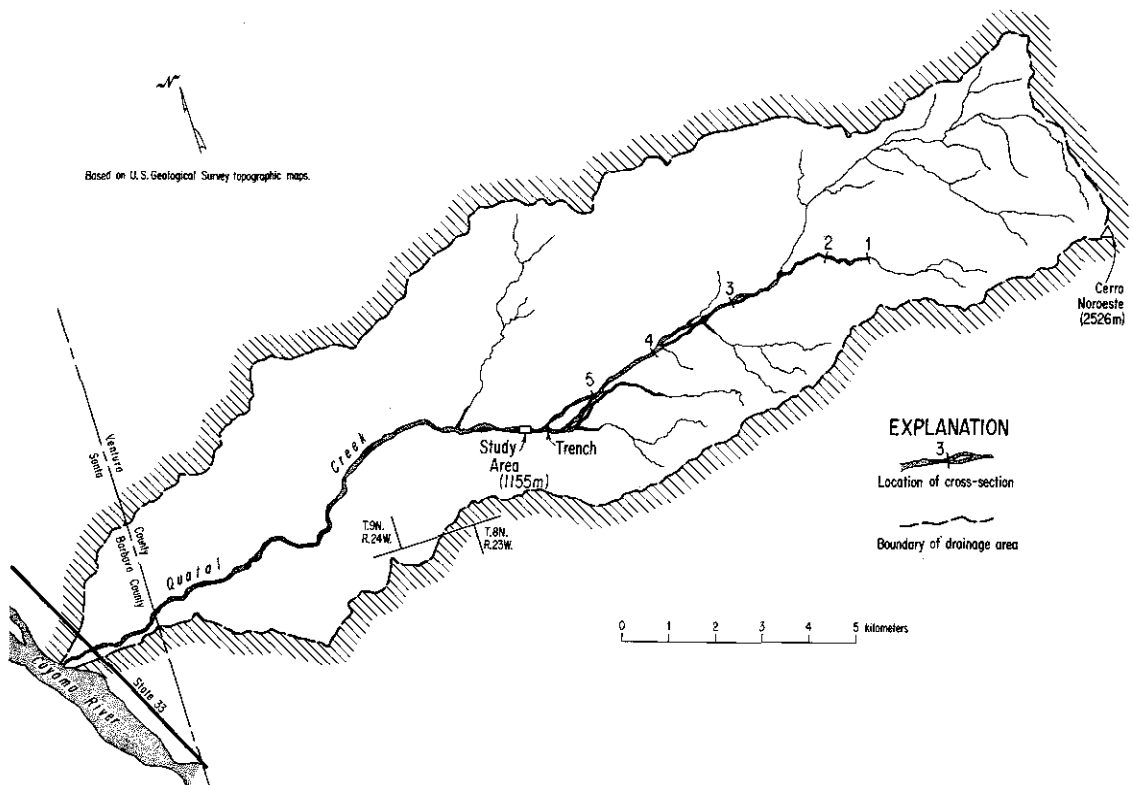


Figure 2. Index map showing Quatal Creek drainage basin.

with a standard deviation of 5.2. By Schumm's criterion (1961, Figure 38) this grain size spectrum and the channel cross-section geometry suggest that Quatal Creek is stable, neither actively aggrading or degrading.

Mean annual precipitation at Pattiway (Figure 1), the nearest rain-gauging station currently operating, is 24.9 cm (9.8 in) over the 1931-1960 period, with most precipitation falling between December and April. Summer thunderstorms can cause significant runoff, but no large thunderstorms affected upper Quatal Canyon during this investigation. All significant runoff events occurred between November and February.

Investigative Procedures. A topographic base map was prepared of the test reach at a scale of 1:1200 with a half-meter (1.5 ft) contour interval, using plane table and telescope alidade (Figure 3). The base map was needed for proper emplacement of a scour-cord network. Previous investigators employed the technique of Miller and Leopold (1963), in which a chain of one- to two-centimeter (.5 to 1 in) links was emplaced vertically in an excavated hole and anchored to the bottom by a rock. Chains were placed singly along the stream centerline with a 150-300 meter (500-1000 ft) spacing, or at 1.5- to 3-meter (5-10 ft) intervals along more widely separated cross-sections (Emmett and Leopold, 1963).

In order to gain better understanding of streambed behavior during floods, a two-dimensional array of 113 scour-cords was installed in the test reach of Quatal Creek. It was anticipated that this array might provide data useful in distinguishing between mean-bed scour and fill and local scour and fill caused by bedform migration. To avoid bias, an equally-spaced rectilinear array was made (see Figure 3). Such an array is inefficient, since some scour-cords are located where little scour or fill will occur, while spots of great scour and fill are as sparsely covered as the least-affected areas. However, any attempt to predict likely areas of major scour and fill in order to design a more efficient array introduces subjective perturbations. The chosen array consisted of 12 sets of 8 to 10 scour-cords, with a 6-meter (20 ft) spacing. Alignment was roughly parallel/perpendicular to the highest banks of the active channel. The array was locally distorted to accommodate a brushy island, since it was felt that the brush-covered parts had not experienced appreciable scour for several years.

In this study, instead of link chain a length of orange 3-mm (1/8 in) diameter nylon parachute cord attached to a penetrator-anchor was used. The scour-cords were driven into the streambed with a 2-meter (7 ft) length of Shelby tubing with an attached 15-kilogram (33 lb) slidehammer. The penetrator-anchor fitted into the open end of the driving tube and the attached parachute cord ran up the tube and out the top to a bulk supply. Using the slidehammer, the tube with the penetrator-anchor as a point was driven into the streambed to a depth of 1 meter (3 ft). The cord was allowed to run freely and loosely through the tube to avoid dislodging the penetrator-anchor from the bottom of the hole, and the slidehammer was then used to extract the tube. The hole was back-filled and tamped using a geology pick, while the cord was held taut. When the hole was filled and the surface leveled, the cord was cut off flush with the streambed.

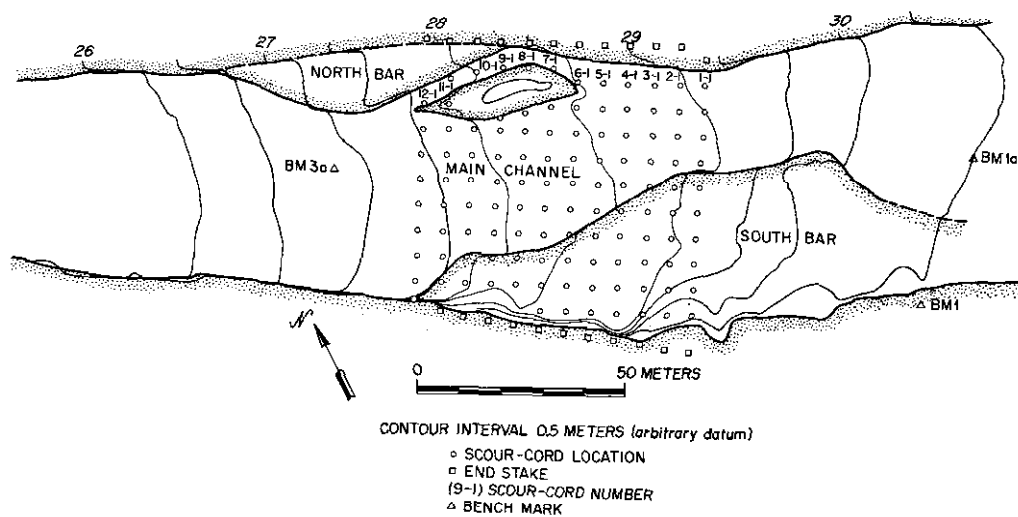


Figure 3. Topographic map of study-area, Quatal Creek.



Figure 4. Recovered scour-cord after January 1974 flood. Scour-cord has been straightened to vertical position.

EXPERIMENTAL RESULTS

Scour-cord emplacement in the study area was completed in November 1973. Scour-cords were recovered in February, October, and December in 1974 and January and March in 1975. The two major runoff events recorded are probably related to storms recorded at Pattiway on 4 or 21 January 1975 and 4 December 1974.

January 1974 Runoff Event. All scour-cords were recovered shortly after the storms of January 1974. An entrenching tool was used to remove small increments of bed material until the scour-cord was seen (Figure 4). The length of scour-cord turned over indicated the maximum scour directly since the end of the cord had been flush with the former streambed surface. Depth of maximum-scour below the new streambed surface was measured to obtain the amount of fill. These data were used to determine the elevation of the post-runoff channel surface relative to the pre-flood surface.

Figure 5a is an enlarged view of the scour-cord array area of Figure 3, with maximum-scour contours shown for flow in the main channel. The south bar was not overtopped and is not included. No signs of over-bank flow on the streambanks in the study area were noted. The scour contoured in Figure 5a is the difference between initial bed elevation and elevation of the bottom of the layer of reworked bed material indicated by the scour-cords. Since the difference between initial and final bed elevation was minor, as can be seen in cross-section AA' in Figure 5a, scour can be measured from either bed surface.

December 1974 Runoff Event. All but 13 scour-cords were recovered eight days after the storm recorded at Pattiway in December 1974. The 13 unrecovered scour-cords were below the temporary water table in the streambed. All were recovered later under drier conditions save one that was presumed lost because scour depth exceeded burial depth. Maximum scour from the pre-flow bed was at least 88 cm (35 in), and possibly in excess of 1 meter (3 ft) at the missing scour-cord location. The contoured map of maximum scour (Figure 5b) is drawn with respect to the pre-flow surface. Modification of the south bar was more extensive than for the January 1974 event, but not major. Maximum bed reworking from the post-flow bed was 73 cm (29 in) in the study reach, and 66 cm (26 in) at cross-section AA' in Figure 5b.

Net Bed Elevation Change. Comparison of survey data from November 1973 and March 1975 shows a net increase in mean bed elevation of .033 m (1.3 in) in the main channel. This small change encompasses at least four runoff events in which the upper .05 to .90 m (2 to 35 in) of bed was reworked. The prediction of active-channel stability made above thus seems valid for short period observations.

DISCUSSION

It will be argued herein that scour shown in Figures 5a and 5b suggests that maximum bed reworking was by antidunes. In January 1974, maximum scour occurred in three troughs subparallel to the flow. In natural streams, antidunes frequently occur in trains with widths smaller than

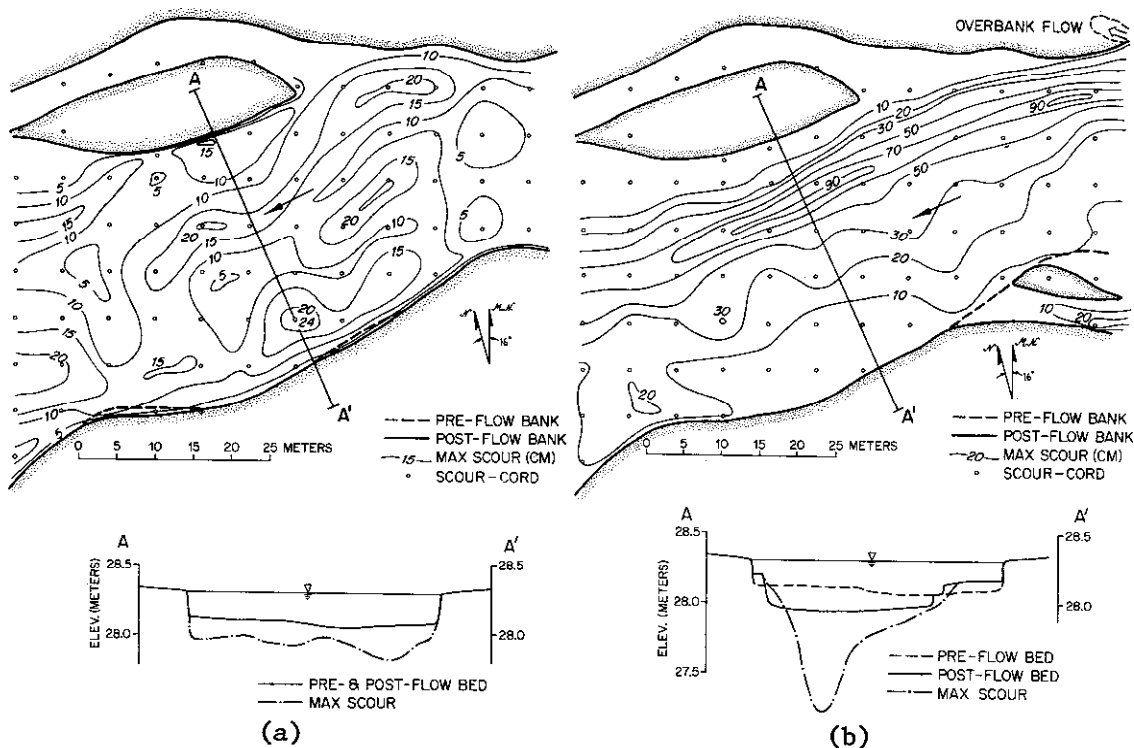


Figure 5. Maximum scour map; (a) January 1974 flood, (b) December 1974 flood.

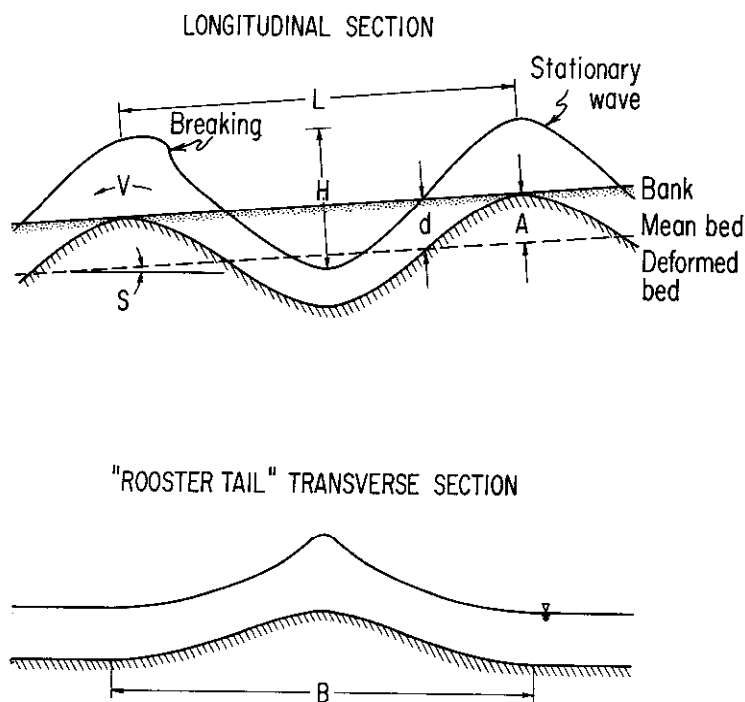


Figure 6. Stationary wave and antidune geometry, vertical exaggeration 2x.

stream width (e.g., Pierce, 1916, p. 42-43). A train of slowly-moving antidunes reworks the streambed along the axis of the train. Since greatest bedform scour and fill occurs where antidunes are the largest, a scour-fill trough results. Figure 5a suggests that three separate trains of antidunes operated during the January 1974 runoff. Figure 5a suggests that the December 1974 runoff had only on such train, and further that the main part of the flow occupied the outside of the bend around the south bar.

Analysis of Flow Behavior of a Steep Sand-Bed Stream. The dune is the most common bedform in alluvial streams, but its amplitude is not rigorously fixed by a specified set of flow parameters (Vanoni, 1971). Antidunes are less common, as they only occur in steep, sand-bed streams, but their maximum amplitudes have a consistent relationship to flow parameters (Kennedy, 1961). For antidune regime, flow velocity can be estimated to within 50 percent if channel geometry and water depth are known.

The steep gradient of Quatal Creek (2.3%) suggests that all flood flows should be in the antidune regime. The relatively flat bed left by floods shows that no dunes form on the waning phase. If dunes had formed in some part of the waning flow, they should be visible as they would not have been destroyed by subsequent lower discharges. Thus, antidunes were presumably the largest bedforms developed during runoff events in Quatal Creek, and flow velocities and antidune amplitudes can be analytically predicted even though flows were not measured, there being no gaging station on Quatal Creek.

If a flow is assumed to be in equilibrium, that is not changing its depth or velocity along a uniform channel at constant slope, the gravitational driving force is balanced by channel frictional resistance. The mean flow velocity, V , can be calculated using the definition of the Darcy-Weisbach friction factor (Kennedy, 1971, eqn. 2-F.2):

$$V = \left(\frac{8grS}{f} \right)^{1/2} \quad (\text{Equation 1})$$

f = Darcy-Weisbach friction factor

where r = hydraulic radius = $\frac{wd}{w+2d}$ (for rectangular channel)

S = channel slope $\ll 1$.

If channel geometry is known and bankfull flow is assumed, the only unknown on the right side of this equation is f . Since f is dependent both on bed-sediment grain roughness and bedform roughness, it cannot generally be determined where bedform size is unknown. However, Taylor and Brooks (1961), and Kennedy (1961) have found for antidune flow:

$$1 \leq f/f' \leq 2 \quad \text{where } f' = \begin{array}{l} \text{equivalent pipe friction} \\ \text{factor for bed sediment} \\ \text{grain roughness.} \end{array}$$

Nordin (1964, Figure 7) found for antidune-regime flow in the Rio Grande that

$$0.9 \leq f/f' \leq 2 \quad (\text{Equation 2})$$

for $d_g = 0.29 \text{ mm}$ (.01 in), $\sigma_g = 1.62$. Apparently, in such a stream the turbulence-damping effects of suspended sediment reduces the friction factor below that for grain roughness of a flat bed (Vanoni, 1944). After f' is determined from an appropriate pipe-friction chart (e.g., Moody, 1944), the range of possible flow velocities can be calculated.

Kennedy (1961) has found that wavelength (L) of antidunes and stationary waves (Figure 6) is related to mean flow velocity by the equation:

$$V = \left(\frac{gL}{2\pi} \right)^{1/2} \quad (\text{Equation 3})$$

This relation was confirmed in field measurements by Nordin (1964). However, equation 3 is only applicable to stationary waves with crests that are wide relative to wavelength. Three-dimensional stationary waves, called rooster tails, are more common in streams. The velocity-wavelength relation derived using inviscid flow theory by Kennedy (1961) for this type of stationary wave and antidune is:

$$V = \left(\frac{gL}{2\pi} \right)^{1/2} \left[1 + \left(\frac{L}{B} \right)^2 \right]^{1/4} \quad (\text{Equation 4})$$

where B is the transverse wavelength (Figure 6). Kennedy (1961) found qualitative experimental agreement with this relation. Thus, if B can be estimated, the range of L can be determined from the flow velocities calculated using equations 1 and 2.

Kennedy (1961) found that maximum stationary wave height, $H_{\max}$ (Figure 6), was:

$$H_{\max} = .142 L \quad (\text{Equation 5})$$

Finally, Kennedy derived the relation between stationary wave height and antidune amplitude a for an inviscid flow:

$$\frac{a}{H} = \frac{1}{2} \left(1 - \frac{g}{V^2 K} \tanh kd \right) \cosh kd \quad (\text{Equation 6})$$

where $K = \frac{2\pi}{L}$ = wave number.

Therefore, using equations 1, 2, 4, 5, and 6, the range of maximum possible antidune amplitudes can be determined for the assumptions of bankfull flow and breaking stationary waves.

January 1974 Runoff Event. For a given channel geometry and bed material, equations 1 through 6 show that the shallower of two flows has a higher relative roughness, hence higher friction factor. This means that the shallower flow has a lower velocity and smaller antidunes. For bankfull stage, the shallowest, hence most conservative, case occurs when there is no mean-bed scour and flow depth is the bankfull depth of the undisturbed channel (Figure 6). Table 1a lists the results of calculations using equations 1 through 6 for the undisturbed channel bed

at cross-section AA' of Figure 5a.

TABLE 1. CALCULATED FLOW BEHAVIOR

		(a) January 1974	(b) December 1974
d	(m)	0.229	.344
W	(m)	32.8	32.8
S	-	.0229	.0229
$\frac{d}{4d}$	-	.0022	.0014
f'	-	.0241	.0210
f	-	.0482 - .0217	.0420 - .0189
V	(m/sec)	2.92 - 4.35	3.84 - 5.72
B	(m)	14.63	15.54
L	(m)	5.15 - 10.01	8.33 - 15.05
H _{max}	(m)	.736 - 1.43	1.19 - 2.14
$a_{\max}/1/2 H_{\max}$	-	.772 - .891	.803 - .907
$a_{\max}$	(cm)	28.4 - 63.7	47.8 - 97.0

In the calculations in Table 1a, flow depth is used instead of hydraulic radius because it gives maximum flow conditions in the channel, and not just average flow conditions for the cross-section. Kennedy (1961) notes the desirability of using local flow conditions for antidune calculations in natural streams, where maximum flow conditions may vary greatly from the mean because of irregular channel cross-sections. The difference between maximum depth and hydraulic radius for the January 1974 runoff event is only 1.4 percent, so the distinction is not very important. The transverse wavelength of stationary waves is taken as the width of the maximum-scour trough nearest the south bar in cross-section AA', Figure 5a.

December 1974 Runoff Event. Calculated flow behavior for December is the same as for January (Table 1a), if initial channel cross-section geometry is used. Maximum bed reworking from initial bed elevation in cross-section AA' (Figure 5b) was 82 cm (32 in). At the maximum antidune amplitude of Table 1a, this amount of reworking implies an additional 18 cm (7 in) of mean-bed scour. Since 17 cm (6.7 in) of net scour of the channel occurred during the December event, use of the initial geometry is valid only if the net scour occurred after peak flow.

If the observed net scour had occurred before or during peak flow, an antidune amplitude range of 48 to 97 cm (19 to 38 in) would have resulted (Table 1b), enough to account for all the bed reworking. Behavior of the channel as a whole supports this interpretation. Net fill, mostly fine sand, occurred at the edges of the scoured channel (Figure 5b, cross-section AA'). Scour-cords showed no reworking of the initial bed under this fill, suggesting that filling occurred early in the runoff event and was not disturbed by subsequent antidune development near the banks at peak flow. This asymmetry of channel reworking with net scour

in one part and fill in another contrasts with channel behavior in the January 1974 runoff event. It appears that the December flow was modifying the channel toward a smaller width-depth ratio. The smaller width-depth ratio implies that the December flow carried a larger proportion of fine material than the January flow (Schumm, 1960) and the relatively fine-grained nature of the channel-edge deposition supports this implication. It is more likely that this modification occurred at peak flow than on the waning flow, so flow conditions of Table 1b probably prevailed.

Comparison between calculated and observed maximum scour supports this conclusion. The January flow inferred from scour-cord measurements is at the lower end of the calculated friction factor range of Table 1a, while the December flow is in the middle of the calculated range of Table 1b. This suggests that the January flow had a higher friction factor ratio than the December flow, hence a coarser bedload and smaller proportion of silt and clay. These differences in load material can be caused by differences in thunderstorm rainfall distribution, resulting in discharge from tributaries with different source area sediment sizes.

Although it is possible to account for bed reworking in the December 1974 runoff event entirely by antidune development, under the assumption that the transported load was more fine-grained than usual, an alternate possibility is that the December flow asymmetry developed because part of the streambed was frozen. The southern half of the streambed is shaded by a cliff, and in episodes of cold winter weather it escapes diurnal thaw. Under such conditions a midday runoff event would scour the northern part of the channel more easily than the southern part. However, the flow would probably thaw the thin (less than 5 cm thick) frozen layer on the southern part of the channel before the end of peak flow, and an initially partly-frozen bed does not explain the fine-grained fill on both the northern and southern channel edges. The assumption of fine-grained load is thus preferred.

CONCLUSION

Data derived from scour-cord measurements resulting from floods in a natural ephemeral stream channel in January and December 1974 showed bed scour and fill equal to peak flood water depth in January and twice peak water depth in the December flood. Steep channel gradient and lack of residual dunes after flooding suggests streamflow was in the antidune regime during these floods. Maximum antidune amplitudes calculated theoretically assuming bankfull flow, no change in mean streambed elevation, and breaking stationary waves were equal to or greater than measured scour and fill. Thus, all scour and fill measured by the scour-cords can be explained theoretically by bedform migration, suggesting that mean-bed scour and fill is of second order compared to local scour and fill.

ACKNOWLEDGMENTS

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CHARACTERISTICS OF STABLE NATURAL CHANNELS

AND THEIR RELATION TO CHANNEL DESIGN

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ABSTRACT

Interest in the design of stable stream channels has been spurred by recent problems of channel stability on artificially straightened or newly constructed channels. Solutions are not readily available.

This is the second of two papers dealing with stream channel stability. The paper is based on data collected at straight, stable reaches of a large number of stream gaging stations in or near Oklahoma. Cross sections of two different straight, stable reaches at each station were used to calculate channel slopes, cross-sectional areas, and other characteristics. Samples of the bank and bed materials were analyzed for Atterberg limits, pH, particle-size distribution, angle of repose, and cation exchange capacity. The first paper presented a statistical evaluation of the relation between the channel characteristics and the characteristics of the bank and bed materials.

This paper presents the regime and tractive force theories of channel design, based on the assumption that the channels are in balance, and relates the bank and bed materials characteristics to the channel characteristics through the regime and tractive force equations of stable channel design.

INTRODUCTION

Recent criticism of channelization requires that we evaluate the techniques used for channel design, particularly in alluvial materials where most of the problems exist. Several different methods of channel design, such as those based on the tractive force and regime concepts, have been used but no one method appears to be dependable.

This is the second of two papers on the design characteristics of straight, stable alluvial channels as related to their flow and site characteristics. It examines the regime and tractive force concepts of channel design, and relates parameters of these equations to site characteristics in an attempt to find physical characteristics of the materials that may relate to channel shape. The first paper, (DeCoursey, 1975), *Bank and Bed Materials Consideration in Alluvial Channel Design*, is a statistical evaluation of the relation between the channel design characteristics of cross-sectional area, mean depth, and bank slope and characteristics of the bank and bed materials.

DATA SET

Data for these analyses were taken from 75 streams, in and adjacent to Oklahoma. The gaging stations had a minimum of 15 years of flow records, were not regulated by major structures or extensive flood prevention programs, and varied widely in size and alluvial characteristics (Table 1). Several sets of aerial photographs of the channels near the flow-measuring stations were studied to locate two straight reaches in which the vegetation had not changed. These were assumed to be stable reaches. After these had been identified, each reach was surveyed to determine the channel cross-sectional area and samples of bank and bed materials were taken. Samples were taken on both banks to a depth of 1 foot about 1/3 and 2/3 of the distance from the channel bottom to the top of the bank. Samples of the bank materials were then composited for analysis. Channel slope was determined from topographic maps.

After compilation, the data were screened for excessive upstream regulation, unreliable estimates of channel slope, and mean velocity. The data were randomly divided into developmental and test sets; the developmental set consisted of about 2/3 of the sample, 39 sites, and the test set consisted of 15 sites. Two sets of data corresponding to the two surveyed locations at each site describe the channel cross sections

and bed bank material characteristics at each location. Flow rate and channel slope were assumed to be the same at both locations.

The site characteristics considered in this study were the same as those described in the previous study, except that the number of points describing the particle size distribution were reduced from 33 to 8. The 14 variables selected are shown in Table 2.

Nearly all channel design methods are based on observations in which the bank-full flow rate is known. In studying natural channels such as these, the bank-full flow rate is not known and would have to be estimated from rating curves at nearby gaging stations, many of which are located at bridges. For this reason, the 2-year recurrence interval flow rate was selected as the bank-full flow. This nearly corresponds to the mean annual flood of about 2.3 years' return period, which is sometimes assumed to be a bank-full rate. Other estimates of the frequency of bank-full conditions range from 1 year to 4 years depending upon the hydrogeology of the area (Leopold, 1964).

Prior to analyses of the various design equations and a review of the previous work, the differences in channel shape characteristics between the two sets of data collected at each site were compared (Table 3). Information obtained from these analyses are useful in interpreting error patterns of fitting such equations as those of the Regime and Tractive Force methods of channel design. These data show about a 15 percent average difference between characteristics of any two cross sections in similar straight stable reaches and that about 2/3 of the pairs of data will differ by 2 to 30 percent.

PREVIOUS STUDY

The first paper based on an analysis of these data was primarily a statistical evaluation. An analysis of the variance associated with using Manning's equation to size a channel showed that 25 to 40 percent of the variance in cross-sectional area not accounted for by hydraulic characteristics of flow, wetted perimeter and channel slope, could be explained by drainage area, moisture capacity and bank materials size. Factor analyses followed by canonical correlation showed that the materials characteristics that correlated with the channel shape characteristics were the volume weight, moisture capacity, the armoring size (d_{95}), and the percent of material of fine silt size or smaller. Further analyses using regression studies showed that, in addition to the d_{95} size and percent fine silt size or smaller, the pH of the soil material and the angle of repose when wet were also significant in explaining variance in channel shape.

A factor analysis of the data after they had been regionalized into 3 sets showed that each group was related to a different set of site characteristics. Variability within all three groups appeared to be related to the percent of the material that was equal to or less than 0.25 mm and 0.0064 mm. The most significantly correlated site characteristic was different for each group. In one group it was the percent of material fine silt size or smaller, in another group it was the weighted size of the bed material, and in the third group it was the armoring size of the bank material, d_{95} .

CHANNEL DESIGN USING THE REGIME EQUATIONS

Water flowing in an open channel of loose movable materials will create a unique channel shape, which is a function of the flow rate, slope, and characteristics of the bank and bed materials. A channel developed from a uniform flow through such materials will in time reach an equilibrium shape. However, it is very unlikely that natural channels in such materials ever reach equilibrium because flow conditions vary so widely. But most streams appear to stabilize enough that year-to-year changes are minor. These channels are said to be in regime (Graf, 1971). The geometry of these channels must be described by three equations because of shape variance associated with the width, depth, and slope that they develop in attaining equilibrium. Empirical equations used to express these relations are known as regime formulas and their use in design is known as regime theory. Because of the few straight stable reaches in readily accessible places in many river systems, much of the data used in the development of these equations came from large irrigation canals in alluvial areas. Thus, the application of the traditional regime equations to channels of natural rivers must be tested.

The study of irrigation canals in India by Kennedy (1895), and later by Lindley (1919) led to empirical relationships of the form

$$v = c_1 D^{c_2} \quad (1)$$

and

$$v = c_3 B^{c_4} \quad (2)$$

in which v is the mean velocity; D is the mean channel depth; B is the channel width; and c_1, c_2, c_3 and c_4 are coefficients.

The Lacey Approach

Lacey (1929) found that he could represent large quantities of data by an equation of the form

$$v = 1.17 \sqrt{f R} \quad (3)$$

in which f is a silt factor and R is the hydraulic radius. Values of f range from 0.36 for very fine silt to nearly 40 for very large boulders. He later proposed that

$$f = 8 d^{1/2} \quad (4)$$

in which d is the predominant size of the sediment transported.

Considering equation (3) as the first of the series of the three independent equations needed to describe a channel, the second can be written as

$$Q f^2 = 3.8 v^6 \quad (5)$$

in which Q is the flow rate. The third equation is obtained by solving equation 3 for hydraulic radius and equation 5 for velocity, using

$$n = 0.222 f^{0.2}, \quad (6)$$

and substituting these three values, R , v , and n into Manning's flow equation. Solving the flow equation for slope yields the third equation

$$S = 0.000387 f^{5/3} / Q^{1/9} \quad (7)$$

Since these equations were developed from irrigation canals, they may not hold for natural streams because of the influence of bank vegetation on bank geometry. However, Lacey's work is recognized as a valuable contribution to the regime theory of channel design and, therefore, was used as a basis for testing with the Oklahoma data.

Since equations of the Lacey type were based on canals and are empirical, there is no reason to assume that the coefficients in these equations can be applied to natural rivers. Substituting equation (4) into equation (6) shows Mannings' n to be a function of particle size, i.e.,

$$n = k_1 d^{k_2} \quad (8)$$

The relation of n to particle size requires several assumptions, some of which may not be applicable. First, if the bed material is fine, less than about 5 mm, a plane bed conditions with fully turbulent conditions is required. If the bed is other than plane, then particle size and bed form resistance such as from dunes or antidunes combine to create the n value. If conditions are laminar, viscous forces may dominate. Equation 8 is most likely valid for large particles, 5-8 cm, such as those studied by Strickler (1923) or Lane et al (1953).

The equation for velocity (3), is a function of both particle size and hydraulic radius, i.e.,

$$v = k_3 d^{k_4} R^{k_5} \quad (9)$$

and flow rate is a function of velocity and particle size, thus

$$Q = k_6 v^{k_7} d^{k_8} \quad (10)$$

These equations plus Mannings' equation

$$v = \frac{1.49}{n} R^{2/3} S^{1/2} \quad (11)$$

can be used to calculate the hydraulic radius, cross-sectional area and channel slope in terms of flow rate, Q ; particle size, d ; and the empirical coefficients, k_1 – k_8 . Values of the coefficients in these equations are functions of the flow rate and particle size designation selected. Table 4 is a summary of the results obtained from fitting the equations to the Oklahoma data. The 2- and 10-year return period flow rates, the d_{95} and d_{50} particle sizes, and the weighted bank and bed size were selected for consideration. Particle sizes were selected on the basis of previous experience and results of the statistical analyses. Since two cross sections were surveyed at each site, the average rather than individual channel and site characteristics were used. Each equation—cross section area, hydraulic radius, and slope—was fitted independently. The results presented in Table 4 show that the 2-year peak flow rate was much more highly correlated with channel geometry than was the 10-year rate.

All combinations appeared to fit the equations about equally well, however the two year peak, P_2 , and the weighted bank size, d_m , was the best. Although some of the parameter values appeared to be similar to those of Lacey (see bottom line of Table 4), many of the values were quite different. There does not appear to be any logical basis for explaining differences in values of the same parameter in different equations. Values should be the same in all three equations. Differences may be due entirely to "curve fitting" of the data.

The Blench Approach

More recent efforts focusing on Lacey's work attempted to separate the effects of the bank and bed on the silt factor, thus tending to generalize the regime equations. Blench (1961) expressed these as

$$F_B = \frac{v^2}{D} \quad (12)$$

for the bed factor and

$$F_S = \frac{v^3}{A/D} \quad (13)$$

for the side or bank factor where D is the mean depth, A is the cross-sectional area, and v is the average velocity, at bank-full capacity.

Given a channel design problem, F_B , F_S , and bank-full capacity, one can calculate the average channel width, mean depth, and flow velocity. If the kinematic viscosity and sediment concentration are known, one also can estimate values of the channel slope (Graf, 1971).

To be useful in channel design, values of F_B and F_S must be estimated for a given site; therefore, values of both were calculated from the Oklahoma data. Velocity for use in equations (12) and (13) was calculated from average cross-sectional area and the 2-year frequency peak flow rate. Average values of F_B and F_S along with the range and standard deviation of calculated values are presented in Table 5. The average value of about 5 for F_B appears reasonable when compared to values obtained from a relation recommended by Blench (1964)

$$F_B = 1.9 \sqrt{d} \left| 1 + 0.12 \bar{C}_s \right| \quad (14)$$

in which d is the median bed material size by weight and $\bar{C}_s$ is the bed load in parts per 100,000 by weight. At a concentration of 1,000 ppm and a mean bed size of 0.07 mm, about average for the Oklahoma data, F_B would have a value of about 6.5. Blench (1957) recommends values of F_S ranging from 0.1 for slightly cohesive soils to 0.3 for very highly cohesive soils. The average value of 1.6 for the 39 stations is significantly different from that of Blench and is probably due to the effect of bank vegetation.

For predictive purposes, we attempted to relate the values F_B and F_S to the site characteristics presented in Table 2. A factor analysis was used to find the most significant variables of this group for both

F_B and F_S . A components regression was then run on both values using these variables. Results of both the factor analysis and components regression showed F_B was more highly correlated with site characteristics than was F_S (Table 6). The most significant site characteristics related to F_B and F_S are the angle of repose of the bank material when wet; the armoring size, d_{95} ; the median size, d_{50} ; the percent of material less than 0.25 mm; and the percent of material fine silt size or smaller, 0.0064 mm. The coefficients for the variables are shown in standardized form so that they can be compared for significance. The predictability of F_B and F_S is poor, as shown by both the squared correlation coefficient, R^2 , and the error standard deviation, SEE.

To test the stability of the prediction equations, they were tested on the alternative data set consisting of 15 sites. Values of F_B and F_S on the average appear to be somewhat different from the developmental data as indicated by their mean, range, and standard deviation as shown on Table 5. The test data appear to be biased toward lower values. Results obtained in predicting values of both F_B and F_S using the equations from the developmental data set are presented on Table 6. The standard error of prediction, SEE, is considerably greater for the test data than for the developmental data. This may be due in part to the bias previously described. Even though predictability is poor, the fact that both F_B and F_S appear to be related to the site characteristics shown on Table 6 is significant.

The Simons Approach

Simons (1963) evaluated data obtained from many canals in the United States as well as data cited by other researchers for canals in other countries. The data covered a wider range than those previously reported. He found that values of the coefficients and exponents in the regime equations depended on the types of canal and bed materials.

He defined the regime equations as:

$$A/D = 0.9 K_1 \sqrt{Q} \quad (15)$$

for average width,

$$\begin{aligned} D &= 1.21 K_2 Q^{0.36} && \text{for } R \leq 7 \text{ ft} \\ D &= 2.0 + 0.93 K_2 Q^{0.36} && \text{for } R \geq 7 \text{ ft} \end{aligned} \quad (16)$$

for average depth, and

$$\frac{v^2}{gDS} = K_4 \left(\frac{v A/D}{v} \right)^{0.37} \quad (17)$$

for a slope relation. In these equations, g is the gravitational constant, 32.2; v is the kinematic viscosity; and all of the other variables are as previously described. Values of K_1 , K_2 , and K_4 were presented in Henderson (1966) and later in Graf (1971) for the types of channels differentiated. This material is reproduced in Table 7.

The data previously described and used in evaluating the Blench equations were also used to evaluate the parameters K_1 , K_2 , and K_4 in equations (15), (16), and (17) respectively. There was no attempt to segment the data, since all sets were assumed to have sandy banks and sandy to coarse beds. Average values of the parameters K_1 , K_2 , and K_4 are 2.33, 0.35, and 0.21 respectively. The range of calculated values and their standard deviations are shown in Table 8. The average values are also shown for comparison purposes on Table 7 along with the values presented by Simons. The calculated values of both K_1 and K_2 would place them in the range of a sand bed with cohesive banks, although the value of K_2 also could be interpreted as indicating a heavy sediment load. These interpretations agree with results of the Blench evaluation in that bank vegetation would indicate a response similar to one obtained from a cohesive material.

Attempts were made to relate the individual site values of K_1 , K_2 , and K_4 to site characteristics as in the Blench equation evaluation. Results of a factor analysis followed by a components regressions are shown in Table 9. Site characteristics considered were those shown in Table 5. None of the correlations was

highly significant and the error standard deviations were almost as great as the standard deviations of the observed values; however, they indicated interesting results. The depth factor, K_1 , was most highly correlated with size characteristics of the bank materials, the moisture capacity and the pH of bank material. The width factor, K_2 , was most highly correlated with the angle of repose of the bank material when wet, the liquid limits, the armoring size and the percent of material smaller than 0.25 mm. The pH of the bank material and percent of bank material fine silt size or smaller, 0.0064 mm, were also relevant. The slope factor had the poorest correlation with site characteristics, $R^2 = 0.16$. This was to be expected as both channel slope and viscosity of the water are subject to sizeable estimation errors. Slope was most highly correlated with particle size characteristics and slightly with the angle of repose in a wet condition.

Comparisons of these results with those obtained by using the equations on the test data are also shown on Tables 8 and 9. Values of K_1 , K_2 , and K_4 for both sets of data, the developmental and test sets, show very similar values for the mean, range, and standard deviation. This is in direct conflict with that obtained for F_B and F_S in the Blench equation. Standard errors of prediction using the equations obtained from the developmental data set on the test data show errors of about the same order of magnitude as on the developmental set, indicating a stable prediction equation even though it is poor, i.e., SEE for K_1 , K_2 , and K_4 are 0.92, 0.14, and 0.19, respectively (See Table 9).

CHANNEL DESIGN USING THE TRACTIVE FORCE EQUATIONS

Stream channel design using the tractive force, or boundary shear, concept is based on the theory of sediment transport equilibrium. It is assumed that equilibrium exists when particles of a certain size are held in pending motion; smaller particles are in suspension. No one particle size fraction is representative of equilibrium; however, the median particle size is often used.

Channel design using these concepts stress three physical properties of channels (Raudkivi, 1967).

1. The boundary shear force over the wetted boundary is not constant
2. Resistance to displacement of particles is different on the sides of the channels than on the bottom
3. The capability of the boundary material to withstand drag forces differs greatly from one material to the next

Further it is assumed that in a stable channel the shear forces and resistance to displacement are in equilibrium at all points in the channel at the same time. In very narrow channels, the force on the banks is critical, whereas in very wide channels, the predominant force is that on the bed. The width to depth ratio of the channels being studied ranged from about 7 to 200 and had an average value of about 30. Therefore, the predominant forces are bed forces. The sites being studied are also stable and the banks are vegetated. For these reasons a two-dimensional analysis involving only the tractive forces on the bed will be discussed.

Many different sediment transport or bedload equations are used as a basis for development of the channel design equations. These bedload equations related bedload transport to the flow rate, depth of flow, particle size, etc. In earlier studies, the Duboy-type equations were used; however, more recent developments use the Shields bedload equation.

Channel design equations for wide channels are obtained by solving the bedload equations under equilibrium stream bed conditions for the particle size at the threshold of movement. For example, under turbulent conditions and assuming a value of 2.6 for the specific weight of sediment, the Shields bedload equation can be written as

$$\frac{G}{\gamma q} = S (3.91 \frac{D_s}{d} - 0.35) \quad (18)$$

in which G is the weight of sediments being transported, γ is the specific weight of water, q is the flow rate per unit width, and the other variables are as previously described.

By expressing the slope, S , in terms of Manning's equation and substituting into equation (18), one obtains a function that can be solved for velocity in terms of slope, the sediment concentration, the flow rate and depth, and particle size. This equation plus the particle size distribution can be used to size a channel such that particles of the size present in the channel will be at the threshold of movement.

In the Oklahoma study, no sediment concentration data were available; therefore, a simpler expression is needed.

During bank-full flow, most streams are highly turbulent. Under these circumstances entrainment or suspension of the sediment, according to Shields (1936), is no longer a function of particle size Reynolds number but is a constant for particles of a given size at the point of incipient motion:

$$\frac{\tau}{\gamma(S_s - 1)d} = 0.056 \quad (19)$$

If the specific gravity of the sediment particles, S_s , is 2.65 and $\tau = \gamma D S$,

$$d = 11 D S \quad (20)$$

If the channel roughness, n in Manning's equation, is related to particle size as is often the case (Lane, 1953),

$$n = j_1 d^{j_2}$$

in which j_1 and j_2 are constants; then the velocity according to Manning's equation can be expressed in terms of flow depth and particle size:

$$V = \frac{0.45}{j_1} D^{1/6} d^{1/2-j_2} \quad (21)$$

The cross section area is

$$A = Q/V = \frac{Q j_1}{0.45 D^{1/6} d^{1/2-j_2}} \quad (22)$$

Similarly, the mean flow depth can be expressed as

$$D = \left(\frac{Q j_1}{0.45 B d^{1/2-j_2}} \right)^{6/7} \quad (23)$$

in which B is the top width at the bank-full flow.

The channel slope can be expressed as

$$S = \left(\frac{0.027 B d^{5/3-j_2}}{Q j_1} \right)^{6/7} \quad (24)$$

The coefficients j_1 and j_2 in equations 22, 23, and 24 were fitted to the set of data previously described. The results are presented in Table 10: The 2-year peak was assumed to be representative of the bank-full flow rate. However, for comparison, the 10-year value is shown for one case. Since there is no agreement as to the particle size that should be used for d , several particle size combinations were considered: d_{95} , d_{50} , d_m , and d_{bm} . The d_{95} value was selected because it is characteristic of the armoring size and could be significant if armoring occurs. The d_{50} value was used because it is the size most often used as a reference. The d_m value was used because it was the best and also the most stable variable in the previously described statistical evaluation. d_{bm} was included because in the two-dimensional case it is the variable most characteristic of channel roughness and bed conditions. Perhaps the most significant fact presented in Table 10 is the lack of uniformity in fitted values of the coefficients. If a stable relation is present, values of j_1 and j_2 should be the same whether they are fitted on area, mean depth or slope. However, values obtained in fitting the equations of area and mean depth were quite similar. The large discrepancies in fitted values from the channel slope equation may be due to errors in the values for slope used for the sites. In fitting the parameters using errors in cross-sectional area as the objective function to be minimized, no one combination of flow rate and particle size is dominant; however, the 2-year peak flow rate and the median particle size on the banks appears to be the best.

In fitting j_1 and j_2 to cross-sectional area, the best combination was the 2-year peak and the weighted bank size, however the coefficient values are unrealistic and indicative of "curve fitting". The most realistic set is the 2-year peak and the median particle size. These coefficient values are also similar to values recommended by previous investigators (See Table 11). In fitting against the mean depth and the slope, the d_{95} bank and weighted bed particle sizes respectively gave the best results. However, values of the parameters j_1 and j_2 were very different from values recommended by previous investigators. Raudkivi

(1967) presents the results of several previous studies. These values are summarized in Table 11.

No attempt was made to relate the coefficients of n to other site characteristics because (1) the values of the coefficients were not similar to previously estimated values and (2) none of the coefficient values for the available channel characteristics related to channel form or roughness.

CONCLUSIONS

The Blench and Simons regime equations gave considerably different results. These equations gave about the same degree of correlation between observed and computed parameter values. However, comparison on the test data showed considerably more stability with the Simons parameters than those of Blench. Also, even though eight coefficients were used in the Lacey regime equations, the degree of fit to the data was not much better than that obtained in the fitting of the two coefficients in the Tractive Force Equations. Table 12 compares these results with those of the previously reported statistical evaluation. These analyses would indicate that the Simons equations have the most potential for use in natural rivers. However, much more evaluation would be needed to recommend one method over another.

Raudkivi (1967) compared the regime and tractive force methods of stable channel design and presented the three Lacey equations; they are functions of flow rate, wetted perimeter, hydraulic radius, channel slope, and the "silt factor." He stated that to satisfy these equations

$$S = \frac{r^{5/3}}{1750 Q^{1/6}} \quad (25)$$

in which the variables are as previously described. Observed channel slopes should be greater than this value. If the slopes were less than this, the flow would not have enough energy to transport the bed material and the channel would aggrade. Using equation 24 as a criterion, we evaluated data from the surveyed channels. Only eight of about 40 stations, about 20%, had observed slopes less than the critical value. This indicates that most of the channels were in a regime condition. Raudkivi (1967) also states that in regime conditions if

$$P \propto Q S^x \quad (26)$$

and if

$$P \propto Q^{1/2} \quad (27)$$

in which P is wetted perimeter, then

$$S \propto Q^{-1/2x} \quad (28)$$

with values of x ranging from 3 for a plane bed to 2 for a dune bed. Analyses of the data from the surveyed stations showed that

$$P \propto Q^{0.84} \quad (29)$$

Thus,

$$S \propto Q^{-0.16/x} \quad (30)$$

The Oklahoma data fitted to equation 30 yielded a value of $x = 0.55$, quite different from the previously reported values.

Results obtained in the tractive force analysis deviated significantly from those of previous investigators (Table 8) in the relation between Manning's n and particle size. Part of this discrepancy is due to the relation between n and particle size being valid for only a plane bed. The values of j_1 and j_2 obtained from the data reported here indicate a much larger " n " than that obtained using the results of previous studies. This is to be expected because the natural streams surveyed did not have a plane bed but were in various stages from ripples and dunes through antidune. The coefficient values obtained resulted from attempts to force some of the roughness associated with grain size to reflect total channel roughness.

SUMMARY

These analyses of the regime and tractive force methods of channel design were carried out to determine if the geometry of stable natural channels in straight reaches is related to site characteristics. The first paper on this subject presented a statistical evaluation of this relation, which showed that the most significant variables related to channel geometry were the volume weight; moisture capacity; angle of repose of the bank material, d_{95} ; the percent of bank material fine silt size or smaller and the weighted bed material size. Results of the regime and tractive force studies showed that in addition to these variables, the liquid limit, the median size of the material, and the percent of the material smaller than 0.25 mm were also significant. Future studies of the regime and tractive force equations of channel design should consider a more in-depth evaluation of the physical relation between geometry and site characteristics.

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TABLE 1
Mean and Range of Watershed and Site Characteristics

Characteristic	Range	Mean
Drainage area, sq. mi.	31 - 54,000	1,278
Average basin rainfall, inches	16 - 57	35
Average basin runoff, inches	0.10 - 23.0	2.9
Channel slope, ft/mile	0.75 - 10.5	3.5
Cross-sectional area, sq. ft.	292 - 12,300	1,900

TABLE 2
Site Characteristics

Volume weight (loose)	w	Size with 50% less than	d ₅₀
Angle of repose (dry)	a _d	Size with 25% less than	d ₂₅
Angle of repose (wet)	a _w	% of sample smaller than 0.25mm	X _{0.25}
Maximum moisture capacity	m	% of sample smaller than fine silt size, 0.0064 mm	X _{0.0064}
pH of material in water	pH		
Liquid limit	ll	Weighted bank size	d _m
Size with 95% less than	d ₉₅	Weighted bed size	d _{bm}
		Standard deviation of % of sample in each size range	S _x

TABLE 3
Differences of some channel site characteristics between the two sets collected at each stream gaging station

Characteristics	Average difference ^{1/} (percent)	Standard Deviation of the difference (percent)
Cross-sectional area	15.3	10.5
Mean depth	12.9	12.0
Hydraulic radius	12.2	11.5
d ₉₅ Size	17.9	14.6
Weighted bank size	23.0	16.4
Weighted bed size	16.6	23.8

^{1/} Based on the ratio of the difference between the values divided by the larger value.

TABLE 4
Fitted Values of the Coefficients in the Lacey Equations

Shape Characteristics Fitted	Flow and Particle Size Combination	Coefficient Values								R	SEE	% Error
		K ₁	K ₂	K ₃	K ₄	K ₅	K ₆	K ₇	K ₈			
Cross-sectional area	P ₂ d ₉₅						0.024	5.50	-0.85	0.87	1286	48
	P ₂ d ₅₀						0.015	5.26	-0.84	0.87	1291	48
	P ₂ d _m						0.016	5.30	-0.84	0.87	1281	48
	P ₂ d _{bm}						0.119	6.89	-0.027	0.87	1303	46
	P ₁₀ d ₉₅						0.002	5.10	-0.83	0.83	1470	72
Hydraulic radius	P ₂ d ₉₅			3.37	0.25	0.51	0.065	6.08	-1.02	0.49	4.50	48
	P ₂ d ₅₀			3.43	0.25	0.51	0.072	6.20	-1.03	0.54	4.36	47
	P ₂ d _m			3.42	0.25	0.51	0.071	6.18	-1.03	0.53	4.38	47
	P ₂ d _{bm}			3.16	0.22	0.48	0.045	5.60	-1.21	0.33	4.87	52
	P ₁₀ d ₉₅			3.47	0.24	0.52	0.077	6.21	-1.06	0.33	4.86	53
Channel slope	P ₂ d ₉₅	0.036	0.082	3.54	0.24	0.50	0.14	3.69	-3.18	0.58	0.00040	50
	P ₂ d ₅₀	0.035	0.087	3.48	0.24	0.48	0.11	3.02	-3.42	0.54	0.00041	50
	P ₂ d _m	0.035	0.087	3.48	0.24	0.48	0.11	3.01	-3.12	0.54	0.00041	50
	P ₂ d _{bm}	0.031	0.00018	3.17	0.18	0.45	0.0073	4.76	-2.63	0.48	0.00043	53
	P ₁₀ d ₉₅	0.042	0.040	3.92	0.21	0.50	0.15	4.10	-1.78	0.55	0.00041	49
Lacey's Values		0.033	0.10	3.31	0.25	0.50	0.059	6.0	-1.0			

TABLE 5
Values of F_B and F_S

	Developmental Data		Test Data	
	F_B	F_S	F_B	F_S
Mean	5.07	1.57	2.65	1.08
Range	0.31 - 29.0	0.039 - 7.4	0.19 - 5.2	0.030 - 4.0
Standard Deviation	5.00	1.68	1.67	1.14

TABLE 6
Fitting of F_B and F_S to Site Characteristics^{1/}

Factor	R^2	SEE	Site Characteristic					Test data SEE
			a_w	d_{95}	d_{50}	%0.25	%0.0064	
F_B	0.53	4.36	0.13	0.31	—	-0.31	-0.15	5.21
F_S	0.25	1.56	0.21	-0.91	1.37	0.07	0.62	2.60

^{1/} Values presented are for standardized variates.

TABLE 7
Values of K_1 , K_2 , and K_4 in Simons Regime Equations^{1/}

Types of Material	K_1	K_2	K_4
Sand bed and banks	3.5	0.52	0.33
Sand bed and cohesive banks	2.6	0.44	0.54
Cohesive bed and banks	2.2	0.37	0.87
Coarse noncohesive material	1.75	0.23	—
Sand bed and cohesive banks with heavy sediment load	1.70	0.34	—
Average values - Oklahoma data	2.33	0.35	0.21

^{1/} Taken from Table 10.2, p. 252, Graf (1971).

TABLE 8
Values of K_1 , K_2 , and K_4

	Developmental Data			Test Data		
	K_1	K_2	K_4	K_1	K_2	K_4
Mean	2.33	0.35	0.21	2.51	0.39	0.18
Range	1.2-5.4	0.10-0.63	0.023-0.64	1.3-6.6	0.14-0.72	0.022-0.58
Standard Deviation	1.06	0.15	0.16	1.40	0.16	0.16

TABLE 9
Components Regression of K_1 , K_2 , and K_4 on Site Characteristics

Factor	R ²	SEE	Site Characteristic								Test data SEE
			a _w	m	pH	11	d ₉₅	X _{0.25}	X _{0.0064}	d _m	
K ₁	0.38	0.90		-0.17	0.14		0.16	-0.15	-0.15		0.92
K ₂	0.46	0.12	-0.18		-0.07	0.13	-0.24	0.24	0.04		0.14
K ₄	0.16	0.15	0.05				0.13	-0.13		0.13	0.19

TABLE 10
Fitted Values of Coefficients in the Tractive Force Equations^{1/}

Shape Characteristic Fitted	Flow and particle size combination		Coefficient Values		R	SEE	% Error
			j_1	j_2			
Cross-sectional Area	P ₂	d ₉₅	0.017	0.122	0.84	1404	42
	P ₂	d ₅₀	0.022	0.230	0.85	1363	43
	P ₂	d _m	0.006	-0.0012	0.86	1325	41
	P ₂	d _{bm}	0.115	0.535	0.83	1435	46
	P ₁₀	d _{bm}	0.051	0.581	0.79	1578	67
Mean depth	P ₂	d ₉₅	0.011	0.030	0.57	4.34	35
	P ₂	d ₅₀	0.047	0.361	0.54	4.42	37
	P ₂	d _m	0.018	0.193	0.56	4.36	35
	P ₂	d _{bm}	0.090	0.416	0.56	4.36	39
	P ₁₀	d _{bm}	0.041	0.511	N ^{2/}	5.46	53
Channel slope	P ₂	d ₉₅	2.20	1.63	0.33	0.00043	42
	P ₂	d ₅₀	1.62	1.58	0.33	0.00043	42
	P ₂	d _m	1.69	1.59	0.33	0.00043	42
	P ₂	d _{bm}	1.48	1.49	0.38	0.00042	44
	P ₁₀	d _{bm}	0.73	1.59	N ^{2/}	0.00050	49

^{1/} Based on average cross section from 39 sets of data

^{2/} Correlation not significant

TABLE 11
Relation between Manning's n and Particle Size, in inches

Investigator	Particle Size	j_1	j_2
Strickler formula	median bed	0.032	0.17
Lacey	median bed	0.033	0.10
Keulegan (1938)	median bed	0.021	0.17
Lane and Carlson (1953)	d ₉₅ bed	0.033	0.17
Irmay (1949)	d ₉₀ bed	0.020	0.17
Tractive Force Fitting to			
Area	median bank	0.022	0.23
Mean depth	d ₉₅ bank	0.011	0.030
Slope	median bed	1.48	1.49

TABLE 12
Comparison of Regime, Tractive Force and Statistical Evaluations

Shape Characteristic Fitted	Correlation Coefficients			Percent Error		
	Regime	Tractive Force	Statistical	Regime	Tractive Force	Statistical
Area	0.87	0.85	0.89	48	43	34
Mean depth or Hydraulic Radius	0.53	0.57	0.87	47	35	19
Slope	0.54	0.38		50	44	

NOTE: Lack of continuity between the correlation coefficients and the percent error is caused by the fact that optimization is not on the percent error but upon squared error deviations.

OBSERVED CHANNEL CHANGES IN A MOUNTAIN STREAM
DUE TO INCREASED FLOW FROM TRANSBASIN IMPORTS

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Abstract

In anticipation of increased flow in the channel of Lake Fork due to two recently completed transbasin diversions, data was collected on the flow, channel geometry and bed composition of that channel for the pre-development period. During early stages of the new diversions the effect of the increased flow on the channel was observed. After a period of years, during which these increased flows will be carried in the Lake Fork Channel, data will be collected which should throw additional light on the processes involved in mountain stream channel adjustment.

Introduction

Lake Fork, a headwater stream of the Arkansas River, lies entirely in Lake County, Colorado. The stream rises on the east slope of the Continental Divide, flows eastward 8.5 miles in a glaciated valley, emerges onto the depositional floor of the Upper Arkansas River Basin, thence flows southward 5 miles to its confluence with the Arkansas River.

The upper half of the Lake Fork Valley shows abundant evidence of alpine glaciation. The valley is typically U-shaped, as is that of Busk Creek, the principal tributary. Busk Creek occupies a hanging valley having a post glacial notch at the lower end. Hagermann and Virginia Lakes are cirque lakes at the head of Lake Fork tributaries. At the point where the stream issues into the Arkansas River Valley is a textbook example of a terminal moraine. (Scoval et al, 1965) Turquoise Lake, enlarged by the construction of Sugar Loaf Dam, occupies the basin behind this moraine.

The watershed boundary is at the crest of the Continental Divide with elevations ranging between 11,700 and 12,500 feet above mean sea level. From headwaters in cirques near elevation 11,500 to about elevation 10,000, Lake Fork flows through an erosional channel. Below that, to the headwaters of the lake, the stream channel is on a depositional surface.

It is the reach of Lake Fork just above the headwaters of Turquoise Lake and below the outlet of the Homestake Tunnel, elevation 9915, that is here under consideration. Though the maximum water surface of Turquoise Lake will reach almost to the confluence, a short reach of channel below the Busk Creek confluence was included in the study reach. Drainage area above the confluence of Busk Creek is 10.9 square miles. The drainage area above a point just downstream from the Busk Creek confluence is 17.5 square miles.

Transmountain Diversions

Busk-Ivanhoe

Prior to 1925 all flow in the study reach of Lake Fork was native. During June 1925 the Busk-Ivanhoe system, now owned jointly by the Highline Canal Co. and the City of Pueblo, began importing water from a 6.9 square mile collection system in the Ivanhoe Creek Basin into Busk Creek in the Lake Fork Basin. Imports are made through the Busk-Ivanhoe (Carlton) Tunnel.

An average of 4,600 acre-feet has been imported annually into Lake Fork through Busk-Ivanhoe Tunnel. The maximum imported has been 125 c.f.s. During the time Boustead Tunnel has been diverting, Busk-Ivanhoe diversion averaged 48 c.f.s.

Homestake Tunnel

The Homestake Project is a transmountain water diversion development, sponsored by the cities of Aurora and Colorado Springs, to provide water for beneficial use by those two eastern slope cities. That part of the Homestake western slope collection system that contributes to the diversion into Lake Fork through Homestake Tunnel diverts the waters of Sopris, Missouri, Fancy, French, and Homestake Creeks in the Eagle River Basin.

The water is regulated by the 45,000 acre-foot Homestake Reservoir at the western end of the 5.5-mile-long, 9-foot-8-inch horseshoe-shaped Homestake Tunnel. (Black and Veatch et al, 1974)

It is planned that an average of 63,000 acre-feet of water will be transported through Homestake Tunnel annually. An agreement with the Forest Service has limited diversion to a maximum of 300 c.f.s.

The maximum mean daily diversion was 250 c.f.s. The maximum instantaneous discharge was 308 c.f.s. During the time Boustead Tunnel was diverting the Homestake Tunnel diversion averaged 55 c.f.s. with the exception of 1972.

Charles H. Boustead Tunnel

Boustead Tunnel is the primary feature of the Fryingpan-Arkansas Project's western slope collection system. When completed, the Fryingpan-Arkansas Project will tap the headwaters of the Fryingpan River and Hunter Creek in the Roaring Fork Basin and divert an annual average of 72,000 acre-feet through Boustead Tunnel. The completed collection system will divert water from 16 diversion points. There will be no storage at the elevation of the collection system. To date (1975) only five diversion points are in operation. These five points are capable of supplying about one-half of the design annual diversion.

The Charles H. Boustead Tunnel is a 5.4-mile, 10.5-foot horseshoe-shaped tunnel with a design capacity of 945 c.f.s.

The largest mean daily diversion was 555 c.f.s. and had an instantaneous peak of 711 c.f.s. This occurred on June 6, 1972.

Lake Fork Channel Flow

Historic

Historic Lake Fork flow, as here used, is the native flow of the stream down to the confluence with Busk Creek. Below Busk Creek, historic flow also includes historic Busk-Ivanhoe transmountain diversion. Shown on Figure 1 are flow duration curves of these two reaches under historic conditions.

Annual flood frequencies for the reach of Lake Fork above Busk Creek under native conditions and for the reach just below Busk Creek under historic conditions are as follows:

<u>Percent of Time</u>	<u>Above Busk Creek (native-c.f.s.)</u>	<u>Below Busk Creek (historic-c.f.s.)</u>
99	71	134
95	125	203
80	198	295
50	286	407
20	390	538
10	451	616
5	504	683
4	522	705
2	570	767
1	616	824
0.1	758	1,004
0.01	893	1,176

Assuming the flows that dictated the shape of the channel to be those flows at or slightly exceeding bankfull stage (Lara, ND) and assuming these flows have the same frequency as the median annual flood, i.e. the annual flood that is exceeded every second year (Blench, 1957), the native dominant discharge above Busk Creek was 286 c.f.s. and the historic dominant discharge in the reach just below Busk Creek was 407 c.f.s.

With-Projects

Four separate reaches can be defined for with-projects Lake Fork flow above Turquoise Lake. Within these, three short reaches were chosen as representative of the channel for purposes of study. Above the confluence

with the Homestake Tunnel outlet channel, the with-projects flow is the same as native flow. Study Reach A is in this segment.

Below Homestake Tunnel outlet and above the Charles H. Boustead Tunnel outlet channel with-projects flow is native flow plus Homestake Tunnel imports. Flow in this reach will be increased by a maximum of 300 c.f.s. giving a bankfull channel capacity of 586 c.f.s. after several years of project diversions. Study Reach B is in this segment.

The flow in the reach below the Boustead Tunnel outlet will be increased by 1,245 c.f.s. during peak import periods. Though it is doubtful that these maximum imports will be sustained for long periods of time, especially in the case of Boustead Tunnel which has no associated storage on the western slope, it is likely that peak diversions through the Boustead Tunnel will correspond temporarily with native peaks in Lake Fork. In the short reach downstream from the Boustead Tunnel outlet and above the mouth of Busk Creek, with-projects dominant discharge will be 1,531 c.f.s.

Below Busk Creek the with-projects bankfull flow will be the sum of all the imports plus historic flow or 1,652 c.f.s. Study Reach C is located below Busk Creek.

The above assumes that Homestake will be importing 300 c.f.s. during the peak runoff period though there is a possibility that Homestake imports will be regulated to avoid high Lake Fork flows from other sources. Also, there is a possibility that, sometime in the future, Busk-Ivanhoe diversions through Carlton Tunnel will be discontinued and these diversions will be made through Boustead Tunnel.

Figure 2 is a plan and profile of Lake Fork Channel showing the study reaches.

Description of the Study Reach of Lake Fork Channel

Channel Morphology

Lake Fork Channel in the study reach sloped overall 0.016. In the 900-foot reach measured above the Homestake Tunnel outlet, the slope is 0.020, between Homestake and Charles H. Boustead Tunnel outlets the slope was 0.018, between Boustead Tunnel outlet and Busk Creek the slope was 0.012. In the reach just downstream from Busk Creek the slope was 0.011.

Prior to the beginning of the Homestake and Fryingpan-Arkansas Project diversions, channel width varied from 9 to 41 feet through the reach. Bed width averaged 17 feet in the reach.

Manning friction coefficients computed from streamflow measurements and simultaneous water surface profile measurements were $n = 0.052$ in Reach A, 90 c.f.s., mean velocity 3.62 ft/sec; $n = 0.042$ in Reach B, 173 c.f.s., mean velocity 5.84 ft/sec; and $n = 0.035$ in Reach C, 219 c.f.s., mean velocity 4.02 ft/sec.

Water surface profile computations for three short reaches computed for dominant discharge under native and historic conditions and near dominant discharges for with-project conditions gave the following results:

<u>Station</u>	<u>Thalweg Elevation</u>	<u>Native Dominant Discharge</u>		<u>With-Projects Dominant Discharge</u>	
		<u>Water Surface Elevation</u>	<u>Channel Velocity</u>	<u>Water Surface Elevation</u>	<u>Channel Velocity</u>
Reach A:		<u>286 c.f.s.</u>		<u>Same</u>	
17+51	9918.6	9921.71	5.59		
16+86	9919.5	9923.28	2.21		
15+34	9921.8	9923.98	3.46		
14+21	9923.5	9926.02	6.60*		
12+95	9926.4	9928.86	5.97		
Reach B:		<u>286 c.f.s.</u>		<u>586 c.f.s.</u>	
28+00	9898.1	9900.58	5.32*	9900.88	7.18*
27+00	9900.3	9902.42	7.81*	9904.02	7.08*
25+94	9901.4	9904.86	4.36	9905.30	7.08
24+42	9903.9	9906.72	7.27	9907.99	7.74*
22+89	9906.4	9910.21	6.13*	9911.00	7.09*
20+66	9912.1	9914.41	6.72*	9915.42	7.23*
Reach C:		<u>407 c.f.s. (historic)</u>		<u>1,652 c.f.s.</u>	
78+84	9834.2	9836.79	3.98	9839.66	6.45
78+26	9833.0	9837.02	3.55	9840.03	5.36
77+56	9834.6	9837.22	4.61	9840.19	6.55
77+00	9834.9	9837.55	3.77	9840.59	4.84

* denotes critical conditions.

Bed Material

Bed material was measured by two methods at three different places in the reach under study. One method of measurement was to scale the intermediate axis of all the surface stones in a typical square yard of streambed surface. (Kellerhalls, 1966; Lane and Carlson, 1953) Another method of measurement was to lay out a grid, in this case, selecting stone under each foot mark of a 100-foot tape stretched out along a typical span of channel bottom, and measuring the stone's intermediate axis. (Kellerhalls, 1966; Wolman, 1954)

The following are the results of the two sampling measurements:

	Size in mm	
	<u>D50</u>	<u>D90</u>
Reach above Homestake outlet (A)		
Tape	168 (6.61")	310 (12.20")
Sq. Yd.	160 (6.30")	260 (10.24")
Reach between Homestake and Boustead Tunnels (B)		
Tape	158 (6.22")	282 (11.10")
Sq. Yd.	133 (5.23")	227 (8.94")
Reach below Busk Creek (C)		
Tape	85 (3.34")	128 (5.04")
Sq. Yd.	62 (2.44")	103 (4.06")

D50 is the screen size (intermediate diameter) passing 50 percent of the sample. D90 is the screen size passing 90 percent of the sample.

Figure 3 is the grain size analysis averaged for the two methods for historic conditions at each of the three measured reaches.

Valley Floor

Historically in the study reach, the flood plain was covered with grass, with occasional evergreen trees or a peat type material with grass and three-foot willows. There was abundant evidence that the peat type deposits resulted from beaver activity. Beaver ponds, both active and abandoned, were found in the upper and middle stretches of the reach. Some of the abandoned ponds were silted full. Beaver runs criss-crossed the peat covering. Test pits indicate that the earth or peat layer extended to a maximum of about three feet and was underlain with sand, gravel, cobbles, and in the upper end of the reach, boulders. Near Lake Fork Channel, directly in front of the Boustead outlet channel, test holes were drilled to a depth of 70 feet without encountering bedrock.

Based on the data listed above, computations were made that indicated the following predicted changes in the channel. An increase in the roughness factor due to winnowing of the finer bed material leaving an armoring layer composed of coarser material. Channel degradation was found to be minimal. Adjustment was assumed to be made predominantly by channel widening. Computations on which these predictions are based are not shown as observations of the results of early diversion have proven them moot.

Channel Monitoring Program

A program to monitor the effect of the increased flow on the channel of Lake Fork was put into effect at the time of the initial large diversion by each tunnel. Homestake Tunnel discharges into Lake Fork were observed for selected periods during the last part of April and in May 1968. During this time flow in Lake Fork was often over 200 c.f.s. and for one two-hour period was 300 c.f.s. The latter high tunnel discharge flow

exceeded channel capacity of Lake Fork. Some bank undercutting was observed. Ponding was observed on the right overbank area behind the approach to the construction road bridge crossing Lake Fork at this point. Opportunistic beavers have filled the gaps where the flow would have normally returned to the stream channel. Backwater from the resulting beaver pond channels most of the Lake Fork flow down its original route. Resurvey of selected cross sections in the reach downstream from Homestake outlet after the initial period of tunnel release revealed remarkably little change in channel cross section.

The initial imports through the Charles H. Boustead Tunnel were begun May 16, 1972. From that date until July 20, the last day of imports in the 1972 season, 31,957 acre-feet were imported from the Fryingpan Basin. During this period of imports the effects of the increased flow on the channel of Lake Fork were periodically monitored. Imports during the period ranged from less than 100 c.f.s. to a mean daily maximum of 555 c.f.s. Flow was over 500 c.f.s. for three days, over 400 c.f.s. for 14 days, and over 300 c.f.s. for 24 days during this period. The diurnal peak during this period was 711 c.f.s.

The most notable channel change resulting from this period of imports was the cutting through of a meander neck immediately downstream from the confluence of Busk Creek. Associated with this channel shortening was a certain amount of entrenchment of the Lake Fork bed. Other observed changes are now under the headwaters of Turquoise Lake. They include denudation of the flood plain downstream from the two tunnel muck piles, a scour hole formed at the base of the Homestake muck pile, and channel widening near the Bear Creek Campground area. The channel in this reach just above Bear Creek Campground was not in the position it had occupied in 1963 (Figure 4), it having been displaced somewhat by tunnel spoil.

Because water, as an architect of landforms, is in tune with the process of nature, the resulting channel is aesthetically adapted to the surrounding landscape.

Observations During 1975 Season

Though high flow in Lake Fork Channel made a planned channel resurvey impossible, general observations were made.

A section was measured in Reach B which was found to have widened by approximately 3 feet. The bed had degraded approximately 0.5 feet. Computations based on mean velocity in the measured section indicated a Manning roughness of $n = .049$ as opposed to $n = 0.042$ measured earlier.

A section was measured between the Boustead Tunnel outlet and the confluence of Busk Creek. It was found that the channel had widened approximately 7 feet and the bottom had been degraded 2.5 feet. Computed Manning roughness factor in the reach is $n = .062$. The channel in the short reach seems to have taken on the characteristics of the Charles H. Boustead Tunnel outlet channel.

Downstream from the cut-off meander just below Busk Creek the channel has migrated back and forth across the flood plain between the two spoil piles denuding the entire area.

An anabranch leading from the right of the main channel just downstream from the contractor's bridge is enlarging to carry a larger share of the flow. Signs of beaver activity are abundant on the left overbank between the contractor's bridge and the Busk Creek confluence. Channel straightening is occurring at several bends in this reach. Figure 5 is an aerial view of Lake Fork Channel taken in July 1975.

Anticipated Channel Changes

The following measurable adjustments are anticipated as a result of increased flow in the study reach of Lake Fork Channel: (1) channel widening, (2) some increase in channel slope as a result of foreshortening of meanders, (3) some increase in channel depth due to change in channel slope, (4) coarsening of bed material size with accompanying change in bed roughness.

Sufficient data has been collected on the pre-transbasin diversion condition of Lake Fork Channel and this, coupled with similar measurements after a number of years of diversion and with concurrent observations, will throw additional light on the processes in action due to increased flow in a mountain stream.

Acknowledgements

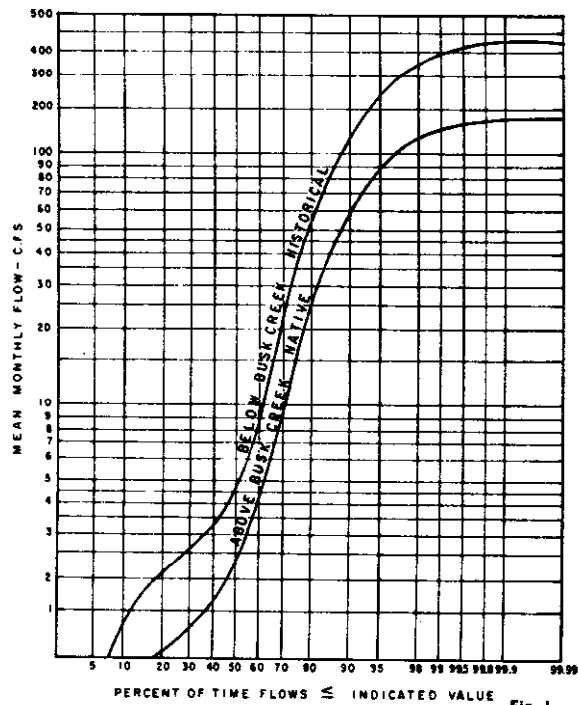
The author wishes to acknowledge the following for assistance in various phases of the studies: W. M. Borland, E. L. Pemberton, J. M. Lara, and R. K. Kuwabara; the staff of the Pueblo Project Office, Fryingpan-Arkansas Project, assisted in the compilation and presentation of data; certain field measurements were made by the staff of the Salida Construction Office, Fryingpan-Arkansas Project; and Bectel Corporation, consultants for the Homestake Project. The Fryingpan-Arkansas Project is a Bureau of Reclamation project.

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MEAN MONTHLY FLOW DURATION CURVES LAKE FORK ABOVE TURQUOISE LAKE

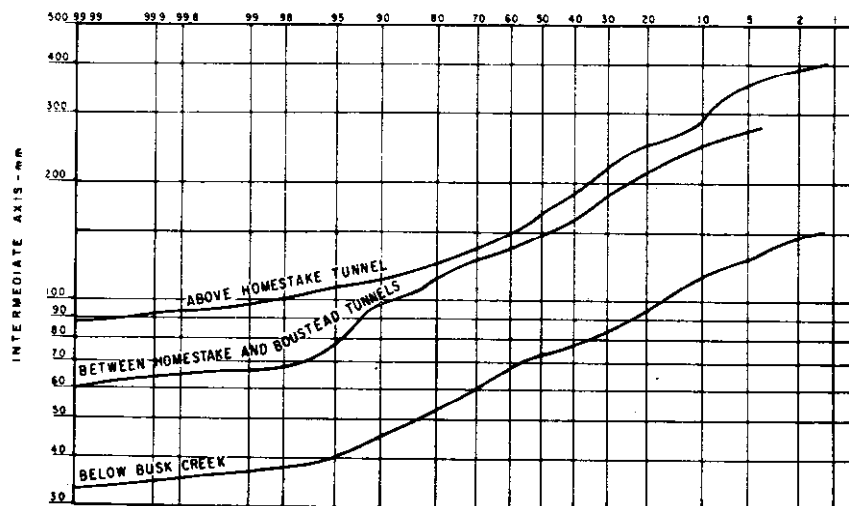
W.Y. 1947 - 1965



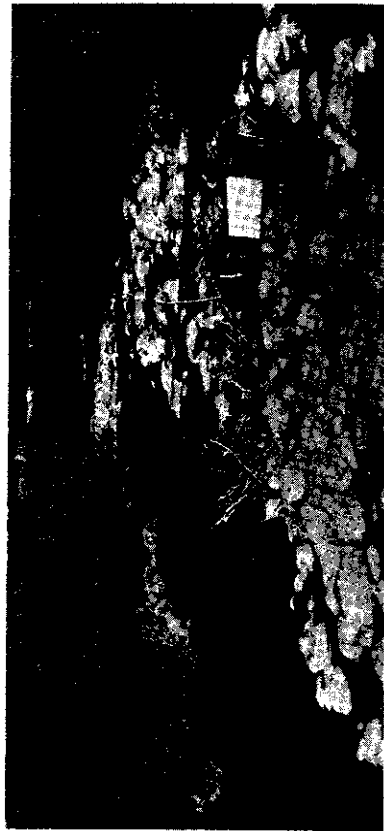
BED MATERIAL SIZE ANALYSIS LAKE FORK ABOVE TURQUOISE LAKE

1966

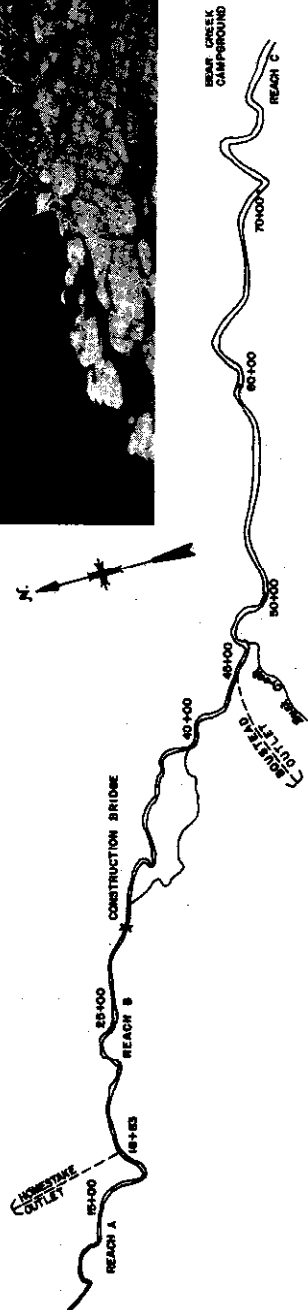
PERCENT BY COUNT EQUAL TO OR GREATER THAN INDICATED SIZE



REACH B



CHANNEL AS OF 9-30-86



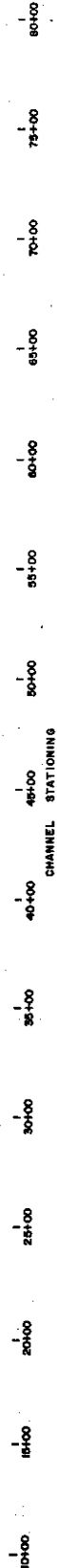
THALWEG PROFILE

TOP CONSERVATION TURQUOISE LAKE

9900—

9850—

ELEVATION



CHANNEL STATIONING

REACH C



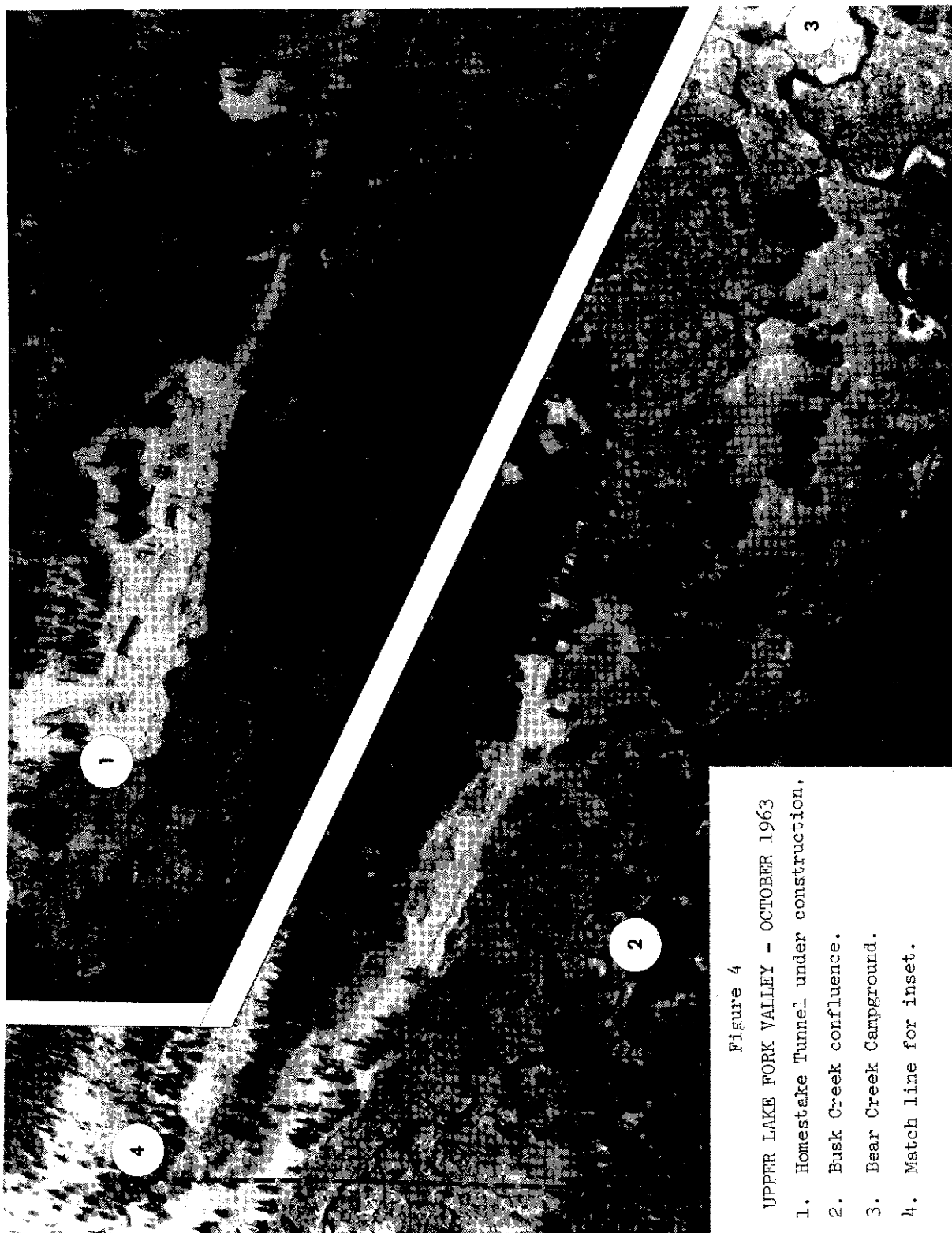


Figure 4
UPPER LAKE FORK VALLEY - OCTOBER 1963
1. Homestake Tunnel under construction.
2. Busk Creek confluence.
3. Bear Creek Campground.
4. Match line for inset.

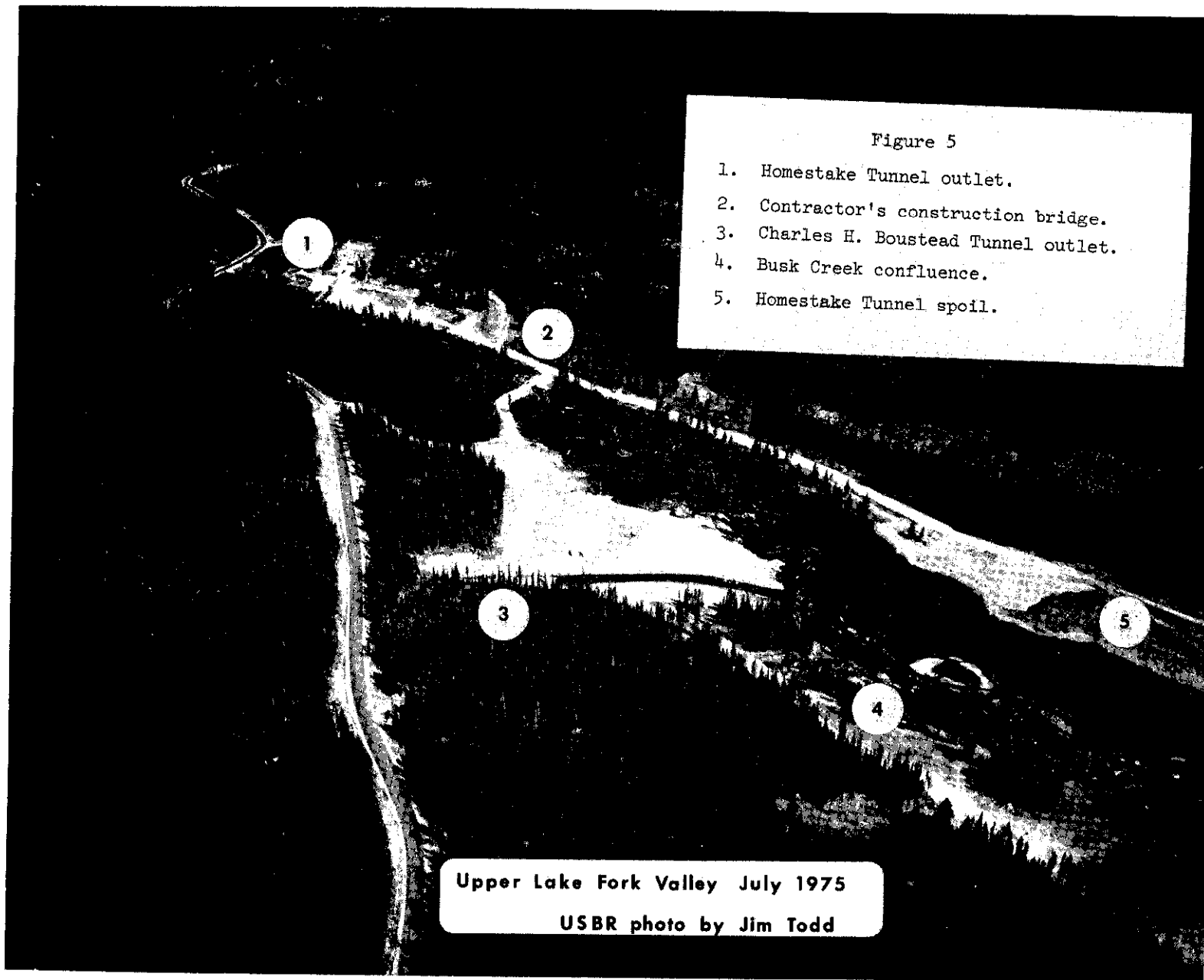


Figure 5

1. Homestake Tunnel outlet.
2. Contractor's construction bridge.
3. Charles H. Boustead Tunnel outlet.
4. Busk Creek confluence.
5. Homestake Tunnel spoil.

Upper Lake Fork Valley July 1975

USBR photo by Jim Todd

APPLICABILITY OF THE UNIT STREAM POWER EQUATION

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ABSTRACT

The usefulness of a sediment transport equation to engineering depends on its applicability to natural rivers. Data collected from six river stations are used to compare the applicability of different equations. The comparisons indicate that the dimensionless unit stream power equation is the one which can provide accurate predictions of total sediment discharge in natural rivers under diversified flow and sediment conditions. The limitations to which this dimensionless unit stream power equation can be applied are explained.

INTRODUCTION

Numerous equations have been proposed by different authors for the calculation of the rate of sediment transport in alluvial channels. Most of these equations were derived under the assumption that the rate of sediment transport depends mainly on one independent variable, such as water discharge, average flow velocity, energy slope, or shear stress. Analysis of existing data (Yang, 1972) raises doubts as to the reliability of these assumptions. A thorough analysis of available data (Yang, 1972) indicates that the rate of sediment transport in an alluvial channel is dominated by the rate of potential energy expenditure per unit weight of water, i.e., the unit stream power. A dimensionless unit stream power equation was proposed by Yang (1973) for the calculation of the rate of sediment transport.

Most sediment transport equations are verified by data collected from laboratory flumes. The usefulness of these equations depends on their applicability to natural rivers. Comparisons between the computed results from different equations and the measured results from natural rivers are made to determine the applicability of these equations. The limitations to which the dimensionless unit stream power equation can be applied are explained in this paper.

THE UNIT STREAM POWER EQUATION

The amount of sediment transported by water in an alluvial channel can be calculated by

$$\log C_t = 5.435 - 0.286 \log \frac{\omega d}{v} - 0.457 \log \frac{U_*}{\omega} \\ + \left(1.799 - 0.409 \log \frac{\omega d}{v} - 0.314 \log \frac{U_*}{\omega} \right) \log \left(\frac{VS}{\omega} - \frac{V_{cr} S}{\omega} \right) \quad (1)$$

in which C_t = total sediment concentration in parts per million by weight; ω = terminal fall velocity of sediment particles; d = median sieve diameter of bed material; ν = kinematic viscosity; U_* = shear velocity; VS = unit stream power; and $V_{cr}S$ = critical unit stream power at incipient motion. The dimensionless critical unit stream power in Eq. 1 can be determined by the product of dimensionless critical average velocity V_{cr}/ω and energy slope S . The V_{cr}/ω value can be computed by

$$\frac{V_{cr}}{\omega} = \frac{2.5}{\log \frac{U_* d}{\nu} - 0.06} + 0.66; 1.2 < \frac{U_* d}{\nu} < 70 \quad (2)$$

and

$$\frac{V_{cr}}{\omega} = 2.05; 70 \leq \frac{U_* d}{\nu} \quad (3)$$

The lower boundary of 1.2 in Eq. 2 is used to avoid the situation of $V_{cr}/\omega \rightarrow \infty$. The basic forms of Eqs. 2 and 3 were derived from the balance of forces acting on a spherical sediment particle at the bottom of an open channel flow. The relative roughness was used in the derivation of the basic forms of Eqs. 2 and 3. Data collected by individual investigators indicate that there is a general tendency for V_{cr}/ω to increase with decreasing relative roughness d/D in which D is the average water depth. However, there is no consistent pattern between V_{cr}/ω and d/D when data collected by different investigators are compared. The coefficients in Eqs. 2 and 3 were determined from laboratory data independently collected by eight investigators without taking the relative roughness d/D into consideration.

The basic form of Eq. 1 was determined from the concept of unit stream power (Yang, 1972) and dimensional analysis (Yang, 1973). The coefficients in Eq. 1 were determined from multiple regression analyses of 463 sets of laboratory data independently collected by different investigators (Yang, 1973). Eq. 1 can be used for the calculation and prediction of total sediment concentration in the sand size range with a specific gravity of 2.65. Since the coefficients of Eq. 1 were determined from data collected in uniform flows with particle sizes between 0.137 mm and 1.71 mm, shape factor of 0.7, and total sediment concentration greater than 10 ppm, this equation is most reliable in predicting total sediment concentration under similar conditions. It is assumed that the energy slope can be approximated by the water surface slope. The parameters required for Eq. 1 are average flow velocity, water surface or energy slope, median bed material particle size, water temperature, and water depth. The relation between particle size, shape factor, water temperature, and fall velocity, can be found in Fig. 6 of Report 12 of Inter-Agency Committee on Water Resources (1965). Wash load should be excluded from total load when the former is significant.

Fig. 1 shows an example of the comparison between the predicted total sediment concentration and that measured from a laboratory flume. A water temperature of 20°C is assumed for Gilbert's (1914) data. Since Gilbert's data are not included in the derivation of Eq. 1, Fig. 1 provide an independent verification of the accuracy of the equation. Fig. 1 also indicates that Eq. 1 can be used for the prediction of total sediment concentration in spite of the existence of different bed forms, such as smooth bed, dune, transition, and antidune.

SEDIMENT TRANSPORT IN NATURAL RIVERS

The usefulness of a sediment transport equation to engineering depends on its applicability to natural rivers. Sediment concentrations in natural rivers have been measured by different agencies and individuals. Among the published data, only a few have velocity, slope, depth, bed material size, water temperature, and composition of suspended material measured simultaneously. Data collected from stations that measure the data simultaneously were used quite often to determine the accuracy and applicability of a sediment transport equation. Comparison between the measured and computed results of the unit stream power equation is limited to cases where hydraulic and sediment data were collected within a 24-hr period to insure that the unit stream power and its corresponding sediment concentration were measured under approximately the same conditions. These comparisons include total sediment discharge measured at three locations and bed material discharge at another three locations.

The ASCE Task Committee On Sedimentation (1971) made an analysis of the data collected from Niobrara River near Cody, Nebraska (Colby and Hembree, 1955). Fig. 2 shows the calculated results from different equations by the ASCE Task Committee (1971) as well as the measured total sediment discharges at different water discharges. Among the 13 different equations used by the ASCE Task Committee, only Colby's, Laursen's, Tofaletti's, and Einstein's bedload function can provide reasonable estimations of the total sediment discharge in the Niobrara River. When the total sediment concentrations calculated with Eq. 1 were multiplied by their corresponding water discharges and then plotted in Fig. 2, they agreed very well with the measurements. Because bed material size was not measured for each set of data, an average median size of 0.283 mm was used for the calculations.

Einstein (1944) measured the total sediment discharge from Mountain Creek at Greenville, South Carolina. Vanoni et al. (1960) computed the total sediment discharge by using 7 different equations and comparing them with the measured results from Mountain Creek. The data shown in Fig. 3 included all measurements except those taken during the falling stages on September 3 and October 22, 1941. According to Einstein (1944), during these two falling stages, the flow was unable to effect the same readjustment of the bed surface as under natural conditions. Fig. 3 shows that the unit stream power equation is the only equation that can provide a close estimation of the total sediment discharge in Mountain Creek. An average median size of 1.0 mm was used in the unit stream power equation for the calculation of total sediment discharge. The Schoklistsch equation ranks second in accuracy.

The total sediment discharge in the Middle Loup River at Dunning, Nebraska, was measured by Hubbell and Matejka (1959). They computed the total sediment discharge with 5 different equations. The actual median bed material with a size range from 0.16 mm to 0.24 mm for each individual set of data was used when the calculation was done with the unit stream power equation. Fig. 4 shows the comparison between the measured and computed results of 6 different equations. Only the unit stream power equation and the modified Einstein equation can provide good estimates of the total sediment discharge in the Middle Loup River (as shown between the two dashed lines in Fig. 4). Other equations cannot even indicate the general trend of variation of total sediment discharge at different water discharges.

The total sediment concentrations of the three cases mentioned above were measured either at a contracted section or at a section with manmade construction such that all the sediment in transport could be measured. Under ordinary conditions, only the suspended sediment concentration can be measured in a natural river. The sediment transported in suspension includes those with particle sizes within the range of the channel bed composition, and those sediments are usually finer in size. Wash load is defined as that part of the sediment load that consists of grain sizes finer than those of the bed (1950). Bed material discharge equals the product of water discharge and the difference between total suspended concentration and the suspended concentration with particle size in the range of wash load. The bed material discharge includes all bed material discharge measured in the sampled zone and part of the bed material discharge in the unsampled zone. It is considered that the total sediment discharge, excluding the wash load, can be represented best by this measured bed material discharge. Comparisons were made between calculated total sediment discharges and measured bed material discharges at three locations where wash loads are significant.

The average bed material size distribution from the Mississippi River at St. Louis, Missouri, and the Rio Grande River near Bernalillo, New Mexico are shown in Fig. 5. An average of only 0.05 percent of the bed material from these two locations is finer than 0.125 mm. Thus, the bed material discharge and the proportions of suspended concentration with particle sizes coarser than 0.125 mm.

Comparison between total sediment discharge computed by the unit stream power equation and the measured bed material discharge from the Mississippi River at St. Louis, Missouri, is shown in Fig. 6. The actual median bed material with a size range from 0.21 mm to 0.78 mm for each individual set of data was used in the computation. Fig. 6 also shows the computed sediment discharge made by Jordan (1965) with other well-known equations. Among the five equations shown in Fig. 6, the unit stream power equation is the most accurate one.

The bed material discharge in the Rio Grande River near Bernalillo, New Mexico, was measured by Nordin (1964) at two different locations. Channel width varies from 267 ft to 273 ft for Section A-2 and 335 ft to 645 ft for Section F. The median bed material size varies from 0.19 mm to 0.45 mm at both locations. Three equations are used to compare the computed total sediment discharge and the measured bed material discharge from these two locations. Fig. 7 shows that the computed results from the unit stream power equation and the Laursen equation compare favorably with the measurements. On the average, the computed results of the modified Einstein method are three times as large as measured bed material discharge.

SUMMARY AND CONCLUSIONS

A total of 156 sets of data collected from five natural rivers were used to compare the applicability of different sediment transport equations. With the exception of the unit stream power equation, the computed results were obtained by other authors. Comparisons between the computed and measured results indicate that most published equations cannot be applied with confidence to natural rivers. Some equations can provide reasonable estimations of the rate of sediment transport in one river but not in others. Fig. 8 summarizes the comparisons between the computed results from unit stream power equation and the measured results from five natural rivers. These 156 sets of field data cover diversified conditions with the median bed material particle size ranging from 0.16 mm to 1.0 mm; the relative roughness from 0.000022 to 0.0164; the channel width from 11.1 ft (3.38 m) to 1746 ft (523.18 m); the channel depth from 0.2 ft (0.06 m) to 49.9 ft (15.21 m); the water temperature from 0°C to 29.4°C; the average water velocity from 1.23 fps (0.375 mps) to 7.82 fps (2.38 mps); the water discharge from 2.73 cfs (77.31 lps) to 467900 cfs (13250928.0 lps); the slope from 0.0000428 to 0.00188; and the total sediment concentration (wash load excluded) from 2.8 ppm to 2440 ppm by weight. Fig. 8 indicates that the computed results from the unit stream power equation agree very well with measurements from such diversified flow and sediment conditions. The application of the unit stream power equation, (Eq. 1), is simple and straightforward. This equation is recommended by the authors for engineers to use in estimating and predicting the rate of sediment transport in natural rivers as well as in laboratory flumes.

ACKNOWLEDGEMENT

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CONVERSION FACTORS

The following conversion factors can be used to convert the results given in this paper from the English System to the International System:

1 cfs = 0.02832 m³/s; 1 lb/sec = 0.4536 kg/s, and 1 ton/day = 907.18 kg/day.

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NOTATION

The following symbols are used in this paper

C_t = total sediment concentration, in parts per million by weight;

D = average depth of water;

d = median sieve diameter of bed material;

Q = water discharge;

q_t = total sediment discharge;

q_{BM} = bed material discharge;

S = water surface slope or energy slope;

U_* = shear velocity;

V = average water velocity;

V_{cr} = critical average water velocity at incipient motion;

VS = unit stream power;

$V_{cr}S$ = critical unit stream power at incipient motion;

ν = kinematic viscosity;

ω = terminal fall velocity

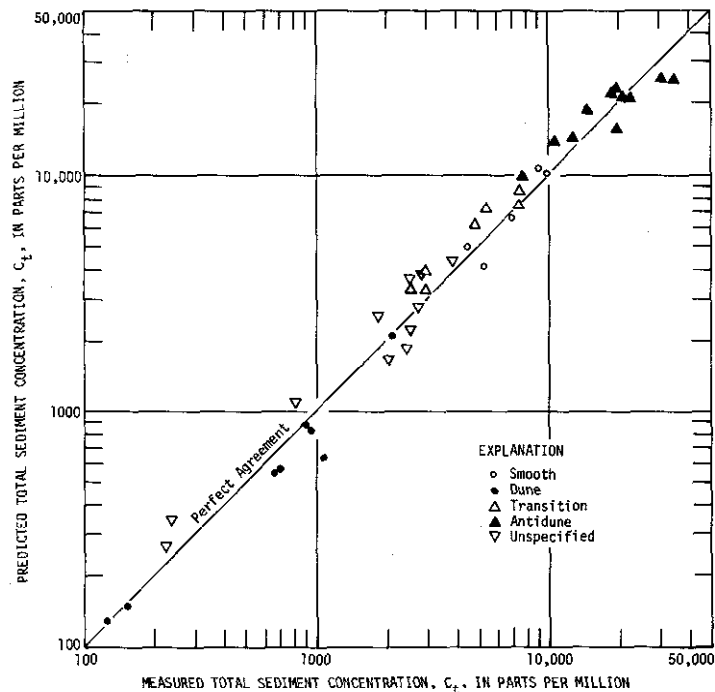


Figure 1. Comparison between predicted total sediment concentrations by Eq. 1 and the measured concentrations by Gilbert with 0.375 mm sand in a 1-ft wide flume

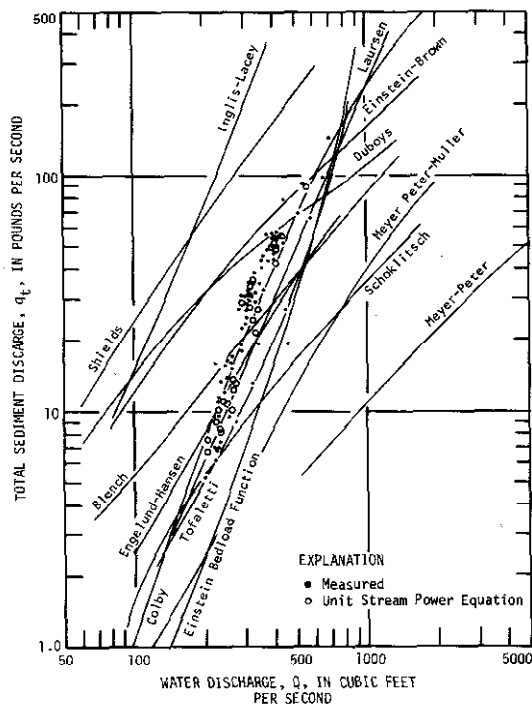


Figure 2. Comparison between measured total sediment discharge of the Niobrara River near Cody, Nebraska and computed results of various equations

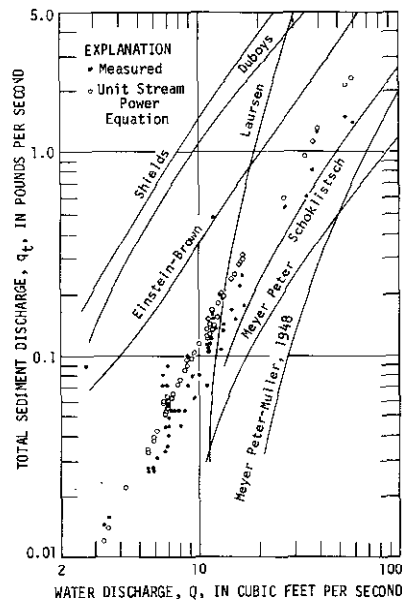


Figure 3. Comparison between measured total sediment discharge of Mountain Creek at Greenville, South Carolina and computed results of various equations

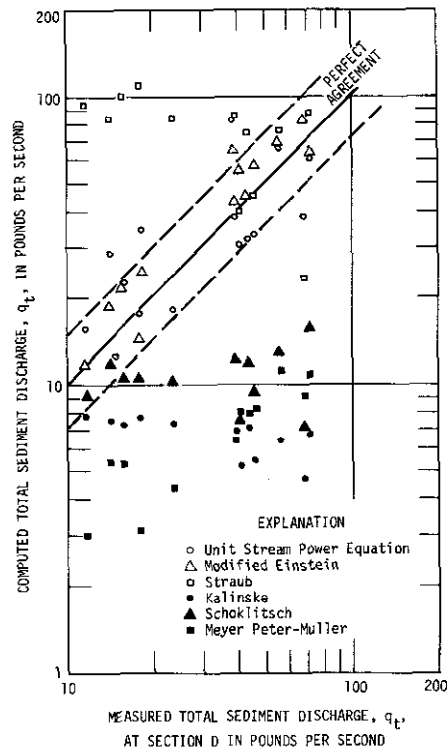


Figure 4. Comparison between measured total sediment discharge of the Middle Loup River at Dunning, Nebraska and computed results of various equations

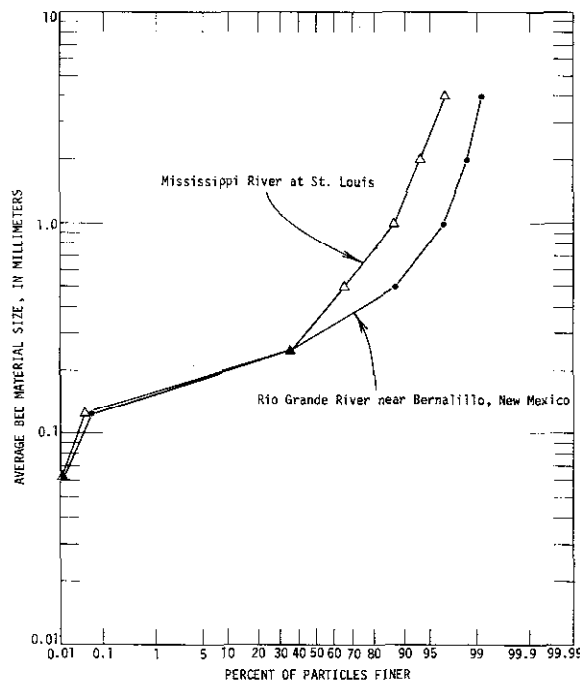


Figure 5. Size distribution of bed material from the Mississippi River at St. Louis, Missouri, and the Rio Grande River near Bernalillo, New Mexico

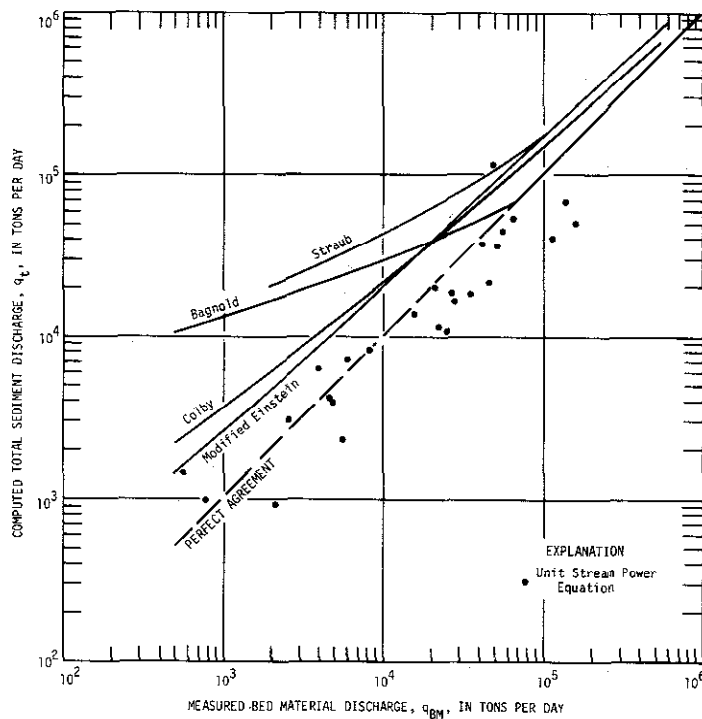


Figure 6. Comparison between measured bed material discharge of the Mississippi River at St. Louis, Missouri, and computed results of different equations

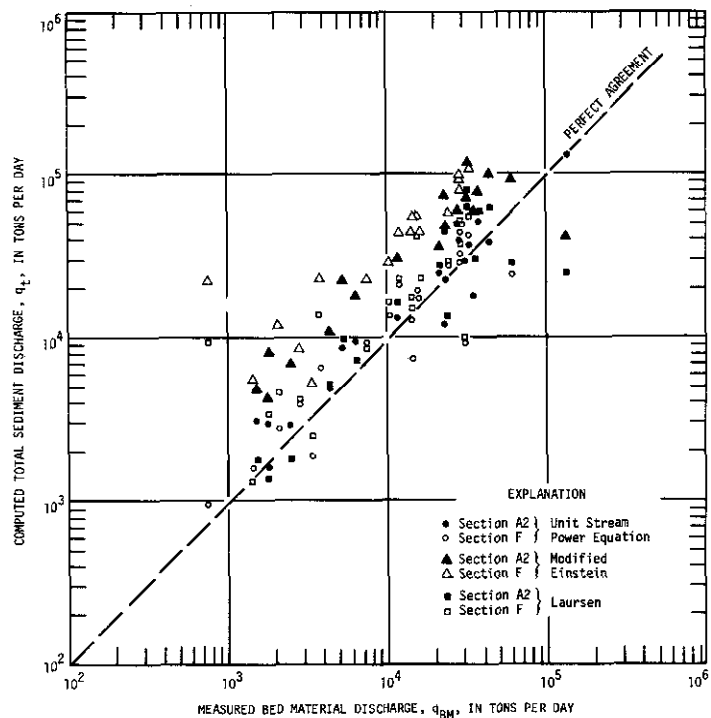


Figure 7. Comparison between the measured bed material discharge of the Rio Grande River near Bernalillo, New Mexico, and the computed results of various equations

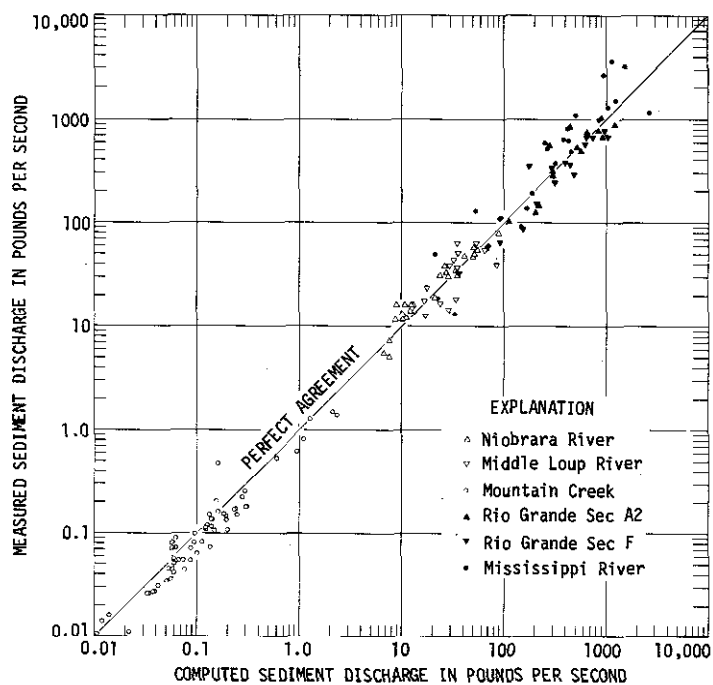


Figure 8. Comparison between measured sediment discharge at six river stations and computed total sediment discharge of Eq. 1

ENTRENCHMENT OF DRAINAGE SYSTEMS IN WESTERN IOWA AND NORTHWESTERN MISSOURI^{1/}

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ABSTRACT

During the past few decades, drainageway erosion in the loess hills region of Iowa and Missouri has accelerated, and there is little deterrent to continued erosion on a massive scale. It is complexly linked to the general runoff regimen and to changes in soil-water content. Overland runoff volumes due to intensive cultivation are two to three times presettlement volumes, and peak flow rates average more than 10 times the presettlement norm that existed before 1850.

The erosion problem was assessed from random channel measurements in three counties of the study region, from intensive measurements of sheet-rill and gully erosion rates on experimental watersheds near Treynor, Iowa, and from existing sediment yield records. Although measures of sediment yields at downstream locations usually are less than 12 or 13 metric t/ha (5 or 6 t/a) per year, the total upland erosion (sometimes called gross erosion) is significantly greater. Intermediate-size channels draining areas of more than 2.6 km² (1 mi²) are degrading rapidly, with average losses of 4.5 metric t/ha (2 t/a) sediment annually; gully erosion rates for smaller drainage basins average at least 4.5 metric t/ha (2 t/a) each year; and the soil losses from sheet-rill erosion on fields average at least 18 metric t/ha (8 t/a) annually.

INTRODUCTION

Recorded erosional changes in drainageways (gullies and continuous-flowing channels that drain areas of a few acres to perhaps 100 square miles) in the loess-mantled region of western Iowa and northwestern Missouri indicate a general pattern of channel trenching and widening, and the extension of gullies into upland valleys and fields. Some geologists consider such erosion part of the normal cycle of landscape development in this region of deep loess. During past geologic time, cycles of channel enlargement have alternated with filling.

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Geologists concerned with problems caused by drainageway erosion have suggested that optimum erosion control could be exercised by concentrating control efforts in locations of potential trenching rather than in locations where an erosion cycle has run its "natural" course. In the arid Southwest, Schumm (1975) has even suggested that the most expeditious method for controlling or remedying trenching in some instances might be to withhold upland conservation treatments at critical erosion stages so that the healing cycle could occur more quickly.

Presently no hard evidence is available for determining, with any great certainty, the future erosion trends for a particular drainageway. In the past, both cut-and-fill cycles have occurred within the same time frame in a given locality. Man-induced effects since settlement, e.g., cumulative hydrologic changes induced by row-cropping, are also a complication in predicting future channel/gully erosion patterns. Our objective is to address the problem of modern valley trenching and gully erosion, as affected by white settlement and subsequent activities, and the implications for future erosion control.

DRAINAGEWAY EROSION IN PERSPECTIVE

Drainageway erosion is manifested by land voiding, which includes upslope retreat of scarps or headcuts, and channel widening and deepening; it also is evident in the increased sediment discharges to downstream locations.

As indicated by table 1, the quantities of soil eroded from gullies and channels usually are much less than the soil losses from the cultivated land surface. Because of sediment deposition during transit, sediment yields measured downstream do not always reflect the total upstream erosion. The table lists measured sediment yields from several sampled watersheds of the region and shows that they vary widely for the following reasons.

Watersheds 1 and 2 in Pottawattamie County near Treynor, Iowa, are cornfields planted on approximate contour, which by itself is not an effective conservation practice for these fields with average slopes of 8 to 10 percent. Measured gully erosion in the headcuts and outlet drainageways of these watersheds was one-fifth of the total sediment removed from the watershed.

Watersheds 3, 4, and 5 at the same locality are conservation watersheds with vegetative or structural erosion controls, or both. Sediment yields from these watersheds were minimal during the period of sediment measurement, although erosion in the outlet drainageway (gully) of brome-grass-seeded watershed 3 accounted for nearly one-half of the total sediment yield.

Most of the drainage systems above the Thompson and Steer Creek gages have been enlarging except where recent channel structures have stabilized the grade. Taylor (1970) reported that, as late as 1967, the measured drainageway erosion was 48 percent of the sediment leaving Steer Creek watershed.

A Mule Creek Watershed Association was formed in 1949 to combat excessive erosion there. It reported, through the Mills County Soil Conservation District (1959), that gully erosion at one bridge location cost about \$1,000/yr for maintenance and that another bridge in the area, which was 9.7 m (32 ft) long in 1910, was lengthened to 40.2 m (132 ft) by 1958 after several replacements. Post-treatment sediment yields since the completion of the project are less than one-fourth the former rates.

Measurements in the Tarkio River basin were made about 40 years ago and may reflect the paucity of soil conservation treatments at that time. Although the sediment yields of 11.2 metric t/ha (5 t/a) annually seem innocuous, erosion was widespread and is continuing on upland fields and channels; based on normal sediment delivery relationships (ASCE, 1970), the average erosion rate was at least 45 metric t/ha (20 t/a).

The last entry of Table 1 is for the largest sampled channel. The measured downstream sediment yields for these larger drainage basins normally are less than 20% of total erosion in the upstream source areas. Therefore, the average annual erosion rate for the East Nishnabotna River basin probably is about 45 metric t/ha (20 t/a), with many areas contributing greater quantities. Some erosion trends in the Nishnabotna drainage system since white settlement are documented herein.

Table 1.--Measured Runoff and Sediment Yields^{1/} of Watersheds in Western Iowa and Northwest Missouri

Watershed	Cropping and Conservation Treatment	Measurement Period	Drainage Area	Runoff			Sediment Yield	
				Subsurface	Surface	Total	All Sources	Estimated from Gullies and Channels
				centimeters/year			Metric Tons/Hectare/Year	Percent
			Hectares					
ARS Watershed 1 near Treynor, IA	Corn on approximate contour	1964-1974	30.4	8.4	9.9	18.3	57.8	22
ARS Watershed 2 near Treynor, IA	Corn on approximate contour	1964-1974	33.6	9.6	9.2	18.8	44.6	18
ARS Watershed 3 near Treynor, IA	Bromegrass, rotation grazed ^{3/}	1964-1974	43.3	11.9	3.4	15.3	1.4	33
ARS Watershed 4 near Treynor, IA	Corn, level terraced ^{3/}	1964-1974	60.7	17.3	3.0	20.3	3.4	0
ARS Watershed 5 near Macedonia, IA	Mixed crop, level terraced	1964-1967	158	12.2	5.6	17.8	2.2	5
Thompson Creek near Woodbine, IA	Mixed cropping and treatment	1964-1969	1,800	--	--	8.6	7.0	2/
Steer Creek near Magnolia, IA	Mixed cropping and treatment	1964-1969	2,400	--	--	5.8	5.4	46
Mule Creek near Malvern, IA	Mixed cropping and treatment	1954-1969	2,740	--	--	13.6	6.7	2/
Dauids Creek near Hamlin, IA	Mixed cropping and treatment	1953-1968	6,760	--	--	13.0	5.6	2/
West Tarkio Creek near Westboro, MO	Mixed cropping and treatment	1934-1940	27,200	--	--	7.5	10.8	2/
Tarkio River at Blanchard, IA	Mixed cropping and treatment	1934-1940	51,800	--	--	7.3	11.7	2/
East Nishnabotna River at Red Oak, IA	Mixed cropping and treatment	1963-1973	231,000	--	--	15.8	8.7	2/

^{1/} Published records of USGS and ARS.

^{2/} No measurements or accurate estimates are available.

^{3/} Changed to conservation-tilled corn in 1972.

Note: 1 Hectare = 2.47 acres; 1 metric ton per hectare = 0.46 tons per acre.

HISTORY AND MEASUREMENTS OF DRAINAGEWAY DEVELOPMENT

Channel Erosion

Daniels (1966) explained the cyclic details of entrenched stream systems and gullies in Thompson Creek, Harrison County, Iowa. He reported that stream and gully entrenchment was nonexistent in 1850 because a previous trenching and filling cycle had just been completed; erosion did not again become evident until about 1880 or 1890 and has accelerated since the 1940's. Beer (1962) recounted the history of drainageway erosion in nearby Steer Creek. Using original surveys, personal interviews with long-time residents, aerial photos, and modern surveys, he also found that channel erosion did not become serious until the 1930's.

Notes from original land surveys in Pottawattamie and Harrison Counties, Iowa (circa 1852), and Atchison County, Missouri (circa 1846), are replete with mention of swamps and marshes, with rather sluggish stream systems in general. Drainageways, by comparing original survey information with present-day measurements, have widened tremendously; e.g., at 30 randomly selected channels in Pottawattamie County, the average width increased from 3 m (10 ft) about 1850 to 18.3 m (60 ft) in 1975. The average depth of these channels in 1975 was 5.5 m (18 ft). Depth changes in channels are not as well documented as the widths, although evidence is available to show that drainageway erosion has been especially severe during the past few decades.

Some of the more accurate records of channel changes have been gleaned from highway bridge surveys. The channel cross section (Figure 1) was randomly selected for resurvey in 1975 from about 20 sites in Atchison County where both original survey data and bridge records were available.

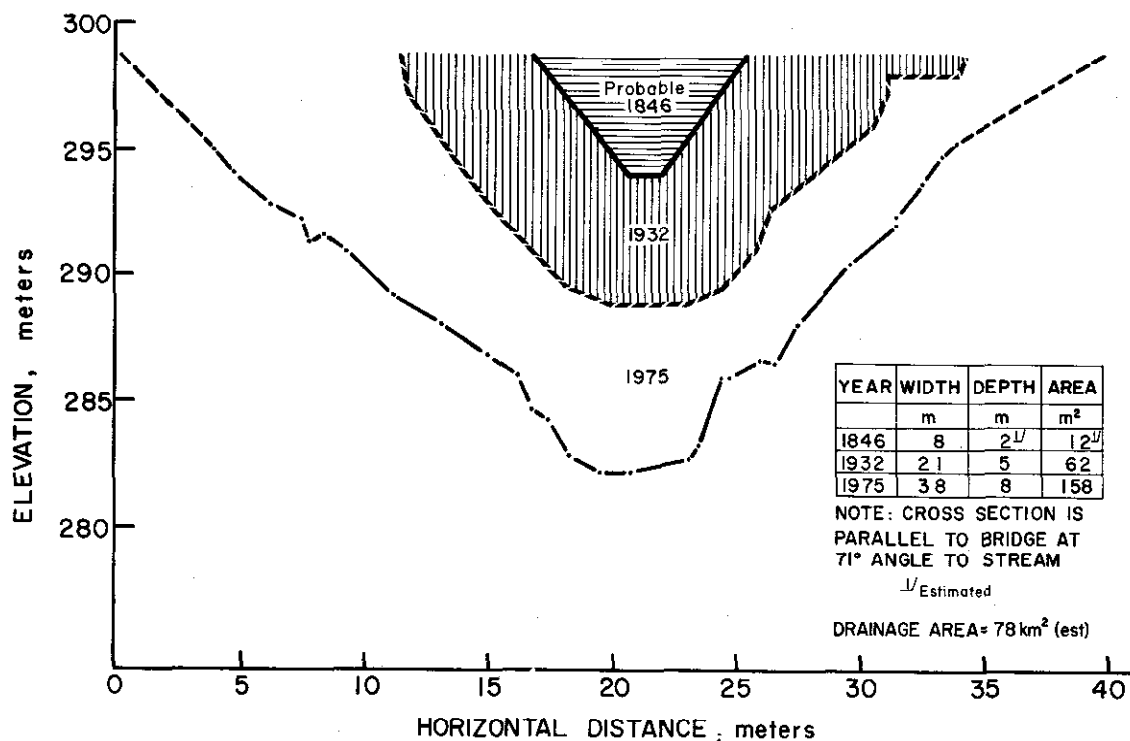


Figure 1.--Cross section showing modern trenching of an Atchison County, Mo. drainageway.

An increase of 2.4 m (8 ft) in channel depth is estimated at this location during the first 86 years since settlement, most of which presumably occurred in the later years; trenching was accelerated after 1932, with a 3.4 m (11 ft) measured channel scour during the past 43 years. Although this is a larger-than-average scour rate, it is not exceptional. The average scour in 15 upland channel reaches in Atchison County, each with drainage areas of less than 259 km² (100 mi²), was 1.1 m (3.7 ft) in 33 years, or about 0.3 m (1 ft) per decade. This conservatively represents the rate of downcutting in these channels. A more serious case of channel trenching has been recorded by the Soil Conservation Service (unpublished) on West Tarkio Creek near the Iowa-Missouri boundary. Channel profile and cross section changes since dredging about 1920 are portrayed in Figure 2. By examining 17 cross sections in a 24.1 km (15 mi) length of channel, the average annual channel erosion rate was calculated at 119,000 metric tons (131,000 tons) per year.

Other Records

Unpublished records of the U.S. Geological Survey for streamflow gaging stations offer another method for calculating channel fill or scour trends, although the number of gaged streams is limited and most records are for larger, more stable streams. We examined about a dozen records and found relative stability at most locations, or slight fill for larger rivers. The streambed of the smallest watershed examined, Mill Creek at Oregon, Mo., 12.7 km² (4.9 mi²), degraded only about 0.15 m (0.5 ft) from 1950-1974, possibly because it is located near the Missouri River floodplain. The Kimsey Creek crossing at Interstate 29, just 9.7

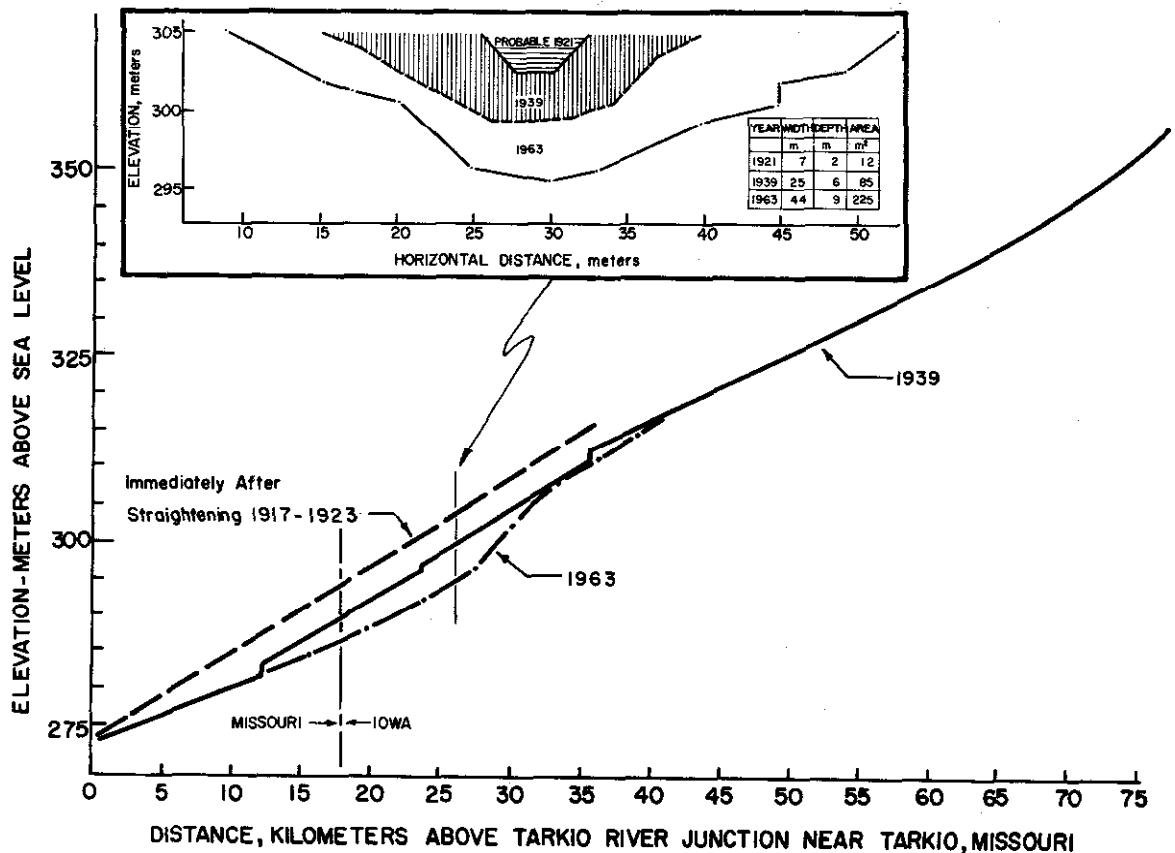


Figure 2.--Profile and cross section changes, West Tarkio Creek near Missouri-Iowa boundary.

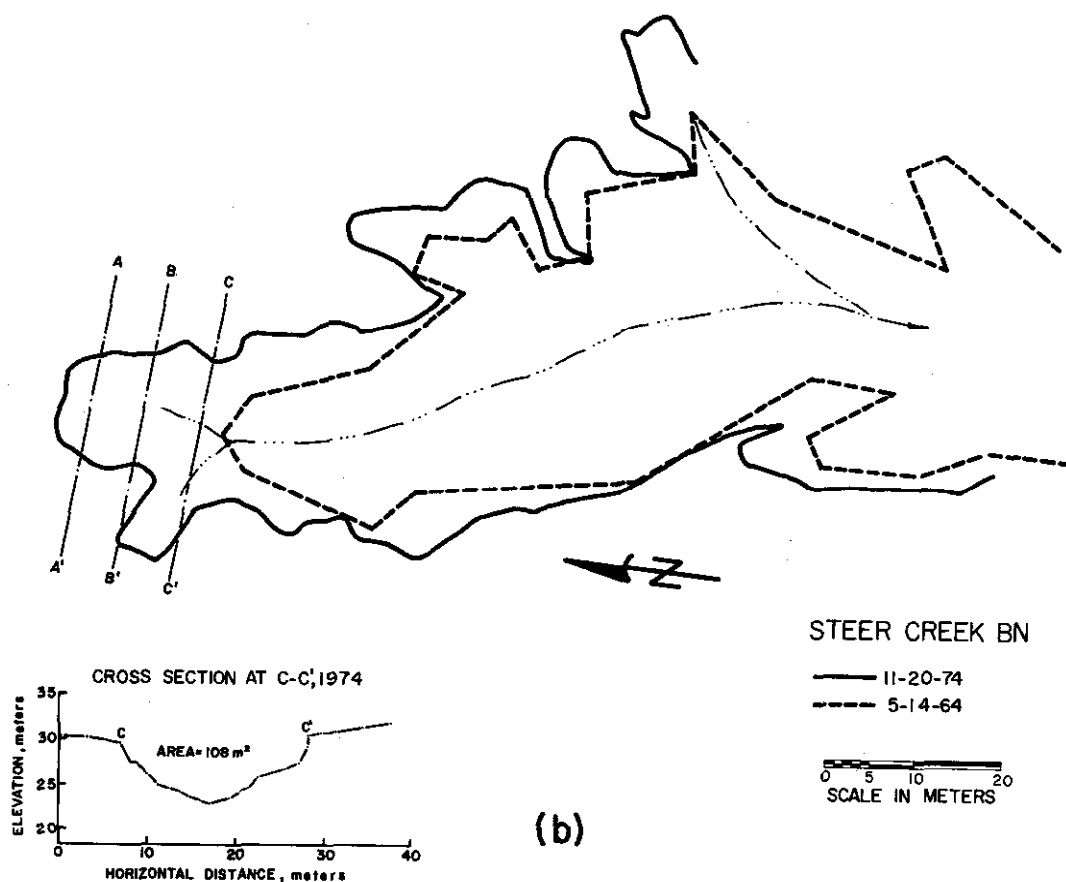
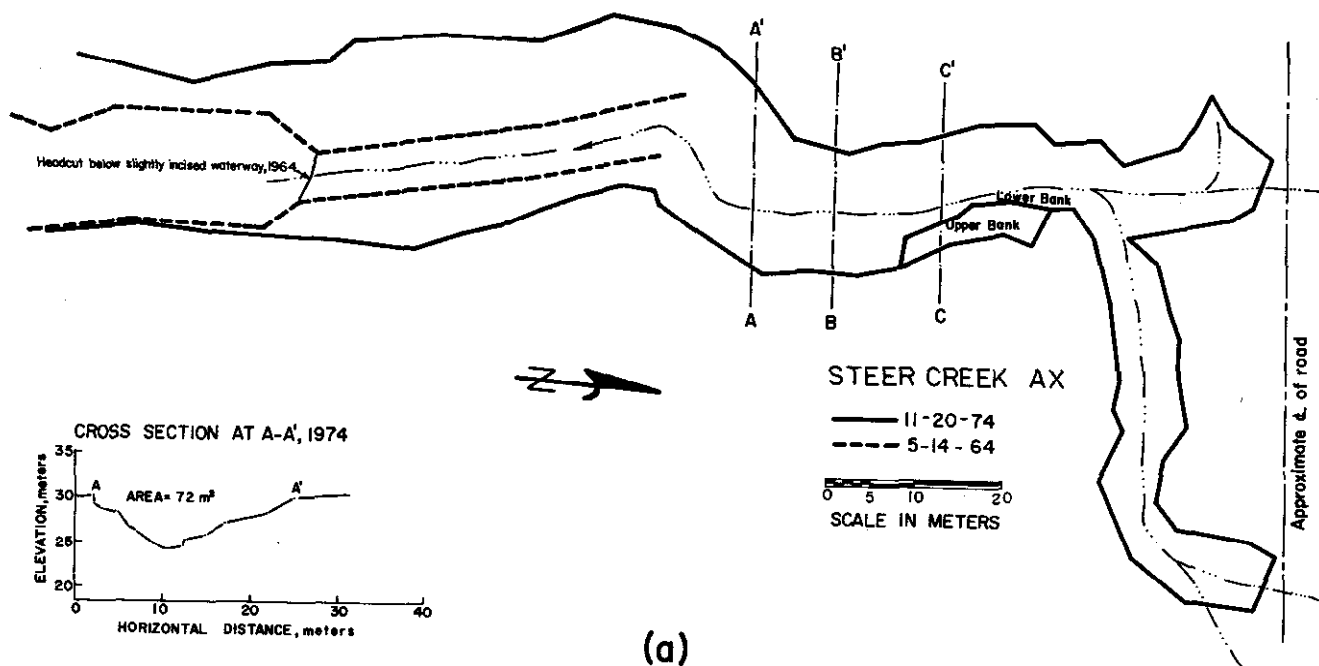


Figure 3.--Headward advance of gullies in upper Steer Creek Basin, Harrison County, Iowa.

kilometers (6 miles) to the northwest, scoured 1.5 meters (5 feet) since the first Highway Department survey in 1957. The most consistent channel degradation noted from the few streamgaging records examined was at the East Fork of the 102 River near Bedford, Iowa, where the channel degraded 2.1 meters (7 feet) during the period 1959-1973.

Gully Erosion

Most of the drainageway erosion previously referred to was on perennial streams with drainage areas exceeding about 2.6 km^2 (1 mi^2), which at one time was probably the minimum size for identifiable channels. In Harrison County, Beer (1962) charted the positions of gully overfalls on smaller tributaries to Steer Creek and derived a prediction equation for gully growth. He also found that gully development on all but two of the 25 tributaries occurred after 1915. Daniels (1966) referred to these as valley slope gullies and, from examination of aerial photographs of Thompson Creek watershed, found that 42 gullies were identifiable in 1938, 63 in 1954, and 68 in 1957. They concluded that, although gullying had been active for 50 years or more, gully activity was greatest after the mid-1940's. We have surveyed the scarp advance of one Thompson Creek and two Steer Creek gullies since 1964. The Steer Creek gullies are mapped in Figures 3a and 3b, along with a typical cross section of the newly eroded area. In a 10-yr period at gully AX, approximately 9,100 metric t (10,000 t) of soil was removed from a 122 m (400-ft) reach of channel, even though erosion rates were somewhat reduced by the gully advancing to the upstream roadway. During the same 10-yr period, gully BN, which drains less than 8 ha (20 a), has advanced 18.3 m (60 ft) through a pasture and has eroded more than 5,440 metric t (6,000 t) of soil.

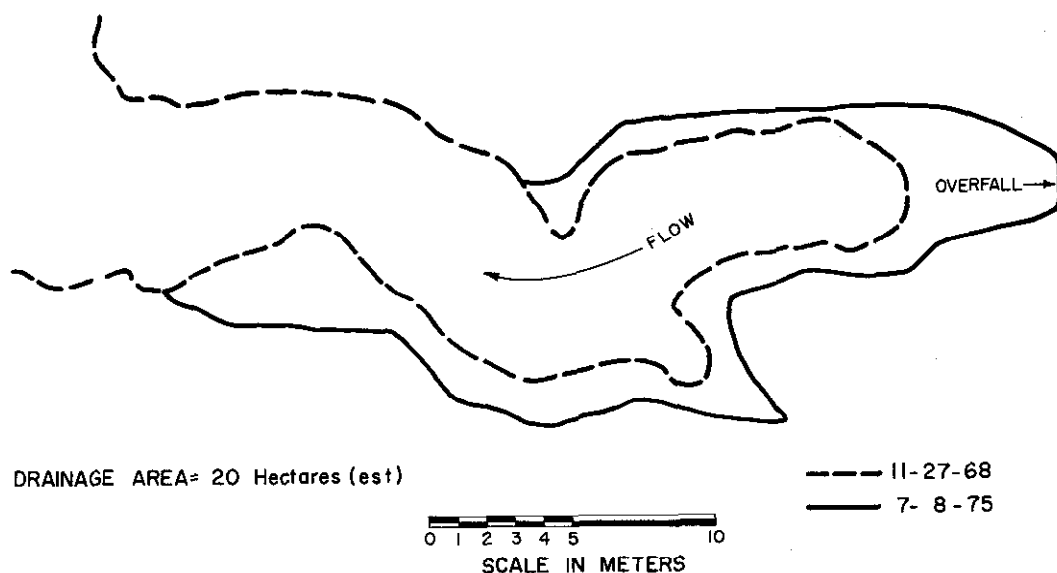


Figure 4.--Headward advance of gullies draining a small field in Atchison County, Mo.

In Pottawattamie County, gully erosion rates and associated runoff were measured from four watersheds near Treynor during the same 10-yr period and are summarized in Table 1. The most active gully retreated headward more than 45.7 m (150 ft) in 10 years, widened considerably, and yielded more than 3,630 metric t (4,000 t) of soil. Gully erosion at these locations is reported elsewhere in more detail (Piest, 1975).

A similar study was begun in 1968 to detect gully advance in Atchison County when two gully headcuts in a field near Tarkio were surveyed. The advance of one of these is shown in Figure 4. Although the quantities of soil eroded were small, the erosion rate relative to the contributing drainage area was large.

THE REGIONAL IMPACT OF DRAINAGEWAY EROSION

The magnitude of erosion from drainageways--gullies and channels--of the entire Iowa-Missouri deep loess region can only be approximated. Available measurements constitute an insignificantly small sample of the area, and the dynamics of sediment entrainment, transport, and deposition often obscure the actual upland erosion rates.

The Treynor, Iowa, watersheds, which are fairly representative of the region in terms of land use, treatment, and physio-hydrologic characteristics, should give some insight as to the magnitude of gully erosion. Table 1 shows that overall gully erosion on these four watersheds averaged about 730 metric tons (800 tons) per year from 162 hectares (400 acres), or 4.5 metric t/ha (2 t/a) annually. This was about 20 percent of the total measured sediment yield from these watersheds; the remainder was sheet-rill erosion from the field surfaces.

Present-day channel erosion also can be approximated by noting erosional changes like those documented for Atchison County. Only medium-size drainageways were included in this survey because those with drainage basins of less than 2.6 km² (1 mi²) were considered in the gully category, and most basins draining more than about 390 km² (150 mi²) were relatively stable. Assuming an average channel degradation of 0.3 m (1 ft) per decade, commensurate widening and areal change based on channel geometry at a number of surveyed cross sections of 6.5 m² (70 ft²) per decade, and an average channel density of 0.47 km/km² (0.75 mi/mi²) based on inspection of the U. S. Geological Survey topographic maps of the Treynor, Iowa, and Tarkio, Missouri, areas, the average annual channel erosion rate is 420 metric t/km²/yr or 902 metric t/channel km/yr (1,200 tons/square mile/yr or 1,600 t/channel mi/yr). This is much less than the known channel erosion on West Tarkio Creek, which is 438 metric t/km²/yr (1,250 t/mi²/yr) based on only the measured erosion in a 24.1 kilometer (15-mi) channel reach of a 27,100 ha (67,000 a) watershed, or 4,900 metric t/channel km/yr (8,700 t/channel mi/yr). This is a serious problem, however, not only because large quantities of soil are transported, but also because there is no natural deterrent to further channel scour, and a subsequent increase in gully erosion can be expected due to lowered base levels.

FACTORS AFFECTING ACCELERATED DRAINAGEWAY EROSION

Piest (1975) discussed the causes of gully/channel erosion and the mechanisms of lateral channel enlargement in western Iowa. Wetting of banks by seepage from groundwater and (or) saturation from rainfall and runoff was cited as a principal cause of bank failure; wetting of the soil profile decreased soil strength and increased soil stresses because of seepage pressures and gravity loading. After failure of banks, runoff above some "threshold level" was required for "clean-out" of the soil debris. The tractive forces of runoff were only casually related to drainageway erosion because measured velocities above 3 m/sec (10 ft/sec) did not appreciably erode the drainageway. Therefore, runoff was considered a necessary, but not sufficient, condition for gully/channel erosion in this loessial region; mechanisms that create a supply of loose soil debris must also be active.

One of the earliest reasons given for modern excessive channel erosion has been the dredging and straightening operations that began during the last century and are continuing. Channel realignment is considered to result in steeper slopes, higher runoff velocities, and resultant channel scour. An equilibrium relationship has been aptly expressed by Lane (1955) as:

$$QS \propto GD, \quad (1)$$

where Q is runoff rate

S is longitudinal slope of stream channel

G is the rate of sediment in transport that is readily available in considerable quantities in the bed

D is the particle diameter of sediment

The relationship is primarily applicable for alluvial channels with noncohesive beds but has some validity for channels in cohesive materials--as generally found throughout smaller drainageways in the Midwest. Therefore, if the channel slope is increased and the runoff rate does not change, the rate of transport of sediment from the streambed will increase.

But there is evidence that runoff has also substantially increased over pre-settlement time when the study region was vegetated with native grasses and a scattering of trees. Experiment station data, like that at LaCrosse, Wisconsin (Hays, 1949) and Clarinda, Iowa (Browning, 1948) showed that surface runoff from cropland is much higher than from grassland. At LaCrosse, surface runoff from bluegrass was less than one-fourth that from continuously cropped cornland, one-third that from small grain, and half that from red clover. Measurements on the Treynor, Iowa, field-size watersheds showed the same trend in runoff yields, Figure 5. Spomer (1973) previously summarized this information by pointing out that surface runoff from the continuously cropped cornland was nearly three times that from rotation-grazed pasture. The total water yield of grassland, which includes surface runoff and groundwater, is less than that from cropland because consumptive use (evapotranspiration) by grassland is greater.

The Treynor measurements also show the extent to which peak runoff rates are increased by cropland, Figure 6. Peak rates for all storms with similar rainfall, for corn cropland versus a grassland watershed for the 8-yr period, are generally about 10 to 1 or larger. The large point scatter is due to some rainfall differences and seasonal variation.

Many publications have tabulated land use trends since settlement. In western Iowa, records show that cropland acreages were gradually increased from time of settlement to about 1890 to 1900 and have not substantially increased since. A recent inventory by the Iowa Crop and Livestock Reporting Service shows that about 80% of the area of nine southwestern counties is cropland, with 43% in rowcrops.

Considering Lane's relationship and the information gained in the Treynor research, some conclusions can be drawn. The steepening of drainageway profiles in the study region has been accompanied by large increases in runoff. With an increase in slope and discharge, the drainage system will respond by degrading and (or) meandering in an attempt to re-establish a lower equilibrium slope. But the natural tendency of most channels also would be to enlarge to accommodate greater discharges. The implication is plain: If the farmland is to be utilized as in the past century, these large increases in runoff over the presettlement norm necessitate drainage development. Unless revolutionary changes in land management are effected that reduce surface runoff to or below presettlement levels, it is impossible to return to the presettlement type of drainageway.

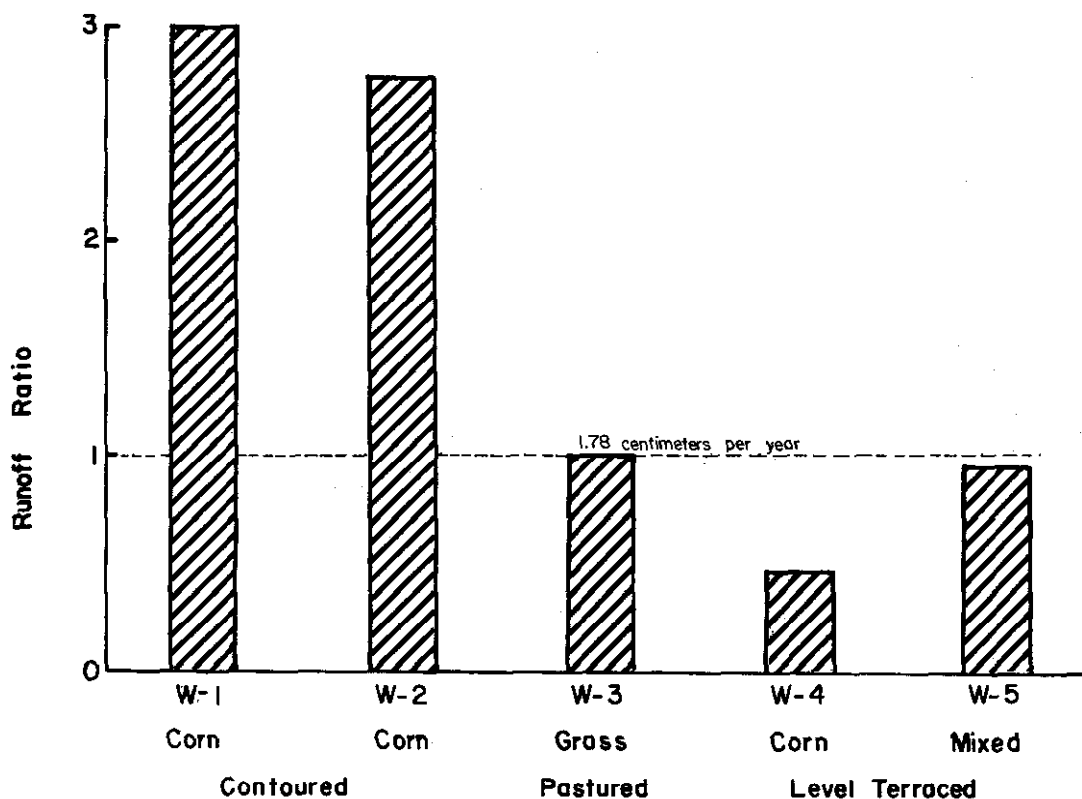


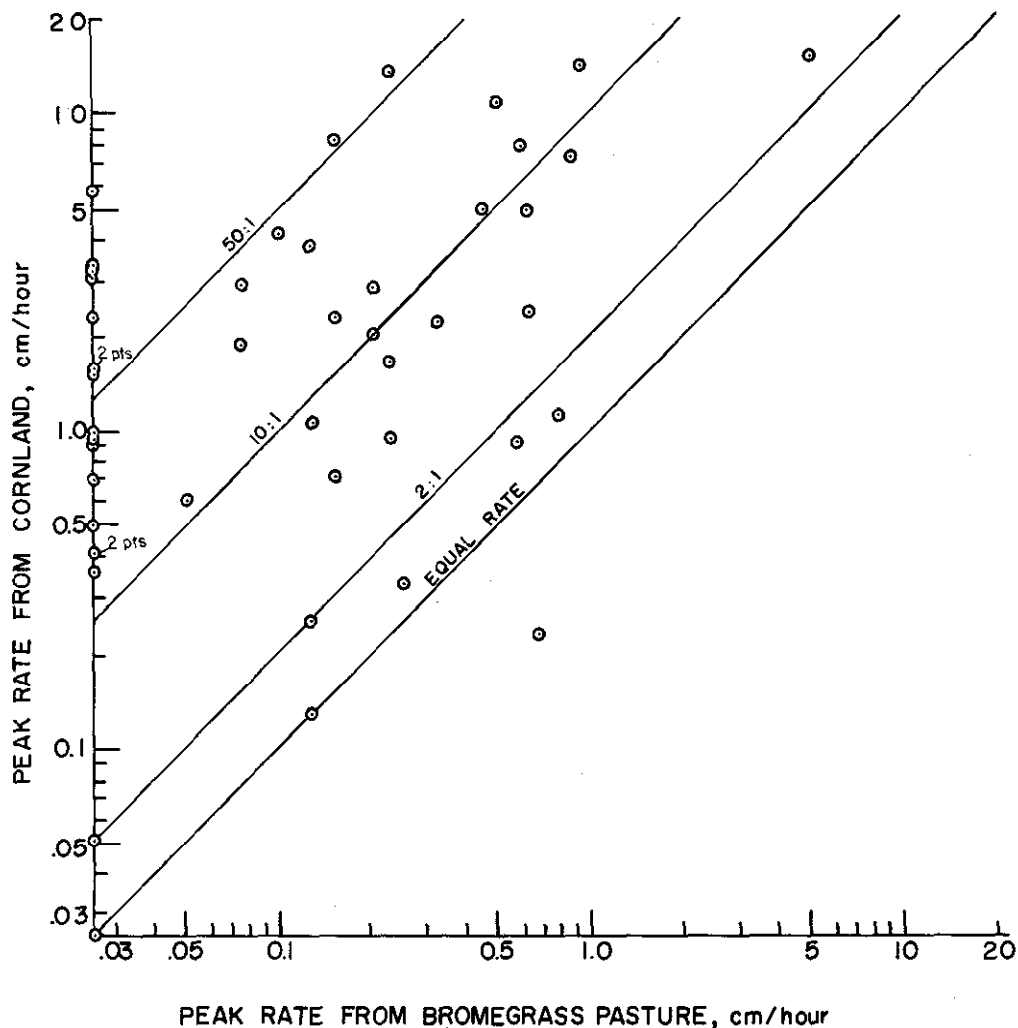
Figure 5.--Surface runoff from cropland as compared to grassland, average annual basis, 1964-1971, Pottawattamie County, Iowa.

FUTURE IMPLICATIONS

Continued downcutting and enlargement of channels can be expected in the loess hills region of western Iowa and northwestern Missouri unless structural controls or unforeseen land-management practices are applied. Upland gullying will accelerate unless overt control efforts are implemented.

Present efforts to control drainageway erosion are inadequate, but the direction of these controls suggests the course for a more concerted future effort. The cost of bridge repair, bridge extension to accommodate increasingly larger channels, and complete bridge replacement has been overwhelming. This is especially true of county expenditures for the many secondary road bridges over streams draining only a few square miles; these incised channels are disproportionately large, their size being only casually related to the flows they convey.

A variety of grade stabilization structures currently is being used to combat drainageway erosion. Atchison County, Missouri, and Pottawattamie County, Iowa, are replacing nearly all failed bridge structures with culverts. This is an economy measure that meets larger load limit standards and also recognizes the need for grade control. In Iowa, a



PEAK RATE FROM BROMEGRASS PASTURE, cm/hour

Figure 6.--A comparison of peak runoff rates, cornland vs. grassland, 1964-1971, Pottawattamie County, Iowa. 5-58

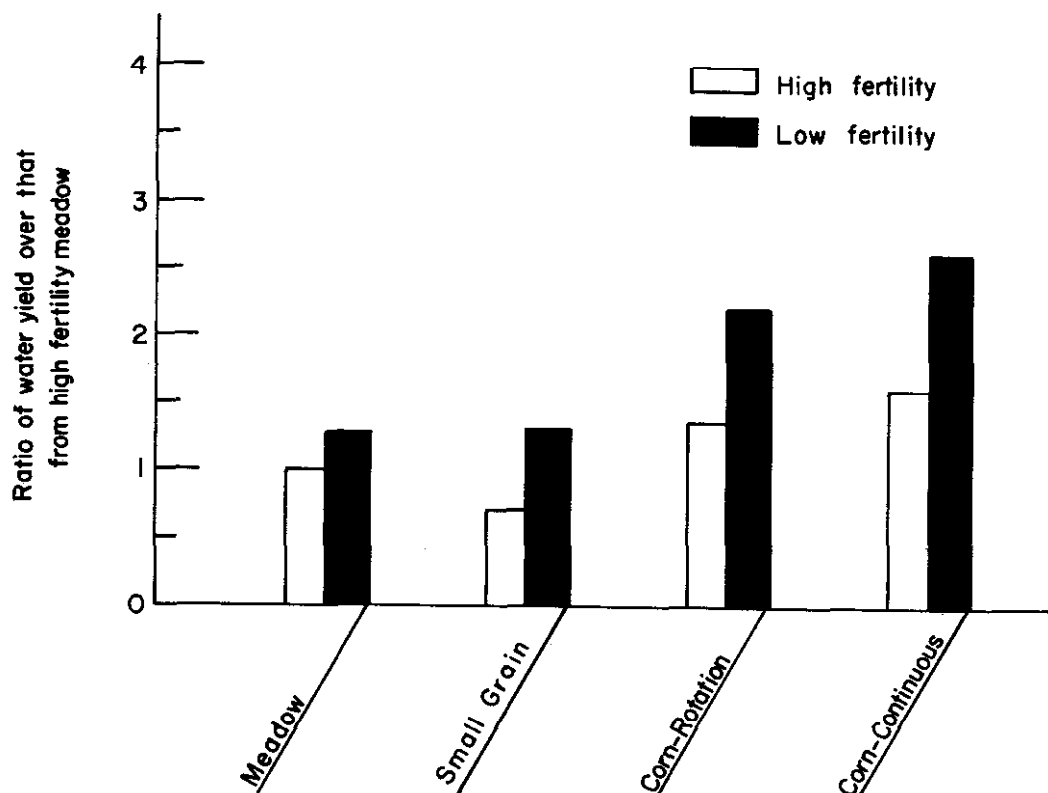


Figure 7.--Effect of fertility level on surface runoff.

series of structures called "Greenwood flumes" is being installed in incised drainageways, each to serve as grade control for several miles of channel and up to a 12.2 m (40 ft) overfall. Road-fill dams and drop inlet structures are increasingly utilized in both states.

But these structures do little to thwart the continued headward advance of gullies into upland fields. Piest (1975) concluded that any gully control effort must be accompanied by reducing runoff rates and amounts to some threshold level not presently known. Recent trends in agriculture offer some long-term hope for runoff reduction. Minimum and no-till conservation systems, if they become widespread, can alter runoff patterns. Although most research to date indicates that water yields from no-till cropland, as compared with normal cropping methods, are not drastically reduced, the peak rates are reduced and flows are spread over a longer time. Another management option to reduce surface runoff involves fertilization practices. Figure 7, from Saxton (1970), shows that surface runoff can be reduced by maintaining high fertility levels.

SUMMARY

The loess hills region adjacent to the Missouri River in Iowa and north-western Missouri has unique erosion problems in many respects. The soil is too fertile to go uncultivated, yet too steep to be farmed by conventional methods without excessive surface runoff and resulting erosion of fields and drainageways. Historic and geologic evidence shows that channel and gully erosion, nearly nonexistent when the area was settled

in the mid-nineteenth century, is now quite extensive. Whether the widespread gully/channel erosion will diminish in a natural cycle, or continue unabated is an important question.

Our evidence strongly suggests that there is presently little deterrent to continued gully/channel erosion on a massive scale. The natural conditions that formerly controlled drainageway development have been irrevocably changed. We are, therefore, committed to the best management of channel systems that are already altered by man, through channel rectification measures or, more subtly, by hydrologic changes occurring in the drainage basin. Overland runoff volumes due to intensive cultivation are shown to be two to three times the presettlement volumes, and peak flow rates now average up to 50 times the presettlement norm. These circumstances alone would have been sufficient to initiate drainageway enlargement (and some straightening) without the overt activities of man.

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CHANNEL CHANGES IN THE COLORADO RIVER BELOW GLEN CANYON DAM

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ABSTRACT

Construction of Glen Canyon Dam on the Colorado River by the Bureau of Reclamation started in 1956, and the dam was officially closed in March 1963. The 1957 degradation study based on a detailed bottom sediment sampling program in 1956 and an analytical approach to sediment transport and armor resulted in a prediction of about a 1.22-meter (4-foot) degradation immediately below the dam and 8.26 million cubic meters (6,700 acre-feet) of erosion to take place in the reach above the mouth of the Paria River in about 14 years.

To verify changes since closure of the dam, resurveys of cross sections in the approximate 24-kilometer (15-mile) reach of the Colorado River from Glen Canyon Dam to the mouth of the Paria River near Lees Ferry were made in 1959, 1963, 1965, and 1975. Bottom sediment samples were also collected in 1966 and 1975 to analyze the changes in material size as a result of the degradation and armoring process. The resurveys confirmed the 1957 predictive study by noting that sufficient armoring material remained to control excessive degradation with about 9.87 million cubic meters (8,000 acre-feet) of material scoured from the channel bottom by 1975. This paper summarizes the 1957 predictive study of probable degradation and describes the changes that have taken place in the channel from 1956 to 1975.

INTRODUCTION

In the design of Glen Canyon Dam, engineers recognized that sediment carried by the Colorado River would be trapped in Lake Powell with an estimated annual sediment inflow to the reservoir averaging about 128 million cubic meters (104,000 acre-feet) (Gessel, 1963). With sediment being deposited in the reservoir, releases through the powerplant or spills through the spillway would be clear water capable of degrading the channel downstream from the dam. A 1956 field study encountered problems in collecting adequate field data on channel hydraulics and river bottom sediments needed to predict downstream channel changes. Development of analytical procedures on sediment transport and armoring material proved equally as difficult as application of engineering judgment to formulate conclusions and quantify downstream channel degradation. The degradation and tailwater studies completed in 1957 served as a basis for final design of the powerplant, outlet works, and spillway outlet for Glen Canyon Dam.

Resurveys of river ranges below Glen Canyon Dam have provided information for quantifying the degradation occurring since 1963. Samples of river bottom sediment and especially the armoring materials provided information on the phenomena of armoring that can occur below such a structure. Surveys of river ranges and sediment sampling were conducted in cooperation with the Upper Colorado Regional Office of the Bureau of Reclamation in Salt Lake City, Utah.

1957 DEGRADATION STUDY

Anticipating the channel would degrade below the dam, a detailed channel hydraulic and sediment sampling program was undertaken in 1956 (Miller and Borland, 1957).

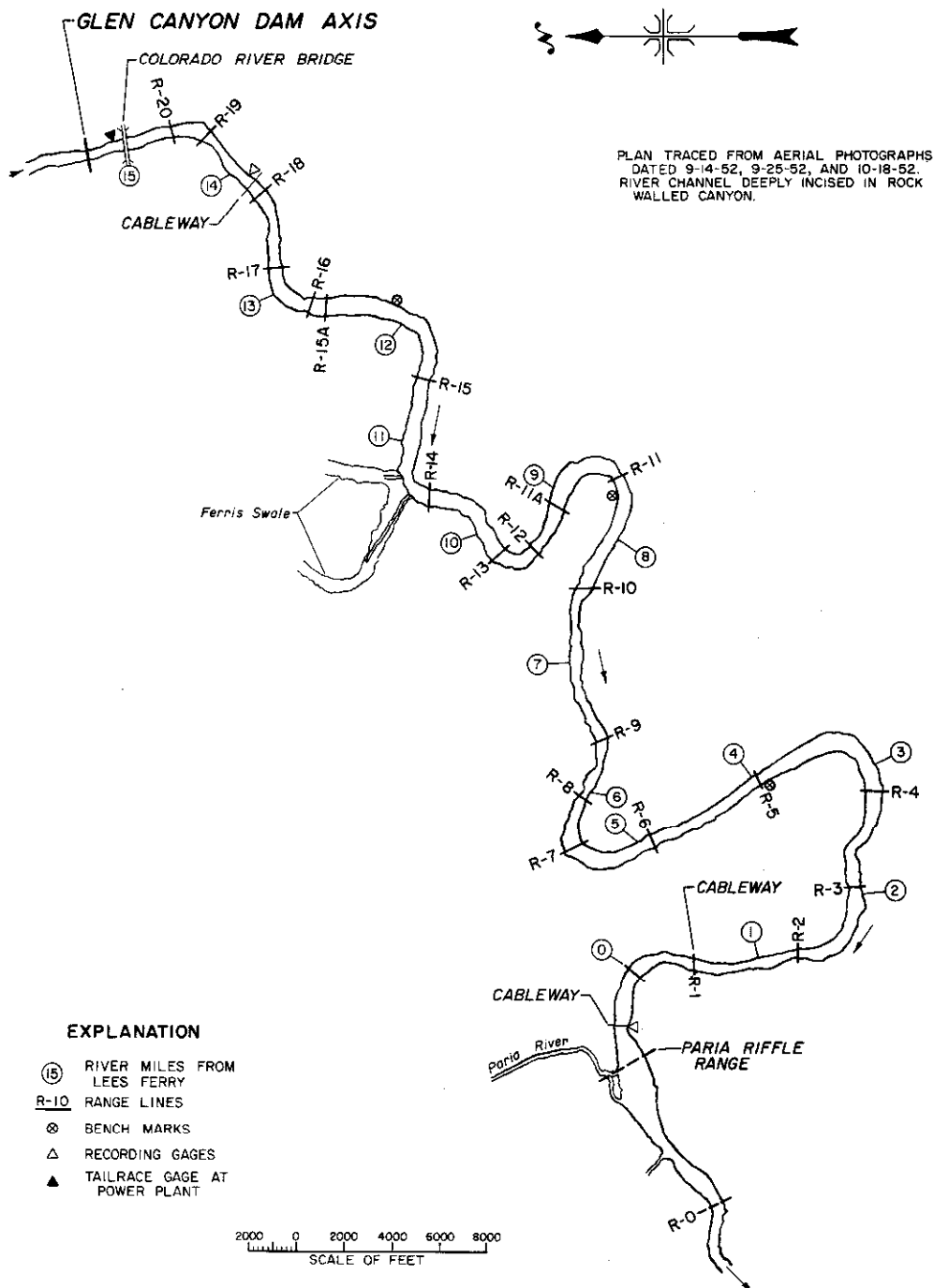
The purpose of collecting field data on the stream bottom sediments was to provide information on the downstream sediments for use in analytical studies on degradation. As a part of the investigations, observations were also made in 1956 on various hydraulic and riverbed conditions that would aid in controlling degradation. This involved locating the exposed gravel or cobble bars and determining the depth to underlying gravel bars or rock controls. Twenty-four river cross sections (figure 1) were also surveyed for use in water surface profile computations. In 1956, 10 downstream ranges (figure 2) were selected for resurvey after the dam was constructed to monitor any downstream changes.

The detailed sediment sampling accomplished in 1956 consisted of three types of samples.

1. Bottom sediment samples of channel bed surface.
2. Ten deep bottom sediment samples penetrating to about 7.6 meters (25 feet) were collected between the damsite and Lees Ferry (figure 3).
3. A jet probe was devised to penetrate through the sand-size material overlying streambed gravel bars and deposits at nine locations randomly selected within the 24-kilometer (15-mile) reach above Lees Ferry. The jet probe was unable to penetrate the underlying gravels thought to consist of large gravels, cobbles, or possibly bedrock.

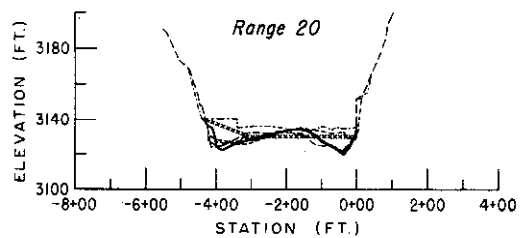
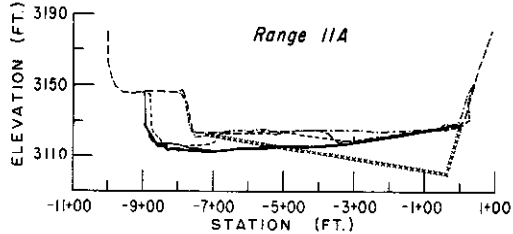
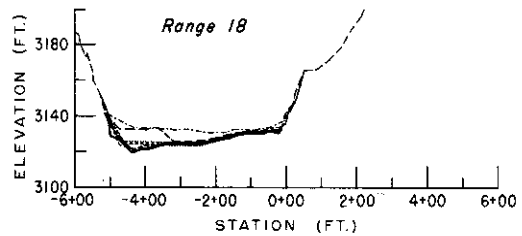
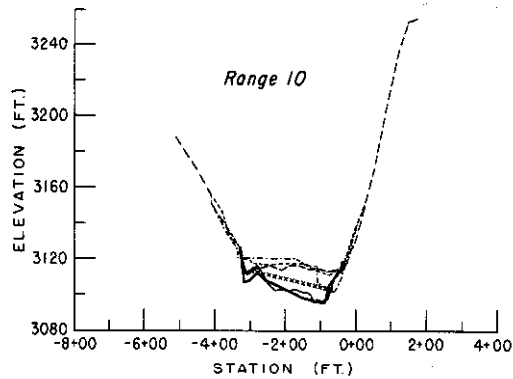
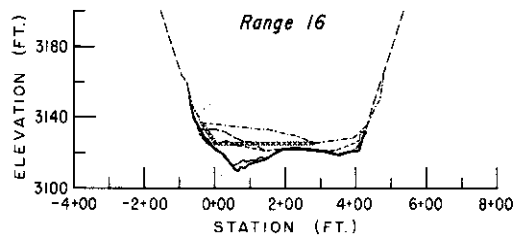
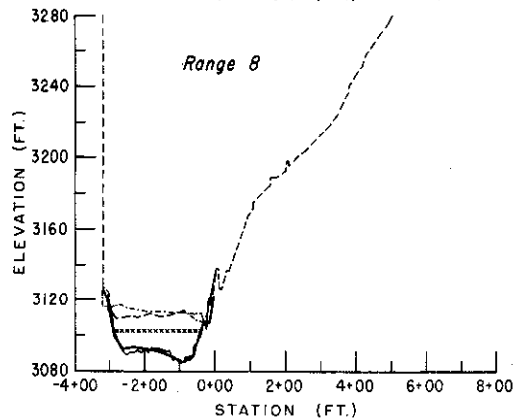
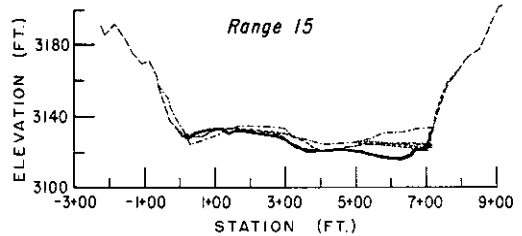
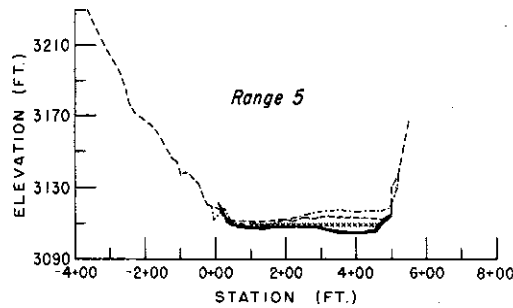
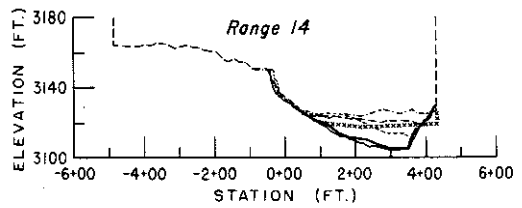
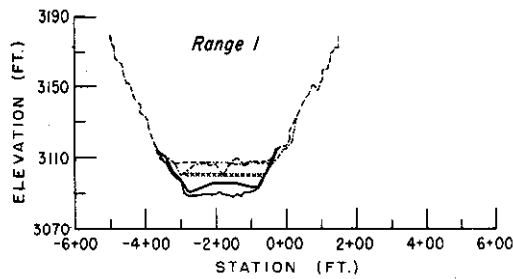
Observations of stream channel conditions influencing degradation indicated that a gravel bar existed below range 15 located about 5.6 kilometers (3-1/2 miles) below Glen Canyon Dam and a bar of gravel and cobble-size material was located at the riffle near the mouth of the Paria River. The bars consisted of rounded, stream-worn, hard rocks varying in size from 4 millimeters to over 203 millimeters (8 inches) in diameter and where exposed (e.g., range 15), they were shingled and well compacted.

The average size analyses for the various channel bottom samples collected in 1956 are shown in figure 4. The mean size (D_{50}) of the surface bottom sediments was 0.21 millimeter, of the sands overlying gravels was 0.32 millimeter, and of the underlying gravels was 20 millimeters. Sediment transport studies for evaluating the degradation below Glen Canyon Dam were made by use of the bedload equations (e.g., Schoklitsch, Haywood, and Meyer-Peter, Muller) and with the anticipated powerplant releases averaging about 382 cubic meters per second (13,500 cubic feet per second). These bedload transport equations indicated that about 617,000 cubic meters (500 acre-feet) of sand-size sediment per year would be scoured from the Colorado River between Glen Canyon Dam and Lees Ferry. A transportable size analysis with varying discharges up to flood flows, indicated sufficient gravels in the size range from 50 millimeters to 203 millimeters (2 to 8 inches) underlying the sands



COLORADO RIVER - ARIZONA
LEES FERRY TO GLEN CANYON DAM AXIS
LOCATION MAP

FIGURE 1



EXPLANATION

- 1956 Survey
- 1959 Survey
- 1963 Survey
- 1965 Survey
- 1975 Survey
- Estimate in 1957
- Degradation Study

NOTES: At the higher elevations all surveys follow the 1959 survey. Looking downstream.

COLORADO RIVER CROSS SECTIONS BELOW GLEN CANYON DAM

FIGURE 2

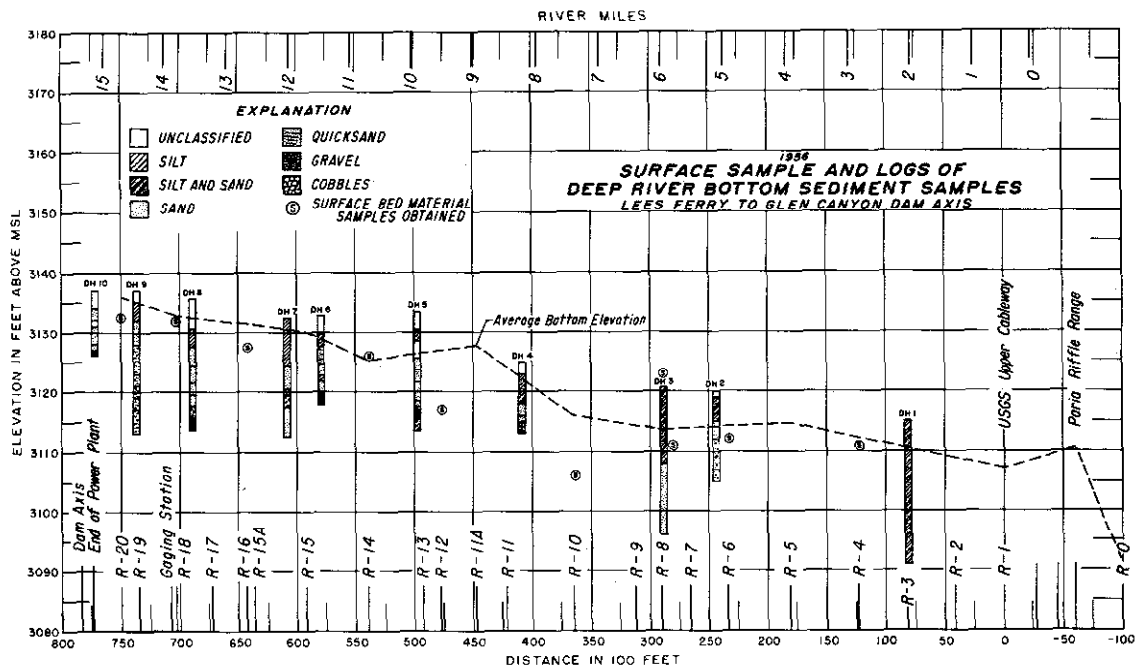


FIGURE 3

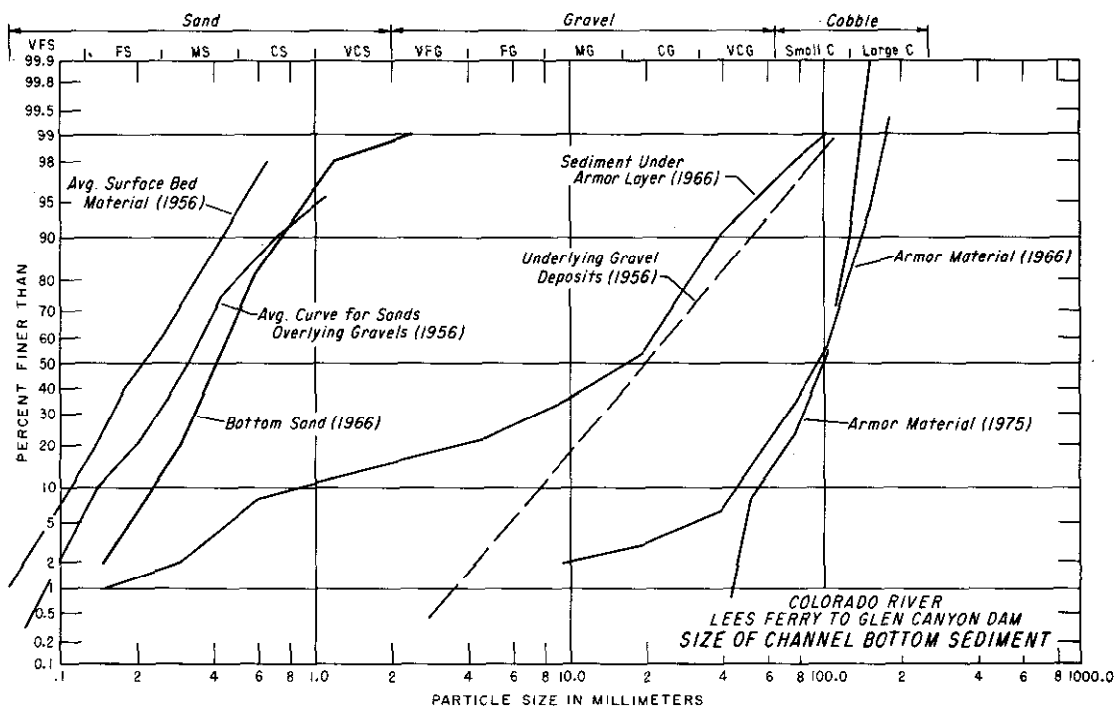


FIGURE 4

to provide stability. Based on the analytical approach for transport, armoring and engineering judgment, the degradation study (Miller and Borland, 1957) concluded that only the sand materials overlying the gravels would be degraded. The volume of this sand-size material that would be degraded was estimated to be about 8.26 million cubic meters (6,700 acre-feet) which for average powerplant releases would take place in about 14 years. Degradation immediately below the dam as limited by armoring would be about 1.22 meters (4 feet).

RESURVEYS AND OBSERVATIONS DURING CONSTRUCTION 1959 - 1963

In July 1959 changes in tailwater in the reach downstream from Glen Canyon Dam construction area were observed. These changes consisted of a 1.2-meter (4-foot) drop in water surface elevation near range 20. A plot of the maximum monthly mean daily peak-flow hydrograph (figure 5) shows that much higher flows occurred during 1959 than the anticipated powerplant releases of about 382 cubic meters per second (13,500 cubic feet per second). Degradation in 1959 was undoubtedly caused by both the high riverflows and the deposition of sediment in the storage area upstream from the cofferdam with about 130 million cubic meters (105,000 acre-feet) of storage.

The 1959 resurveys of the ranges selected for monitoring purposes showed a maximum lowering in mean streambed elevation at range 20 of 1.83 meters (6 feet) since the 1956 survey. A comparison of the 1956 with 1959 surveys showed that 2.17 million cubic meters (1,760 acre-feet) of sediment had been scoured from the Colorado River channel in the 9.7 kilometers (6 miles) below Glen Canyon Dam. There was very little degradation in the remaining 22.5 kilometers (14 miles) above Lees Ferry. Early in 1960, observations of the downstream channel showed that the scouring had also uncovered gravel and cobble bars that were retarding any further scour in most areas and were already becoming quite stable.

To further monitor the apparently high rates of degradation that were taking place during construction, another survey of the downstream ranges was made in 1963 in the reach upstream from range 10. The volume of material degraded from the channel bottom in the 3.55-year period from 1959 to 1963 was 1.52 million cubic meters (1,230 acre-feet) in the reach above range 10. The deposition of sediment above the upstream cofferdam combined with the high relatively clear discharges below the dam between 1959 and 1963 scoured sediment from the reach above range 10 and left a series of controls in the riverbed that were well armored with gravel and cobble-size materials.

1965 AND 1975 RESURVEYS

The complete set of ranges was resurveyed in 1965 with 7.57 million cubic meters (6,140 acre-feet) of sands scoured from the river bottom in the 5.8-year period from 1959 to 1965. This survey was the first since closure of the dam in March of 1963. The high peak release of 1,645 cubic meters per second (58,100 cubic feet per second) in May of 1965 is depicted on figure 6 with a photograph of the dam taken in 1965. This type of release created some deep holes that continued to degrade the downstream channel.

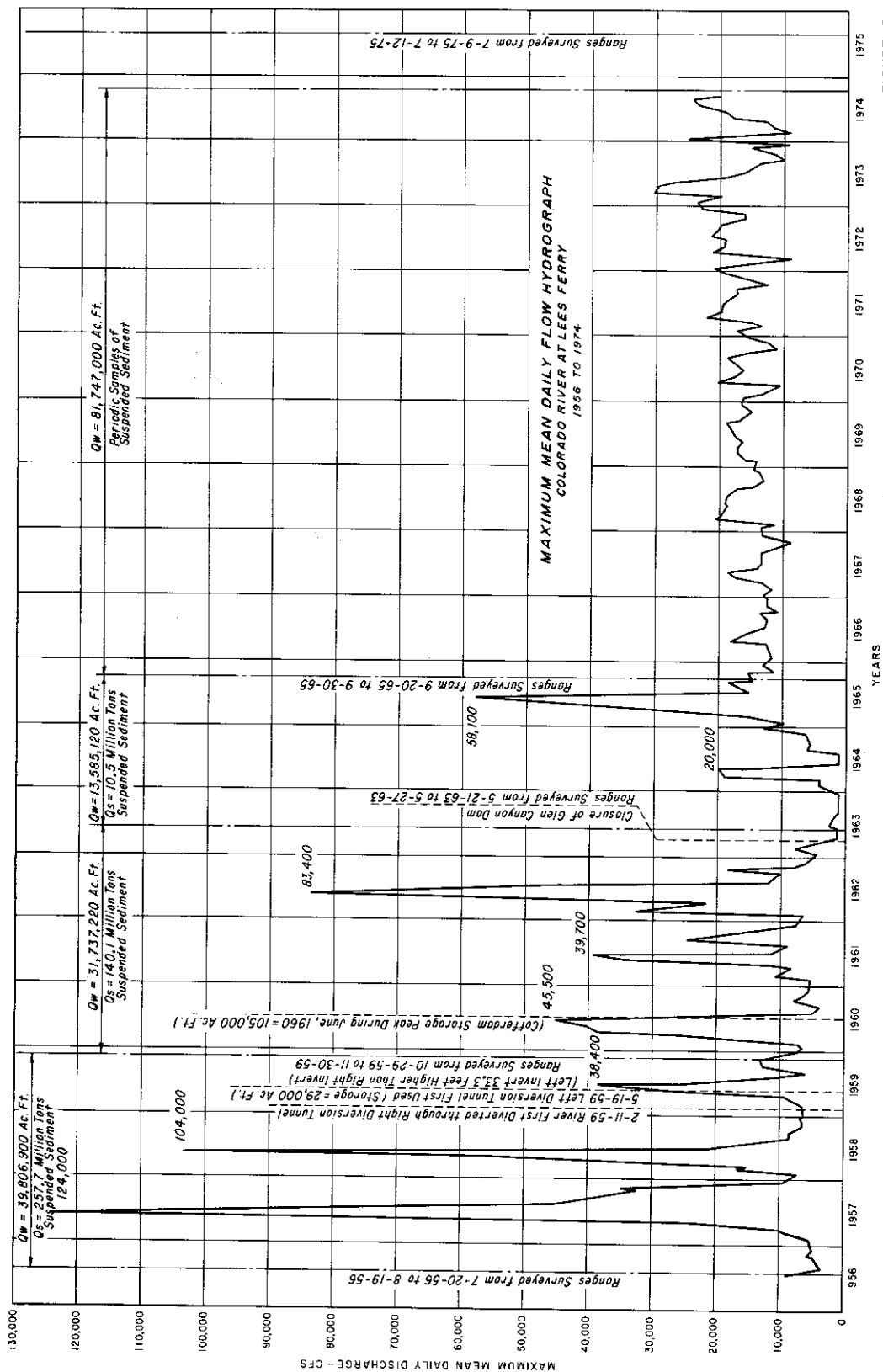


FIGURE 5

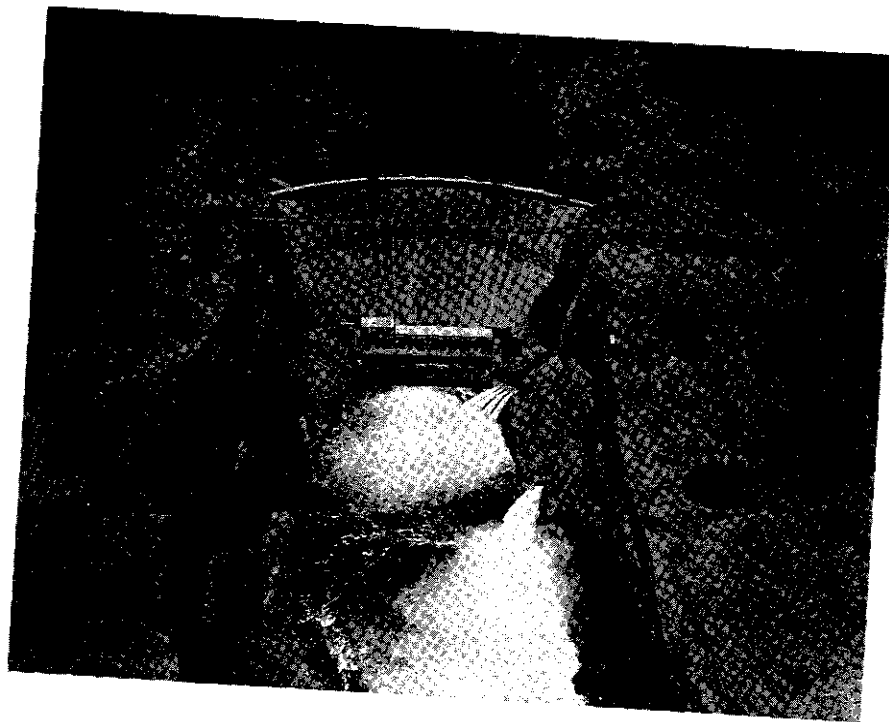


Figure 6 - Releases below Glen Canyon Dam on Colorado River in May 1965.

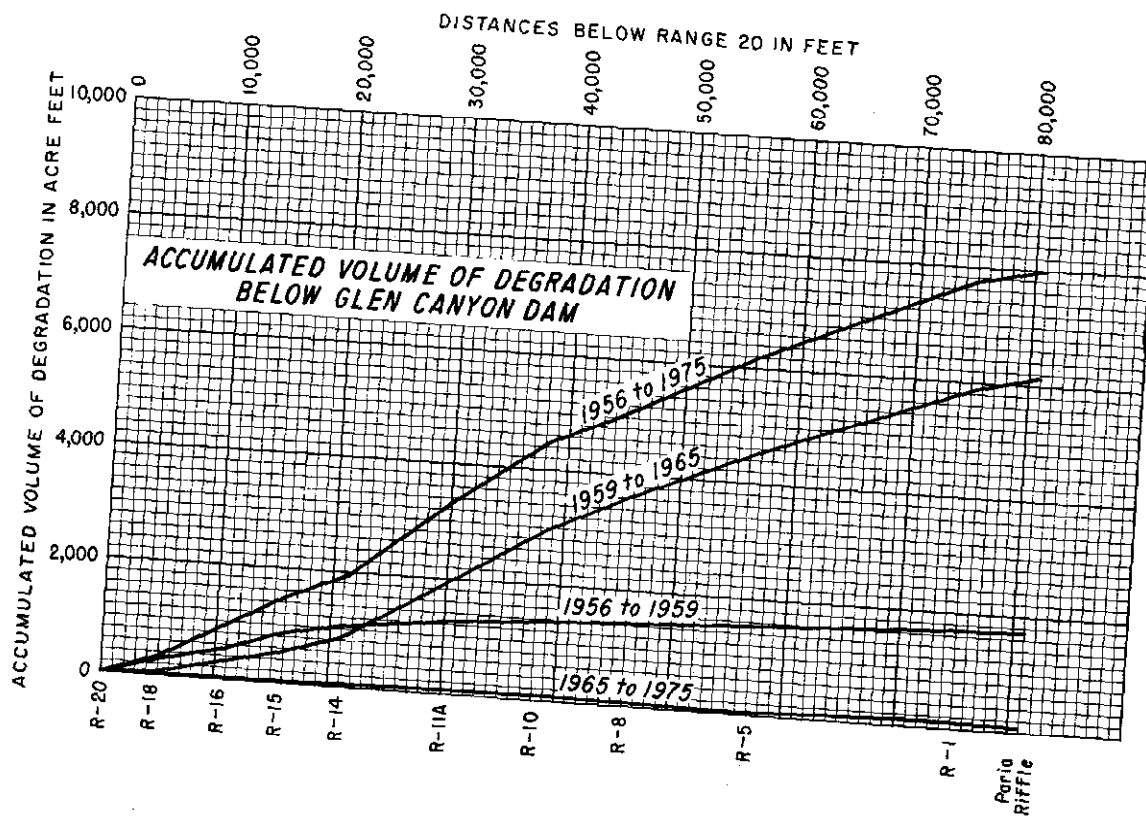


FIGURE 7

Degradation from 1963 to 1965 had now moved downstream from range 15 and was much greater in the lower 14-1/2 kilometers (9 miles) of the river immediately above Lees Ferry.

The resurvey of ranges in 1975, when compared to ranges surveyed in 1965, showed no significant degradation with only about 120,000 cubic meters (97 acre-feet) of material scoured from the river bottom in the 9.8-year period. The accumulated volumes of sediment degraded from the Colorado River channel above Lees Ferry between selected resurveys is shown in figure 7.

SAMPLES OF BOTTOM SEDIMENTS TAKEN IN 1966 AND 1975

In the 1966 observation and sampling program, three major controls (similar to that at range 20 (figure 8)) and six minor controls in the stream bottom were found where the armoring-size material had provided a positive control. The average-size analysis for the armoring materials from samples collected in 1966 is shown on figure 4 with a D_{50} of about 100 millimeters (4 inches). Observations made during the 1966 sampling indicate that the armoring layer was probably about 1 diameter in thickness, but underlying this layer was a mixture of about 20 percent sand, 75 percent gravel, and 5 percent cobbles up to 203 millimeters (8 inches). Most of the sand deposits overlying the gravels that were identified in the 1956 jet probing had been removed leaving an armoring layer of gravels and cobbles at nine locations controlling further degradation. There was still some evidence of medium to coarse sand-size material on the channel bottom above the gravel and cobble controls. The sands were more predominant in the lower reaches above the Paria Riffle.

Observations of channel conditions in July 1975 supported those made in 1966 with about 10 gravel and cobble bars providing positive streambed control similar to that shown in figure 9. At least eight of these controls were located in the upper 13 kilometers (8 miles). The bottom between the controls is predominantly gravel with very little sand. Some sandbanks were noted, but these are being held in check by an increase in vegetative growth. In the upstream reach there is considerable algae growth on both gravels and cobbles. In the downstream reach between controls there was more sand on the bottom and at range 1, the surveyed cross section showed a local sandbar to be moving with a D_{50} of about 0.37 millimeter.

Photographs of the typical armor layer found in 1975 are shown on figures 10 and 11 which were taken near range 20. The size of the armor layer with a D_{50} of 100 millimeters (4 inches) is shown on figure 4 and was almost identical to that found in 1966.

ARMORING ANALYSIS

The standard Bureau of Reclamation procedures used to evaluate the armoring material size (Strand, 1973) were applied to the reach below Glen Canyon Dam. In addition to the Competent Bottom Velocity, Meyer-Peter, Muller, and Schoklitsch equations used to determine the nontransportable beginning or zero transportable material to define an armoring size, the armoring analysis,

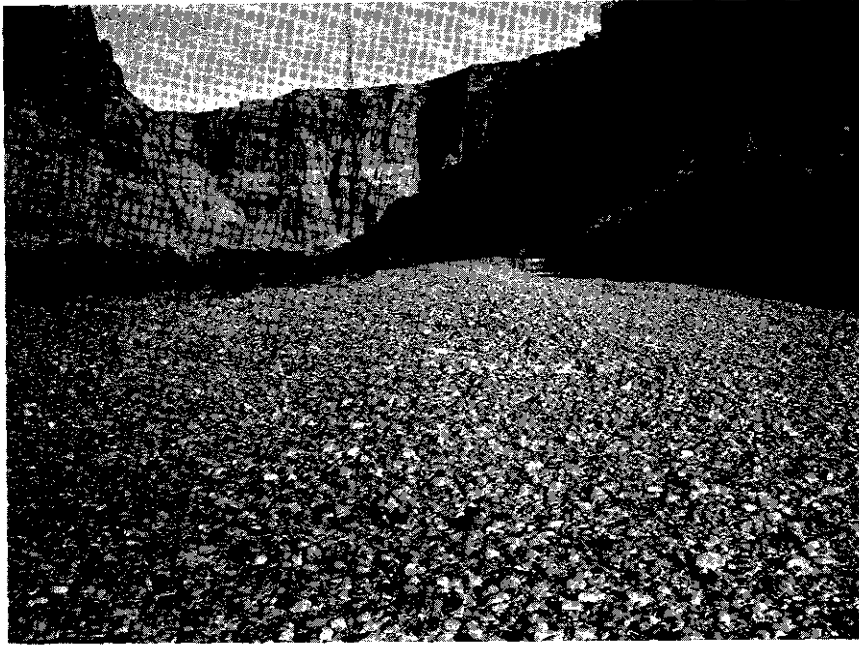


Figure 8 - Colorado River at a major channel control
by armoring near Range 20 in October 1966.



Figure 9 - Colorado River below Glen Canyon Dam
showing minor control by armoring near
Range 10 in July 1975.

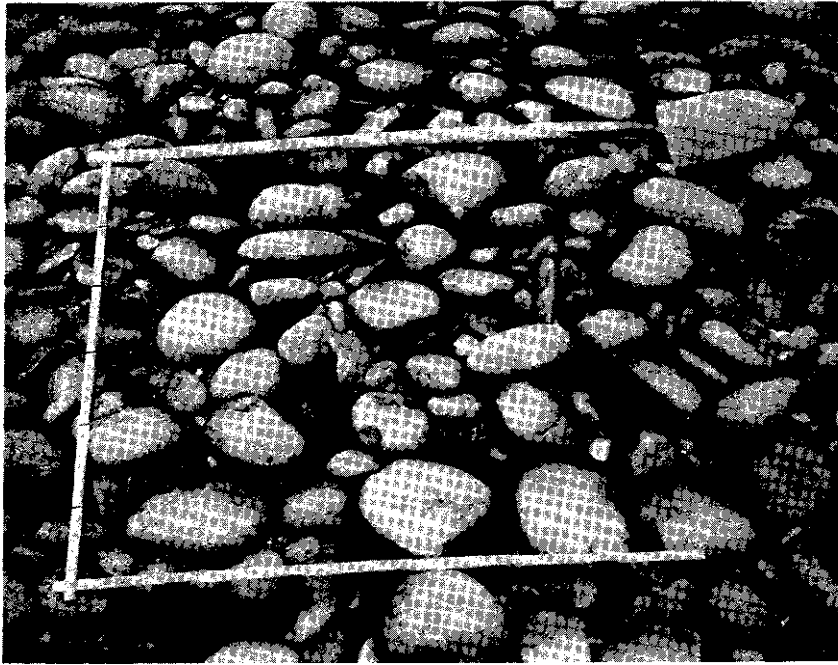


Figure 10 - Gravel-cobble size armoring near Range 15A on Colorado River below Glen Canyon Dam in July 1975.

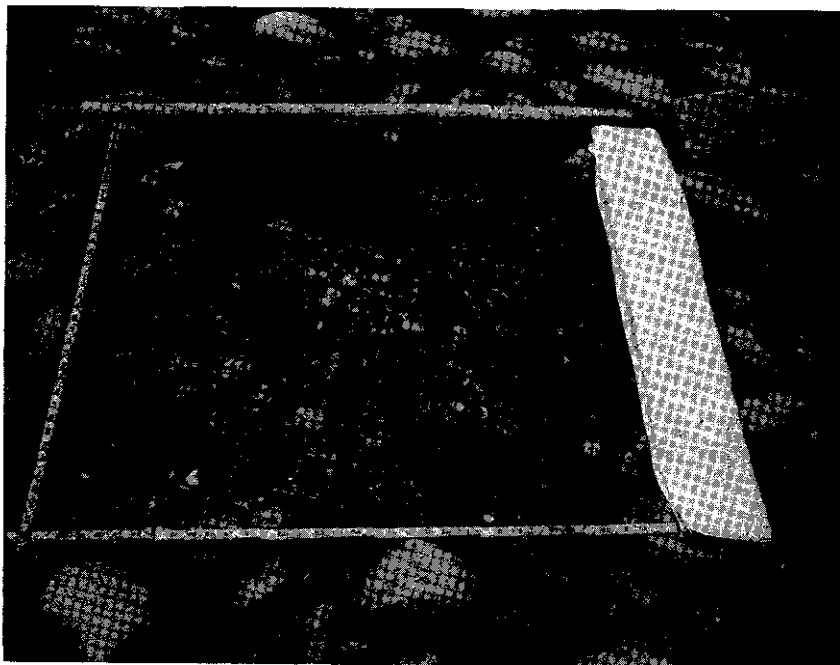


Figure 11 - Material underlying armoring layer near Range 15A on Colorado River below Glen Canyon Dam in July 1975.

(Little and Mayer, 1972) and the Velocity-Xi Adjustment to Einstein's total load transport (Pemberton, 1972) were tested. In this evaluation of armor the critical period for degradation was considered to be from beginning of construction until 1963 as shown on figure 5. The dominant discharge for this period as defined by the maximum mean daily flow hydrograph (figure 5) was assumed to approximate the 2-year flood peak frequency and was 1,840 cubic meters per second (65,000 cubic feet per second). The average channel hydraulics for this discharge together with the size analysis of materials assumed to be available for armor from the underlying gravel deposits in 1956 gave the following armoring sizes:

<u>Armoring Technique</u>	<u>Armoring Size (Millimeters)</u>
Competent Bottom Velocity (High Velocity Range)	76
Meyer-Peter, Muller	23
Schoklitsch	10
Little and Mayer	97*
Einstein Total Load (USBR Velocity-Xi Adjustment)	23

* Geometric mean diameter of armor mixture.

The results tabulated above were supported by the samples of armoring material as indicated by figure 4. The minimum size of armor shown for the 1975 samples is about 40 millimeters, while that shown for 1966 was about 10 millimeters. The geometric mean diameter of armor indicated by Little and Mayer of 97 millimeters is in good agreement with the 1966 and 1975 armoring samples.

CONCLUSIONS

Degradation below Glen Canyon Dam has been controlled by armoring of cobble-size materials with a D₅₀ of about 100 millimeters. There are presently about 10 of these gravel-cobble control bars in the 24-kilometer (15-mile) reach of the Colorado River from Glen Canyon Dam to Lees Ferry. The 1957 degradation study indicated that the control below range 15 and Paria Riffle would control extensive degradation. In the approximately 19 years since construction first started on the dam up until 1975, resurveys of ranges have supported most of the conclusions made in the 1957 study. The Paria Riffle, consisting of cobbles and boulders, has not changed appreciably since construction of Glen Canyon Dam.

An accelerated rate of degradation took place immediately below Glen Canyon Dam during the construction period from 1959 to 1963. The cofferdam located above the construction area provided a storage area for sediments, which in

combination with large releases of clearer water, resulted in the accelerated degradation. As anticipated, the degradation that first occurred below the dam has moved downstream. The resurveys of ranges have shown that about 9.87 million cubic meters (8,000 acre-feet) of the bottom sediment have scoured from the channel compared with the predicted value in the 1957 study of 8.26 million cubic meters (6,700 acre-feet). In comparing the 1975 profiles with the 1965 profiles, the channel has become quite stable with very little change in the upstream reaches and with only minor changes, such as the medium and coarser sand-size materials being moved into and through the lower reach of the Colorado River above Lees Ferry.

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CHANNEL CHANGES PRODUCED BY THE 1973 FLOOD
ON THE LOWER MISSISSIPPI RIVER

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ABSTRACT

The 1973 flood on the Lower Mississippi River offered a unique opportunity to observe the effects of a major flood on the morphology of the channel. A limited number of segments or reaches of river were chosen for a study of changes based on availability of previous hydrographic surveys and preflood color infrared photography. The Togo Island Bend to Middleground Island Reach was selected for discussion in this paper.

Water surface flow patterns were identified using color infrared photography and surface float measurements in conjunction with hydrographic surveys.

Field trips taken during the following low water season revealed definite changes in the shape and areal extent of both sand bars and islands. In most cases the islands were larger and had increased in elevation to near the top bank height of the main channel. Gravel deposits were present to a greater extent than in previous years. The gravel sizes observed were much larger than could be sampled with the sediment sampling equipment now used. The effects of the 1973 flood clearly indicate that large floods accelerate changes in channel shape and alter the characteristics of the alluvial deposits.

TOGO ISLAND BEND TO MIDDLEGROUND ISLAND REACH

The bend at Togo Island began to take its present position after a natural cutoff occurred in 1867. The size and shape of the bend has changed over the years, but its location has remained essentially permanent. A chute cutoff began developing through the point bar about 1969 and now conveys the larger percentage of flow during the entire year.

Middleground Island developed following Yucatan Cutoff in 1929. The island grew from sediment deposition in the cutoff channel located just below Togo Island Bend. The Yucatan Dike System was constructed in 1970 for the purpose of partial channel closure in order to divert as much of the lower stage flows as possible through the left channel.

DATA COLLECTION

Large scale color infrared photographs were taken on April 2, 1973, and used in conjunction with the hydrographic survey of April 25 - May 3, 1973. A comparison was made with preflood aerial photos of May 4, 1972, and the hydrographic survey of May 1-18, 1972. The surveys and over-flights are shown with respect to the stage hydrographs at the Vicksburg Gage located about 20 miles upstream (Figure 1).

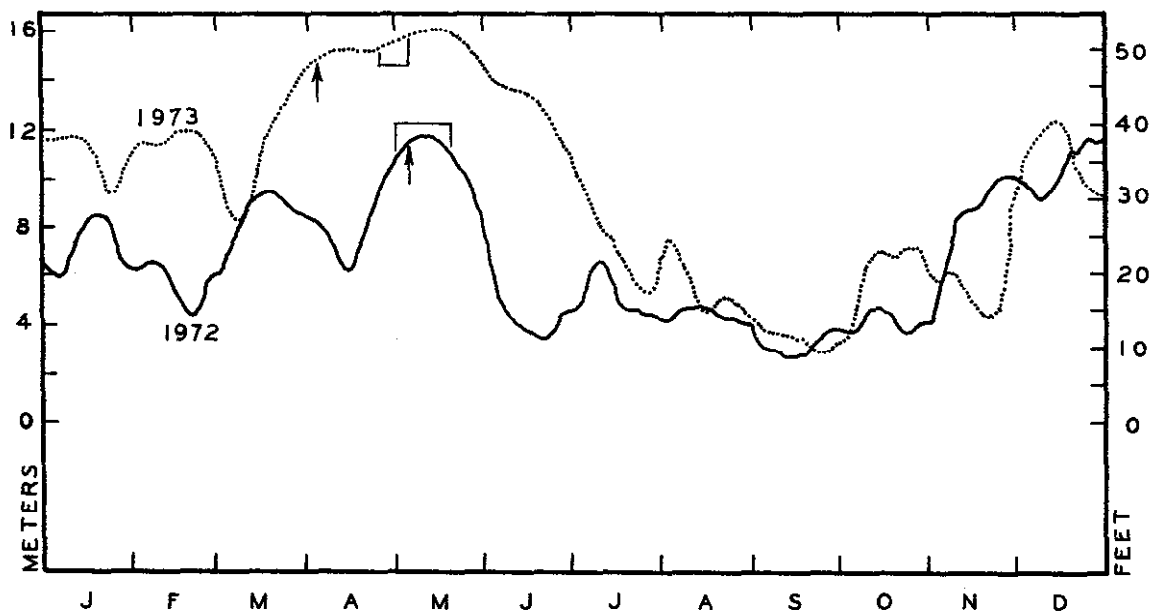


Figure 1. Stage hydrographs for the Vicksburg, Miss., Gage. The arrows denote time of overflights and brackets the time period of hydrographic surveys.

FIELD OBSERVATIONS

The most conspicuous changes had occurred at Middleground Island. Large tree trunks were on the upstream end of the island aligned in the direction of flow. The gravel deposits that extended over a large portion of the upstream end of the island were only as thick as the largest size fraction but completely covered the underlying sand (Figure 2). Based upon on-site inspection, the gravel sizes graded from a very fine size of 2 mm. (.078 in.) up through the maximum gravel size classification of 64 mm. (2.5 in.). Cobble sizes were found in abundance and ranged from 64 mm. up to 152 mm. (6 in.). At some locations,



Figure 2. Gravel layer on Middleground Island.
Scale is 152 mm. (6 in.)

small sand dunes had moved in over the gravel deposits. Probing the subsurface with a shovel confirmed the existence of the gravel layer covered by the overlying sand dunes. The largest size of material found on the surface was an agglomerate, approximately 229 mm. by 356 mm. (9 in. by 14 in.), containing large size gravel. Its origin, whether natural or artificial, was not determined. It was definitely deposited by the flood as it rested on top of several feet of sand accumulated above the preflood surface.

There were no surface deposits of gravel on the island in Togo Island Bend, but there was evidence that large sand waves had migrated over the surface during the higher flood stages. One sand wave extending laterally about half the distance across the island was about 1.2 to 1.5 m. (4 to 5 ft.) at its lee side (Figure 3).

WATER SURFACE FLOW PATTERNS AT TOGO ISLAND BEND

The color infrared photography identified the turbulence patterns by lines of foam and debris and by differences in color tone. Figures 4 and 5 show water surface turbulence patterns traced from the aerial photographs of Togo Island Bend.

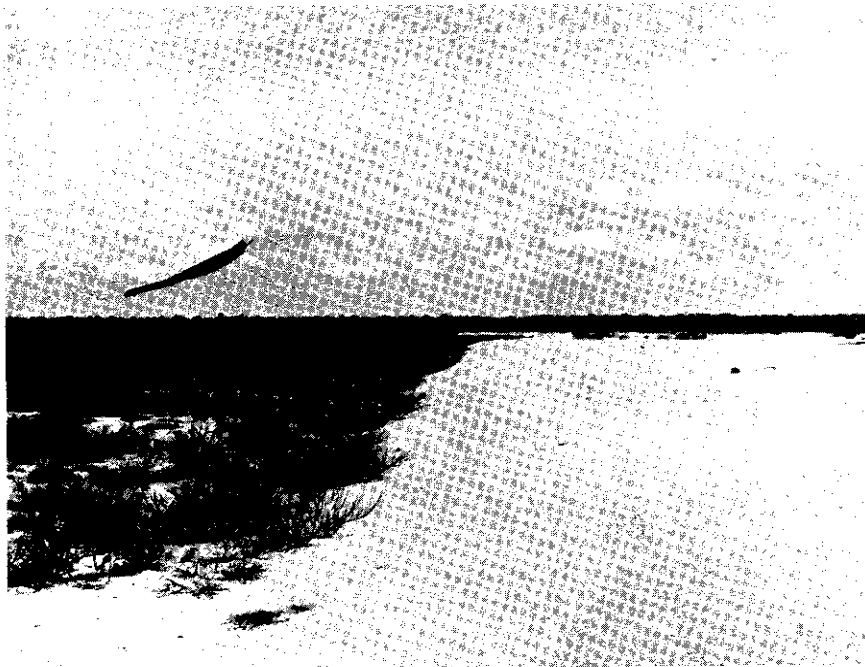


Figure 3. Sand wave at Togo Island Bend. View is in upstream direction.

In 1972, it was observed that, at the entrance to Togo Island Bend, the flow maintained a velocity of 3 m.p.s. (10 f.p.s.). As the channel cross section expanded, a separation zone formed along the right bank. The vortex trail extended well into the bendway and was diffused over the upstream end of the island. The float velocity measurements observed within the circulation pattern show a much slower velocity and a curvilinear path (Figure 4a). Flow was much slower over the upstream end of the island. Velocities slightly greater than 1.2 m.p.s. (4 f.p.s.) occurred laterally across the island toward the right bank and around the island crown, which was exposed above the water surface. The surface area not submerged was outlined by flow patterns showing vortex development along both sides.

During the 1973 flood, the only apparent flow occurred in the left channel. The right channel had almost a total absence of any measurable velocity. The separation zone began at the same point as before, but the circulation patterns were much larger and extended farther into the bendway (Figure 5). An average float velocity observed in a large vortex at a point near the shear zone was 1.0 m.p.s. (3.4 f.p.s.), but the float pattern shows a gradual increase in velocity occurring in the downstream direction as it nears the main flow (Figure 5a). Other float observations in the circulation pattern show no measurable velocities.

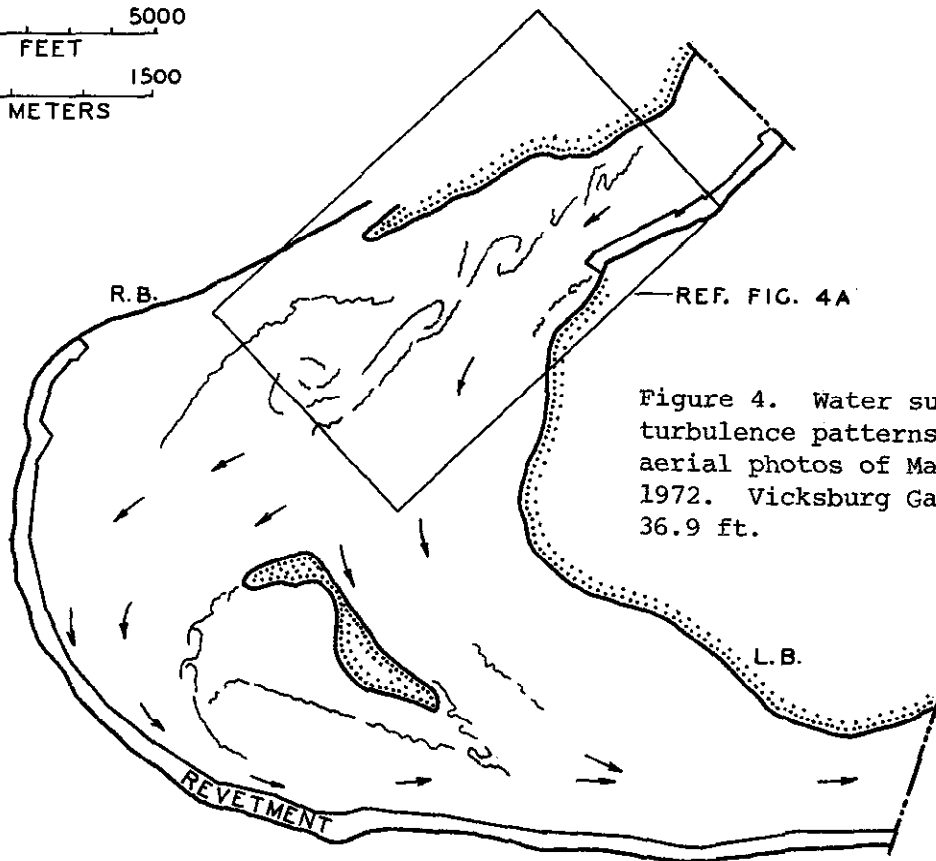
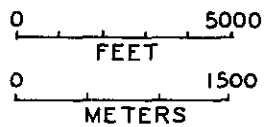


Figure 4. Water surface turbulence patterns from aerial photos of May 4, 1972. Vicksburg Gage at 36.9 ft.

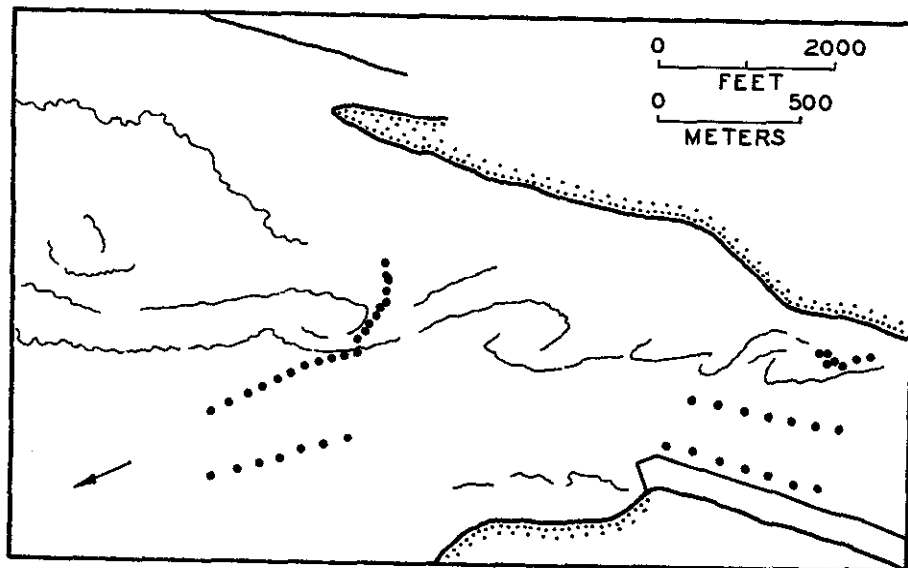


Figure 4A. Surface floats positioned at 30 second intervals.

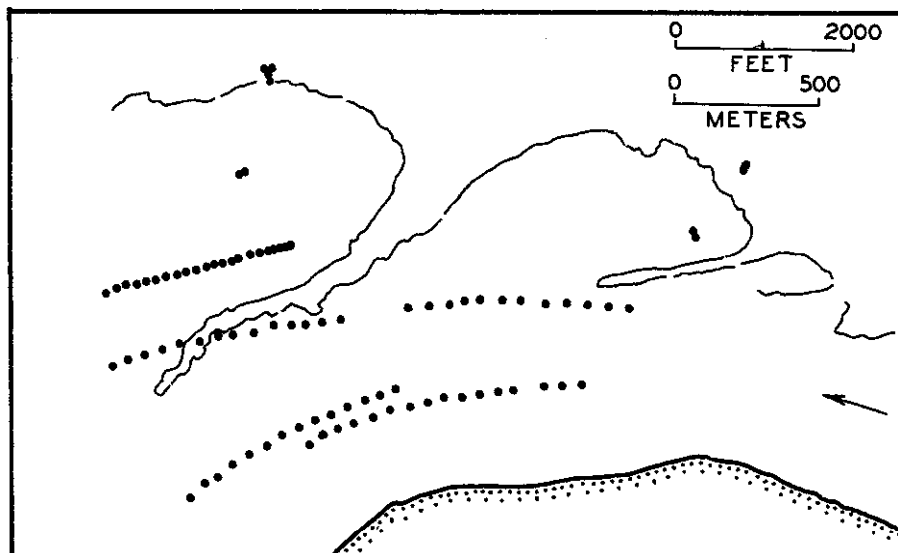
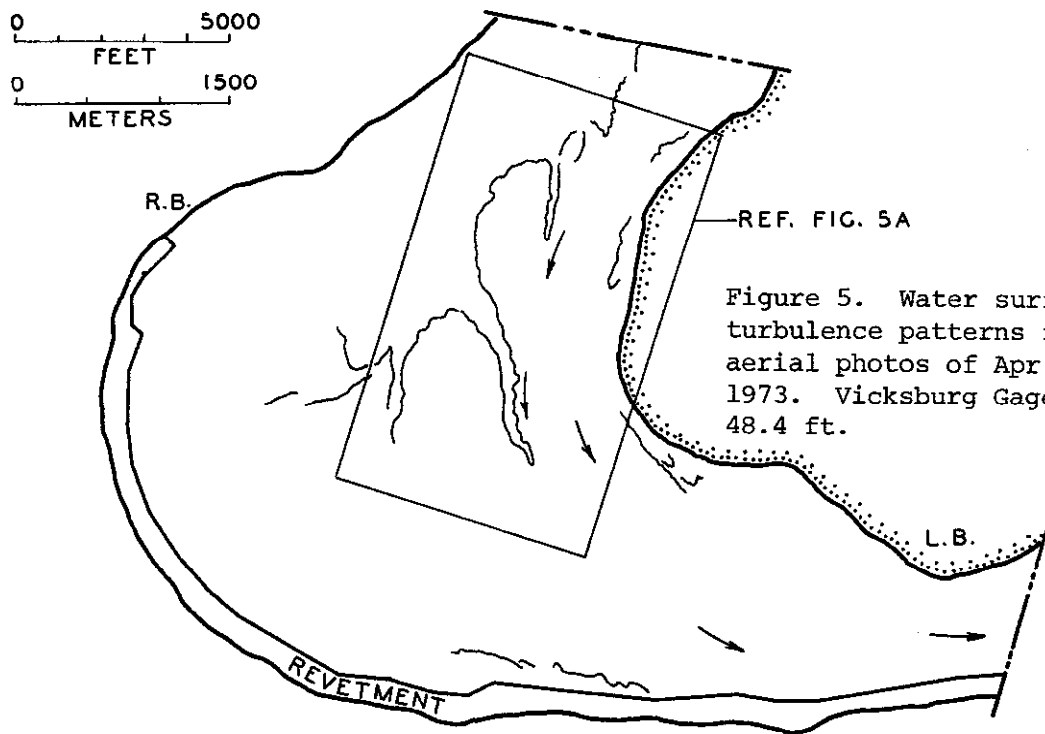


Figure 5A. Surface floats positioned at 30 second intervals.

WATER SURFACE FLOW PATTERNS AT MIDDLEGROUND ISLAND

As the flow approaches the Yucatan Dike System, there is a shift in velocity distribution across the channel for the 1972 and 1973 surveys (Figure 6). At bankfull stage in 1972, the maximum velocity occurred between the outshore dike terminals and the left bankline. Velocities measured over the dike system were roughly one-half the channel velocities.

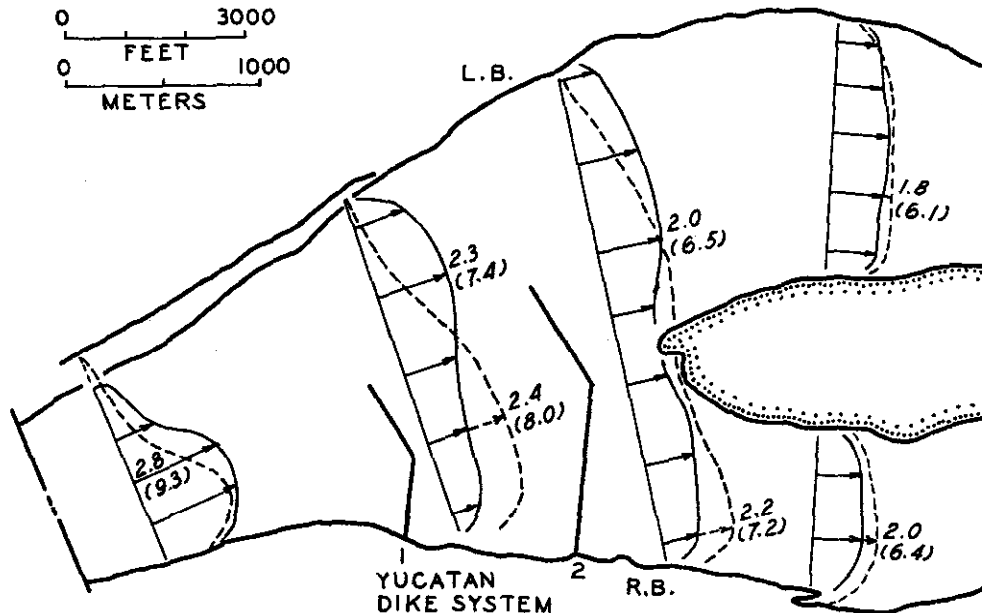


Figure 6. Velocity profiles for 1972 and 1973. Maximum velocities in m.p.s. (f.p.s. in parentheses).

A foam line recorded by the color infrared film in May 1972 indicates the beginning of a separation zone and the formation of vortices just below dike no. 1 (Figure 7). The zone appears fully developed opposite dike no. 2 but diffuses about 1370 m. (4500 ft.) downstream of no. 2. The indication is that the dikes have caused or else contributed to the development of the separation zone. If the dikes increase the flow velocity to a value greater than that which would normally occur, then the flow at the dike terminals would begin to take on the characteristics of a submerged water jet. This would cause an abnormal increase in discharge outshore of the dikes. It seems probable that this could amplify any tendency to form a separation zone where a more uniform velocity distribution would discourage it.

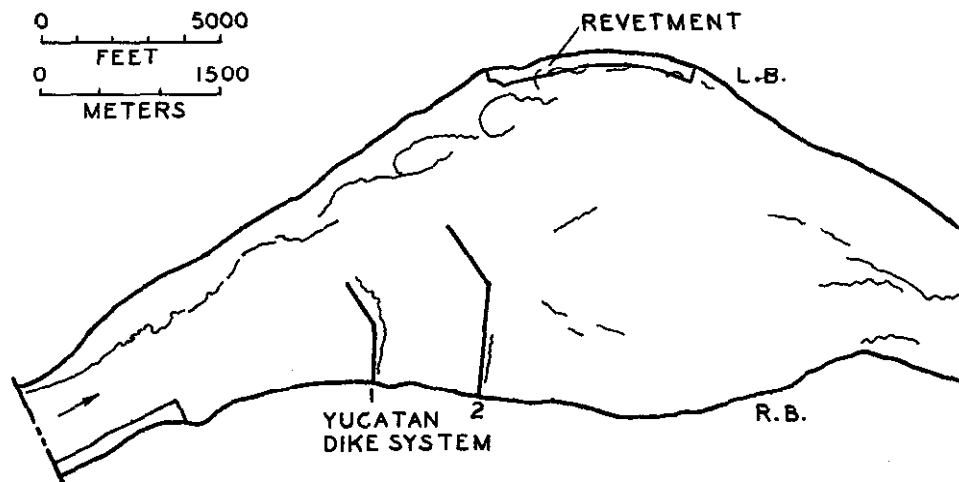


Figure 7. Water surface turbulence patterns from aerial photos of May 4, 1973.

During the 1973 flood, the velocity distribution was the reversal of 1972 with the maximum velocity of 2.4 m.p.s. (8.0 f.p.s.) occurring directly over the dikes (Figure 6). The velocities diminished over the cross section to less than 0.3 m.p.s. (1.0 f.p.s.) near the left bank. After the flow divided around the island the velocity distributions in both channels were very similar with the 1973 observations being slightly greater.

In both the 1972 and 1973 photography there was no indication of separation zones around the crown of Middleground Island except for a very minor wake disturbance downstream. This can possibly be attributed to the more equal velocities in both channels and the streamlined shape of the island below the dike field.

CORRELATION OF FLOW PATTERNS AND GRAVEL DEPOSITS

The location of maximum velocity would determine where the maximum sediment discharge occurs and where the largest sediment sizes would be transported. There had been no gravel deposits present in any appreciable size or quantity on Middleground Island following the 1972 bankfull stage as the maximum velocities had occurred in the left channel in 1972. In 1973 the flow headed directly into the island with velocities decreasing as the higher island elevations were reached. The gravel deposits left by the 1973 flood began just below dike no. 2 and gradually diminished in size and quantity toward the island tree line.

The 1973 aerial photography did show small wakes downstream of the treeline clumps. Field inspection of the clumps found streamlined shaped gravel and sand deposits mounded up in the wake area and overlying the preflood surface. Although the flow depth was considerably less and velocities reduced, enough transport capacity was available to carry medium sized gravel on to the island crown. It is assumed that the gravel deposit at the treeline would have traveled farther up on the island except for the trees.

CHANGES IN CHANNEL TOPOGRAPHY

The zero datum contour shown in Figures 8 and 9 is a reference plane derived from the average low water plane (ALWP). Changes which occur in the zero contour are a good indication of the trends in erosion and deposition.

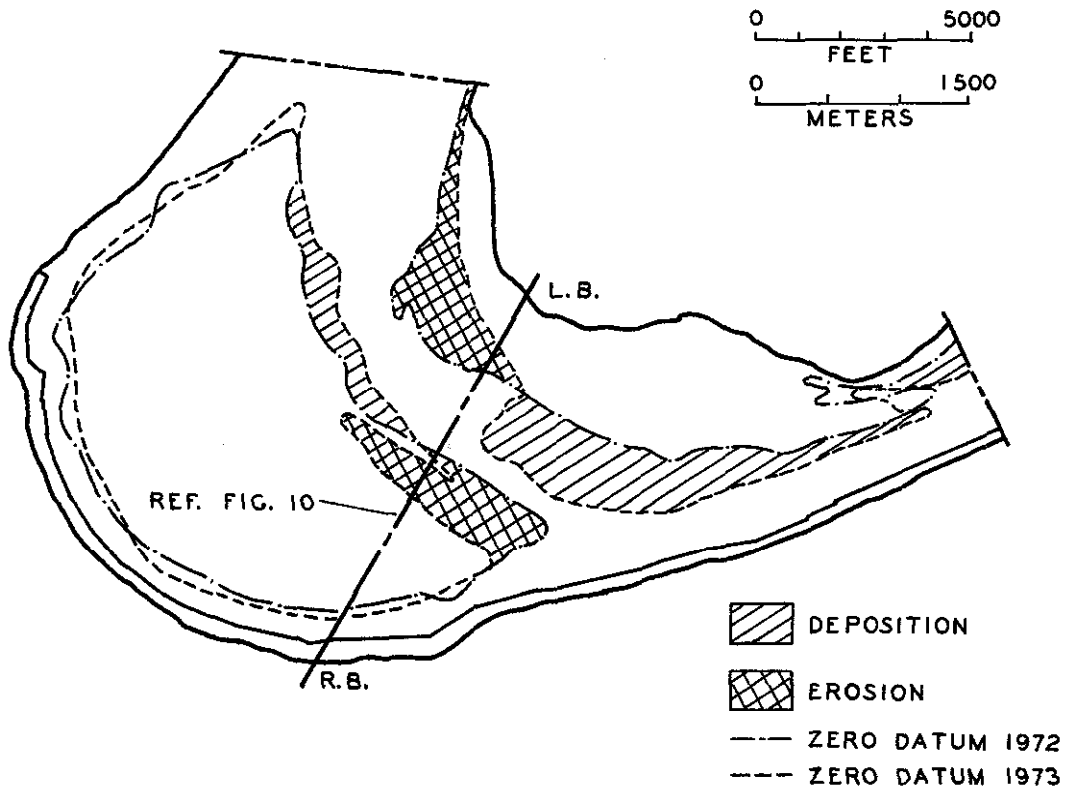


Figure 8. Togo Island Bend

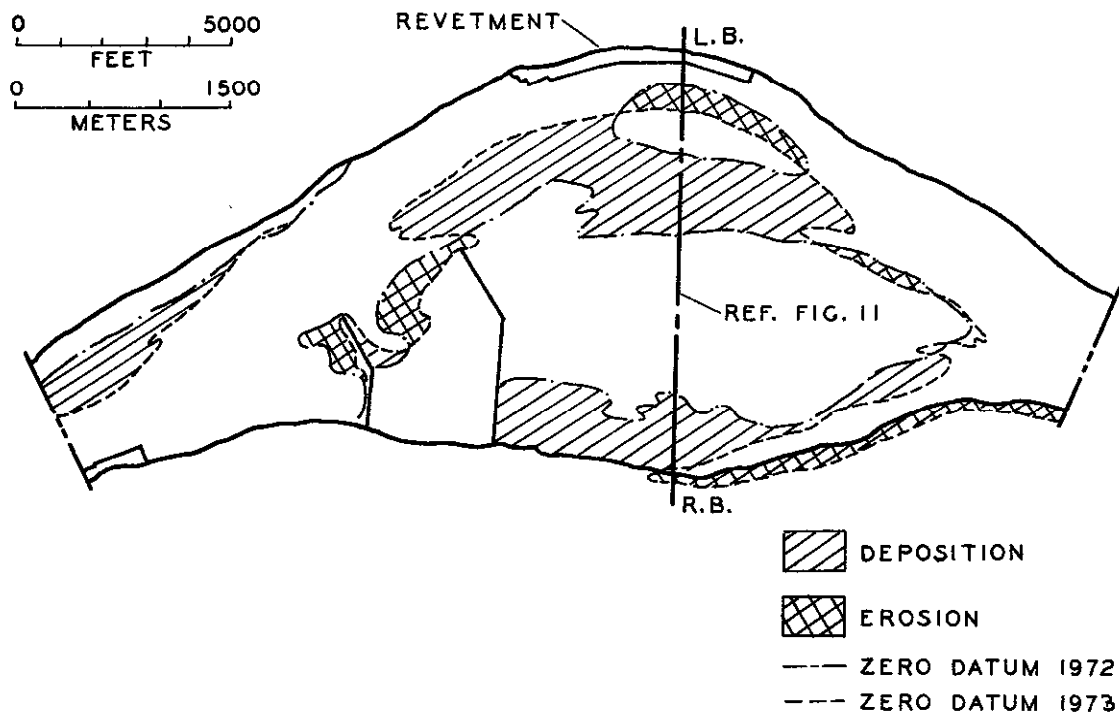


Figure 9. Middleground Island

The changes at Togo Island Bend show a migration of the left channel toward the right bank and an overall enlargement of the channel (Figure 10). If this trend continues, it will eventually lead to a recapture of the old bendway.

The Middleground Island location has been an area of sediment deposition since Yucatan Cutoff was fully developed about 1934. The smaller submerged sand bar in the left channel (1972 survey) was the result of dredged material accumulation from a dredge cut made along the left side of Middleground Island. The 1973 flood deposited enough material in the dredge cut to join the sand bar to the island and establish a topography above the zero contour (Figure 11). This appears to be due to the shifting of the maximum flow velocity toward a direct alignment with the head of Middleground Island. With the island in the path of the maximum flow, the velocity distribution in the left and right channels changed very little and showed only a slight increase in magnitude. The overall change at Middleground was that the island grew laterally in both directions contributing to channel deterioration. The only deepening at this section was along the toe of the right bank revetment.

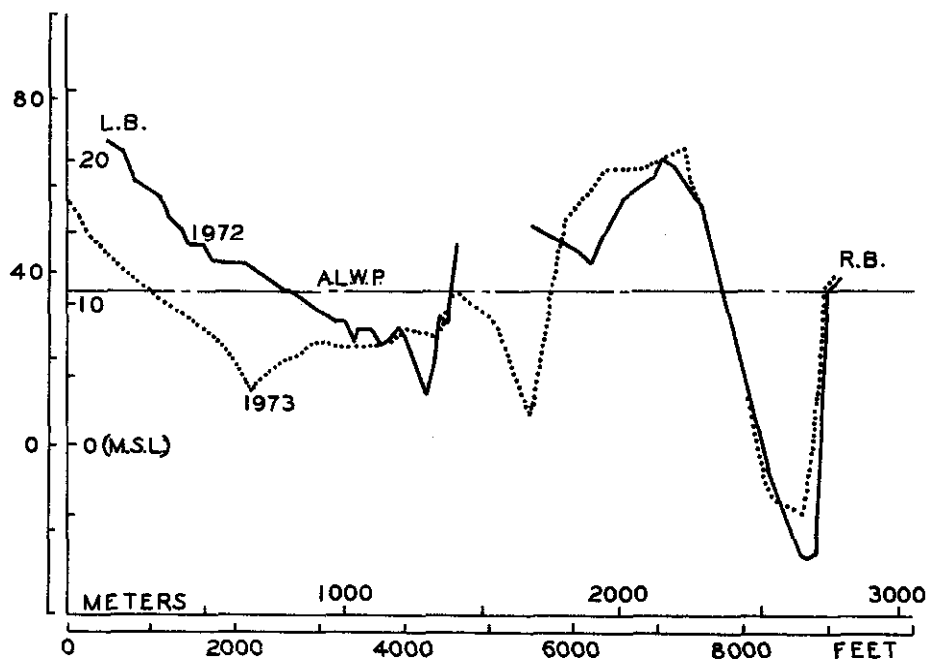


Figure 10. Cross section profile of Togo Island Bend. Vertical scale Zero is mean sea level.

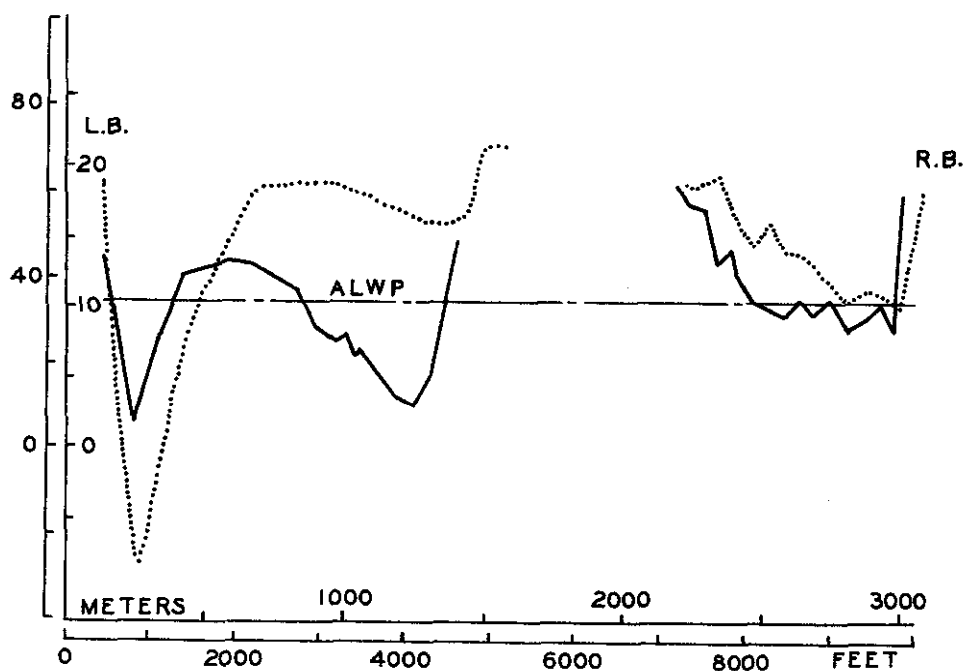


Figure 11. Cross section profile of Middleground Island. Vertical scale Zero is mean sea level.

CONCLUSIONS

Field observations in this reach showed that the 1973 flood transported much larger size bed material than the current sampling apparatus will retrieve. The bucket opening on the bed material sampler is 76 mm. (3 in.) wide and samples the top 51 mm. (2 in.) of channel bed. The maximum size possible in a bed material sample would probably be on the order of 38 mm. (1.5 in.). Therefore, bed material samples obtained in gravel deposit areas would show a gradation in fraction size over a limited range and would not be representative. This would seem to hold true for the bankfull stages since the magnitude of the velocity did not vary much but only the location of the higher velocity.

The suspended sediment sampling apparatus now in use has an opening of 4.76 mm. (.1875 in.) where the maximum size sampled would be much less. The gravel deposited at the tree line indicates that a wide range of material greater than 4.76 mm. could possibly be transported in suspension above the channel bed.

Since sediment discharge depends directly on velocity and is very sensitive to slight changes, suspended sediment sampling locations could be chosen to avoid direct contact with vortex trails, if this is desired, and concentrated in the areas of more uniform flow. Discrepancies in earlier samples may be explained by apparent locations of separation zones.

Cross sections for discharge measurement could be chosen with the least amount of turbulence. The discharge range may require a different location for low and high stages due to the absence or presence of separation zones.

Training structures are generally located where they will assist in developing the low water navigation channel. However, the channel forming discharges and sediment loads seem to be associated with the higher stages. The implication regarding dike location and design is that a proper evaluation of present training structure locations should be made considering turbulence patterns, flow directions, velocities and sediment deposition areas. From this study, future locations could be chosen where dike systems would be more effective at the higher flows.

MODEL STUDY OF RIVERBED MATERIAL
IN CANYON FERRY DAM SPILLWAY STILLING BASIN

By

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ABSTRACT

Hydraulic model studies were made to determine the cause and recommend a solution for the deposit and movement of riverbed material into the Canyon Ferry Dam spillway stilling basin. Tests indicated that movement of riverbed material results from operation of the river outlet works at discharges greater than 3,000 ft³/s (85 m³/s). Several solutions to the problem are suggested, including a limitation on operation of the river outlet to 3,000 ft³/s (85 m³/s). Studies were conducted to determine the effectiveness of the spillway discharge in clearing various amounts of deposited material from the basin. As a result of these studies, a recommended spillway discharge was applied to the Canyon Ferry stilling basin to clean 900 yd³ (688 m³) of riverbed material from the basin. Soundings of the basin were taken immediately after the suggested release and confirmed the model results.

INTRODUCTION

Canyon Ferry Dam is located 17 miles (27 kilometers) northeast of Helena, Montana, on the Missouri River. The dam is a concrete gravity-type structure approximately 1,000 feet (305 meters) in length with a maximum height of 225 feet (68.6 meters) above the foundation. The maximum water surface elevation in the reservoir is 3800 feet (1158 meters). The powerplant on the right side of the dam is rated at 50,000 kilowatts. The dam was constructed in the period 1949-1954.

Construction of the cofferdam for the Helena Valley Pumping Plant, which is immediately downstream from the spillway stilling basin, was started in May 1957. By June the cofferdam was three-fourths complete when large releases required to pass reservoir inflows washed away a major part of the earth cofferdam. The cofferdam was reconstructed and severe cutting and removal of sand and gravel occurred again during the 1958 releases.

Soundings have been taken of the stilling basin and of the river channel immediately downstream from the basin periodically since 1960. The soundings indicate that a considerable amount of riverbed material was carried into the stilling basin. In 1972 over 17,000 yd³ (13,000 m³) of material were removed from the basin. The material was carried to a disposal site approximately 1 mile (1.6 kilometers) downstream from the dam. Soundings

taken in July 1973 indicated that approximately 900 yd³ (688 m³) of riverbed material were again deposited in the basin.

In 1974 hydraulic model studies were requested to:

- a. Determine the cause (what release method) for movement of riverbed material into the spillway stilling basin,
- b. Determine if the existing riverbed, downstream from the basin, had stabilized, and
- c. If not, what would be required to stabilize the riverbed or otherwise prevent the riverbed material from entering the spillway stilling basin.

THE MODEL

Description

The model, constructed to a scale of 1:48, included 190 feet (58 meters) of the upstream reservoir, the dam, powerplant, Helena Valley Pumping Plant, and 450 feet (137 meters) of the downstream river channel, figure 1. To properly model the various releases from the reservoir, provision for controlled releases from the spillway, river outlets, powerplant, and pumping plant were included in the model. Each control was calibrated before the test program started. A tailgate assembly and sand trap were used to control the downstream tailwater elevation and collect eroded sand and gravel. Water was supplied to the model through the permanent laboratory system. Discharges were measured by one of a bank of venturi meters installed in the laboratory.

Scale Relations

To express the mathematical relationship between the hydraulic quantities of the model and the prototype, the Froude Law of model similitude was applied. The scale relations based on the Froude Law can be expressed in terms of L_r , the length ratio, as shown below:

<u>Dimension</u>	<u>Scale ratio</u>
Length	$L_r = 1:48$
Area	$(L_r)^2 = 1:2,304$
Velocity	$(L_r)^{1/2} = 1:6.93$
Discharge	$(L_r)^{5/2} = 1:15,963$
Time	$(L_r)^{1/2} = 1:6.93$

The movement of riverbed materials was of special interest in this study. To properly represent the prototype conditions in the model, determination of the size, specific gravity, and in turn the settling velocity of the model riverbed material was very important.

Using the Froude Model Law, the settling velocity ratio of the model to prototype riverbed material was $(L_r)^{1/2}$ or 1:6.93. Figure 2 relates the settling velocity of the model and prototype riverbed material. The top 30 percent of the model material is very close to the scaled settling velocity of the prototype material. The lower 70 percent of the model material is lighter than the prototype material.

Verification

The scaling technique used to model the riverbed material indicated that the model sand selected for the study adequately represented the prototype sands and gravels. To verify the Canyon Ferry model, the August 1970 sounding of the prototype stilling basin and downstream river channel, figure 3, was modeled. The significant river outlet and spillway releases were applied to the model on a compressed time scale representing the 1971 releases from the prototype test 5, see table 1. A major portion of the 1971 releases involved the river outlet works. The results of the test were compared with the September 1971 sounding of the prototype stilling basin and downstream river channel. Although the test results did not fully duplicate the 1971 sounding, the configuration of the relocated material in the model was similar to that in the prototype. In both the model and prototype, the 1970 deposition moved away from the retaining walls and upstream onto the sloping apron of the basin.

The model tests included a study of spillway releases required to clear approximately 900 yd³ (688 m³) of sand, gravel, and rock fragments from the basin. The test predicted that a spill of 28,200 ft³/s (799 m³/s) for approximately 3 hours would sweep the basin clear of this riverbed material. Shortly after these model tests, the suggested release was made at Canyon Ferry Dam and soundings taken immediately after the spill verified this model test.

THE INVESTIGATION

Effect of Flow Release Methods on Riverbed Stability

Release of surplus water through the Canyon Ferry Dam spillway and river outlets has resulted in the deposit of riverbed material in the spillway stilling basin. The movement of this material has eroded the concrete floor of the basin. The areas of greatest concrete erosion, up to 1.5 feet (0.46 meter), have occurred on the sloping apron of the stilling basin, figure 3. The model was tested to determine the effect of flow release methods on the movement and deposition of riverbed material.

River outlet works. - The river outlet works consist of four 96-inch (218-centimeter) diameter, horseshoe-shaped conduits placed horizontally through the spillway section, which exit on the face of the spillway chute at an invert elevation of 3649.91 feet (1112.5 meters). Each conduit has a 77-inch (195.6-centimeter) high-pressure regulating gate. The design discharge for the four river outlets is 9,500 ft³/s (269 m³/s). Although the outlet conduits are symmetrical with the stilling basin center wall, they are not centered in the stilling basin bays, figure 3. A series of tests 1, 7, 8, 9, and 14, with releases of 9,500, 4,000, 5,000, 6,000, and 3,000 ft³/s (269, 113, 142, 170, and 85 m³/s), respectively, was conducted for time intervals representing 43 hours each in the prototype. Figures 4A and 4B illustrate the downstream erosion and deposit of riverbed material in the basin after tests 1 and 14.

Erosion of the downstream river channel and deposition of the eroded material on the sloping apron of the stilling basin increased as the river outlet discharge increased from 3,000 to 9,500 ft³/s (85 to 269 m³/s). The test results clearly indicate that operation of the river outlets can carry large amounts of riverbed material into the stilling basin. River outlet releases limited to 3,000 ft³/s (85 m³/s) or less result in very little movement of riverbed material into the stilling basin, figure 4B.

Velocity measurements in the model determined the direction of flow (in or out) and the velocity of three half sections in the stilling basin for the 9,500 ft³/s (269 m³/s) river outlet release. As indicated earlier by the erosion patterns, a strong undercurrent moves upstream into the basin. At the end sill section, station 4+10, the core of the upstream current lies on the sill approximately 75 feet (23 meters) from the training wall. The core rises from elevation 3605 feet (1099 meters) at the sill to 3623 feet (1104 meters) at station 2+74. The core velocity of the undercurrent increases from 5 ft/s (1.5 m/s) at the sill to 7.5 ft/s (2.3 m/s) at station 2+74.

The jet leaving the river outlet conduit stays in the upper 20 feet (6.1 meters) of the basin depth and does not penetrate to the floor of the basin. A large longitudinal eddy is established in the vertical plane, providing the means for carrying riverbed material into the basin.

Test 13 determined the effect of asymmetrical releases from the river outlets. The design capacity of 2,375 ft³/s (67 m³/s) was released through outlets No. 3 and 4 for a total discharge of 4,750 ft³/s (135 m³/s). The erosion is more severe near the pumping plant with the asymmetrical operation. Although a comparable amount of material appeared to be carried into the basin on the left side, it was not all carried onto the sloping apron as in test 1. Approximately one-third remained in the downstream portion of the basin.

Test 15 compared the operation of outlets No. 1 and 4 releasing a total discharge of 4,750 ft³/s (135 m³/s) with test 14, releasing a discharge of 3,000 ft³/s (85 m³/s) through four outlets. Operating all four

outlets at 3,000 ft³/s (85 m³/s) produced less erosion and subsequent deposition than operating the two outside outlets at 4,750 ft³/s. These tests indicate that all four river outlets should be operated uniformly to achieve the best flow distribution possible.

Spillway. - Tests 24 and 25, with spillway releases of 4,100 and 6,600 ft³/s (116 and 187 m³/s), respectively, were conducted to determine if low spillway releases would carry riverbed material into the basin. There was no significant movement of material into the basin at low spillway releases; however, material initially present in the basin will be exposed to secondary currents and will continue to erode the concrete floor when spillway releases are not large enough to sweep the basin clean.

Powerplant and Helena Valley Pumping Plant. - Test 6A determined the effect of powerplant operation on the movement of riverbed material near and in the stilling basin. Before the start of the test some very fine material was observed on the stilling basin floor. The powerplant discharged 6,000 ft³/s (170 m³/s) for approximately 31 hours (prototype time scale). There was no movement of the fine material initially present on the floor of the basin over this time span. Test 6B was an extension of test 6A with the powerplant discharging 6,000 ft³/s (170 m³/s) and the Helena Valley Pumping Plant releasing 463 ft³/s (13 m³/s) into the downstream channel for a time span representing 10 hours in the prototype. Again, there was no indication of any movement of fine material on the stilling basin floor. A local scour hole and buildup occurred in front of the turbine outlet for the Helena Valley Pumping Plant. However, this local scour phenomena did not affect the movement of riverbed material in or near the stilling basin.

Combined river outlet and spillway releases. - Model tests indicated that the movement of riverbed material can be controlled by limiting the four river outlets to a total release of 3,000 ft³/s (85 m³/s). When larger releases are required, the river outlet works should be closed and all releases made over the spillway.

With the recent interest in the effect of gas supersaturation on fish life in the Columbia River, spillways with relatively deep stilling basins have become suspect. Water released over the spillway carries large quantities of air deep into the stilling basin. The hydrostatic pressure in the basin forces gas into solution, resulting in supersaturated water. Fish swimming in these waters take in dissolved gases through their gills and in turn these gases are transported into the body tissue by the blood stream. Gas bubble disease results when the fish swim into waters of lower pressure where the dissolved gas returns to its gaseous state.

The Canyon Ferry River outlets discharged across the water surface of the stilling basin in contrast to the deep plunging-type discharge of the spillway. With respect to supersaturated water, the Canyon Ferry

River outlets provide a more acceptable release method than the spillway.

Simultaneous operation of the two outside river outlets (Nos. 1 and 4) and the two center spillway gates (Nos. 2 and 3) should result in less gas supersaturation than spillway-only operation and also will result in less movement of riverbed material into the basin than river-outlets-only operation. Tests 34 and 38 were conducted releasing 4,750 ft³/s (135 m³/s) through the two outside river outlet conduits and 4,750 (test 34) and 10,000 ft³/s (283 m³/s) (test 38), through the two center spillway gates. The simultaneous operation was successful in controlling the movement of riverbed material into the basin.

Severe erosion of the riverbed occurred with the 10,000 ft³/s (283 m³/s) release through the two center spillway gates. Periodic soundings immediately downstream from the basin would allow the project to monitor the extent of erosion of the bedrock in this area.

Riverbed Stability

The term "riverbed stability," as used in this paper, will refer to noticeable movement of riverbed material with time and particularly in reference to material moving upstream into the stilling basin.

One objective of the study was to determine if the river channel downstream from the spillway stilling basin was stable. Tests 1, 7, 8, 9, and 14 indicated that the model riverbed was not stable when the river outlets released flows larger than 3,000 ft³/s (85 m³/s). Therefore, studies were conducted to determine what would be required to make the riverbed stable.

Natural channel. - Plywood was placed in the model to represent the location of bedrock in the prototype river channel. Three tests were conducted releasing 9,500 ft³/s (269 m³/s) for a time representing 42 hours in the prototype. After each test, riverbed material was removed from the stilling basin, measured, and not returned to the model. The material carried into the basin was 1,300, 700, and 530 yd³ (994, 535, and 405 m³) for tests 20, 21, and 22, respectively.

The tests indicated that the riverbed downstream from the stilling basin will stabilize in time with repeated operation of the river outlets and subsequent removal of the riverbed material carried into the basin. However, this is not a practical solution because of continued abrasion damage and high costs for material removal. As an alternative the downstream river bottom could be artificially stabilized with concrete grout or bituminous grout.

Modification. - A series of three tests (29, 30, and 31) was conducted to determine the distance downstream from the basin end sill which

would need to be cleared to bedrock to eliminate the movement of riverbed material into the basin. For each test the riverbed was cleared of material to the simulated bedrock, the full width of the basin extending 10 feet (3 meters) beyond the right training wall. The lengths of riverbed cleared to bedrock downstream from the basin end sill were 50, 75, and 100 feet (15, 23, and 30 meters) for tests 29, 30, and 31, respectively. Downstream from the cleared area, the invert sloped upward on a 4 to 1 slope to the existing riverbed. For each test the discharge was 9,500 ft³/s (269 m³/s) through the river outlets and 6,000 ft³/s (170 m³/s) from the powerplant, for a time period representing 43 hours in the prototype. These tests indicated that the river bottom should be cleared to bedrock for a distance of 100 feet (30 meters) downstream from the stilling basin end sill.

Studies conducted to determine the height that would be required of a wall on top of the end sill to prevent movement of riverbed material into stilling basin indicated that a 12-foot (3.7 meters) high wall would be needed. There are some obvious disadvantages to the end sill wall modification. The wall would make it very difficult to sweep the basin clear by spilling action and also might have a detrimental effect on the stilling action of the basin at larger spillway discharges.

Spill Required to Clean Stilling Basins

Since spillway or river outlet releases are necessary approximately 6 out of every 7 years at Canyon Ferry Dam, the idea of sweeping future deposits of material out of the basin by spillway releases was considered. Tests 17, 18, and 19 were conducted to determine the spillway discharge and time required to clear the basin of 1,000 yd³ (765 m³) of debris. The 24,300, 28,200, and 30,200 ft³/s (688, 799, and 855 m³/s) spills cleaned the basin in 6, 3, and 2-1/2 hours, respectively. These tests verified the fact that the larger releases cleared the basin of debris with less time and total water than the small releases. The curves in figure 5 illustrate the relationship of the variables, spillway discharge (Q), time of spill (T), and volume of deposited material to be removed from the stilling basin (V_m).

The project office used the results of these tests to formulate the operating procedure to remove approximately 900 yd³ (688 m³) of debris from the Canyon Ferry Dam spillway stilling basin. On May 8, 1974, the Canyon Ferry Dam spillway gates were opened in increments of 5,000 ft³/s (142 m³/s) over a 10-minute period and held at each increment for 20 minutes until a total spillway flow of 28,200 ft³/s (799 m³/s) was released. The spillway discharged for 3 hours at 28,200 ft³/s (799 m³/s) and then the gates were closed over an 18-minute interval. During the spill the powerplant releases continued unchanged discharging an additional 5,900 ft³/s (167 m³/s) into the downstream channel. The Helena Valley Pumping Plant turbine also operated to prevent riverbed material from being washed into the turbine draft tubes. The peak discharge into the river was

approximately 34,500 ft³/s (977 m³/s) for 3 hours. Soundings taken on May 9 and 10 indicated that the spill was successful in flushing the 900 yd³ (688 m³) of material from the basin. It was later verified by divers that the spill had swept the basin clean of all material except that material lodged under the exposed reinforcing bars.

Summary of Results

1. When the river outlet releases exceed 3,000 ft³/s (85 m³/s), riverbed material move into the spillway stilling basin.
2. Releases from the spillway, powerplant, or the Helena Valley Pumping Plant do not carry riverbed material into the stilling basin.
3. Model tests indicated that the deposited riverbed material could be cleared from the stilling basin with adequate spillway releases. The time and amount of spill required to clear the material from the stilling basin can be determined from figure 5.
4. The spilling technique developed in the model successfully cleared approximately 900 yd³ (688 m³) of riverbed material from the Canyon Ferry Dam spillway stilling basin.
5. Clearing the river bottom of loose riverbed material down to bedrock for a distance of 100 feet (30 meters) downstream from the end sill will prevent the movement of riverbed material into the basin for river outlet releases up to the design discharge of 9,500 ft³/s (269 m³/s).
6. Uniform operation of all four river outlets or all four spillway gates gave best results.
7. The simultaneous operation of the two center spillway gates (Nos. 2 and 3) with the two outside river outlets (Nos. 1 and 4) may minimize the dissolved gas uptake. This simultaneous operation will also prevent the movement of riverbed material into the stilling basin. To prevent riverbed erosion downstream from the stilling basin this simultaneous operation should be limited to a total release not to exceed 10,000 ft³/s (283 m³/s) over the spillway and 4,750 ft³/s (135 m³/s) through the river outlets.

Table 1
Log of Model Tests

Test No.	Time in hours		Comments	Operations					
	Model	Prototype		Spillway		River outlets		Powerplant	Pumping plant
				Gates	Discharge ft ³ /s	Gates	Discharge ft ³ /s	Discharge ft ³ /s	Discharge ft ³ /s
1		6.25 43		—	—	1, 2, 3, 4	9,500	—	—
2		6.25 43		1, 2, 3, 4	9,400	—	—	—	—
3		6.00 41		1, 2, 3, 4	24,000	—	—	—	—
4	A	3.00 21		—	—	1, 2, 3, 4	8,000	6,000	—
	B	3.50 24		1, 2, 3, 4	7,400	—	—	6,000	—
5	A	8.00 55	Verification test	—	—	1, 2, 3, 4	7,300	6,000	—
	B	4.00 28		1, 2, 3, 4	8,000	—	—	6,000	—
	C	4.00 28	Gates 1, 2, 3, 4. 6,000 ft ³ /s decreasing to 2,000 ft ³ /s over 4 hours	—	—	*See Comments		6,000	—
6	A	4.50 31 ¹		—	—	—	—	6,000	—
	B	1.50 10		—	—	—	—	6,000	463
7		6.25 43		—	—	1, 2, 3, 4	4,000	6,000	—
8		6.25 43		—	—	1, 2, 3, 4	5,000	6,000	—
9		6.25 43		—	—	1, 2, 3, 4	6,000	6,000	—
10		2.75 19	Initial condition, 400 yd ³ debris in basin	1, 2, 3, 4	11,000	—	—	6,000	—
11		2.28 16	Initial condition, 400 yd ³ debris in basin	1, 2, 3, 4	20,800	—	—	—	—
12		1.37 9.5	Initial condition, 400 yd ³ debris in basin	1, 2, 3, 4	24,400	—	—	—	—
13		6.58 46		—	—	3, 4	4,750	6,000	—
14		6.25 43		—	—	1, 2, 3, 4	3,000	6,000	350
15		5.75 40		—	—	1, 4	4,750	6,000	350
16	A	4.00 28		—	—	1, 2, 3, 4	7,900	6,000	350
	B	1.80 12		1, 2, 3, 4	15,000	—	—	—	—
	C	6.17 43		—	—	1, 2, 3, 4	3,000	—	—
17		1.20 8.3	Initial condition, 1,000 yd ³ debris in basin	1, 2, 3, 4	24,300	—	—	—	—
18		1.20 8.3	Initial condition, 1,000 yd ³ debris in basin	1, 2, 3, 4	28,200	—	—	—	350
19		1.20 8.3	Initial condition, 1,000 yd ³ debris in basin	1, 2, 3, 4	30,200	—	—	—	350
20		6.25 43	Velocity measurements (bedrock installed)	—	—	1, 2, 3, 4	9,500	6,000	350
21		6.00 42		—	—	1, 2, 3, 4	9,500	6,000	—
22		6.00 42		—	—	1, 2, 3, 4	9,500	6,000	—
23			Velocity measurements	—	—	1, 2, 3, 4	3,000	6,000	350
24		6.25 43		1, 2, 3, 4	4,100	—	—	6,000	350
25		6.00 42		1, 2, 3, 4	6,600	—	—	6,000	350
26		3.00 21		2, 3, 4	14,500	—	—	6,000	—
27		3.00 21		2, 3, 4	8,300	—	—	6,000	—
28		3.00 21		2, 3, 4	3,900	—	—	6,000	—
29		6.25 43	Riverbed cleared to bedrock 50 feet downstream from basin	—	—	1, 2, 3, 4	9,500	6,000	—
30		6.25 43	Riverbed cleared to bedrock 75 feet downstream from basin	—	—	1, 2, 3, 4	9,500	6,000	—
31		6.25 43	Riverbed cleared to bedrock 100 feet downstream from basin	—	—	1, 2, 3, 4	9,500	6,000	—
32		6.25 43	Two 5-foot-high, 20-foot-long walls on top of end sill	—	—	1, 2, 3, 4	9,500	6,000	—
33		6.25 43	8-foot-high wall on top of end sill, extended full width of basin	—	—	1, 2, 3, 4	9,500	6,000	—
34		6.25 43	Simultaneous operation	2, 3	4,750	1, 4	4,750	6,000	—
35		2.17 15	Initial condition, 1,900 yd ³ debris in basin	1, 2, 3, 4	36,300	—	—	—	—
36		6.25 43	12-foot-high wall on top of end sill, extended full width of basin	—	—	1, 2, 3, 4	9,500	6,000	—
37		5.67 39	12-foot-high wall on top of end sill, extended full width of basin	1, 2, 3, 4	40,000	—	—	6,000	—
38		6.25 43	Simultaneous operation	2, 3	10,000	1, 4	4,750	6,000	—

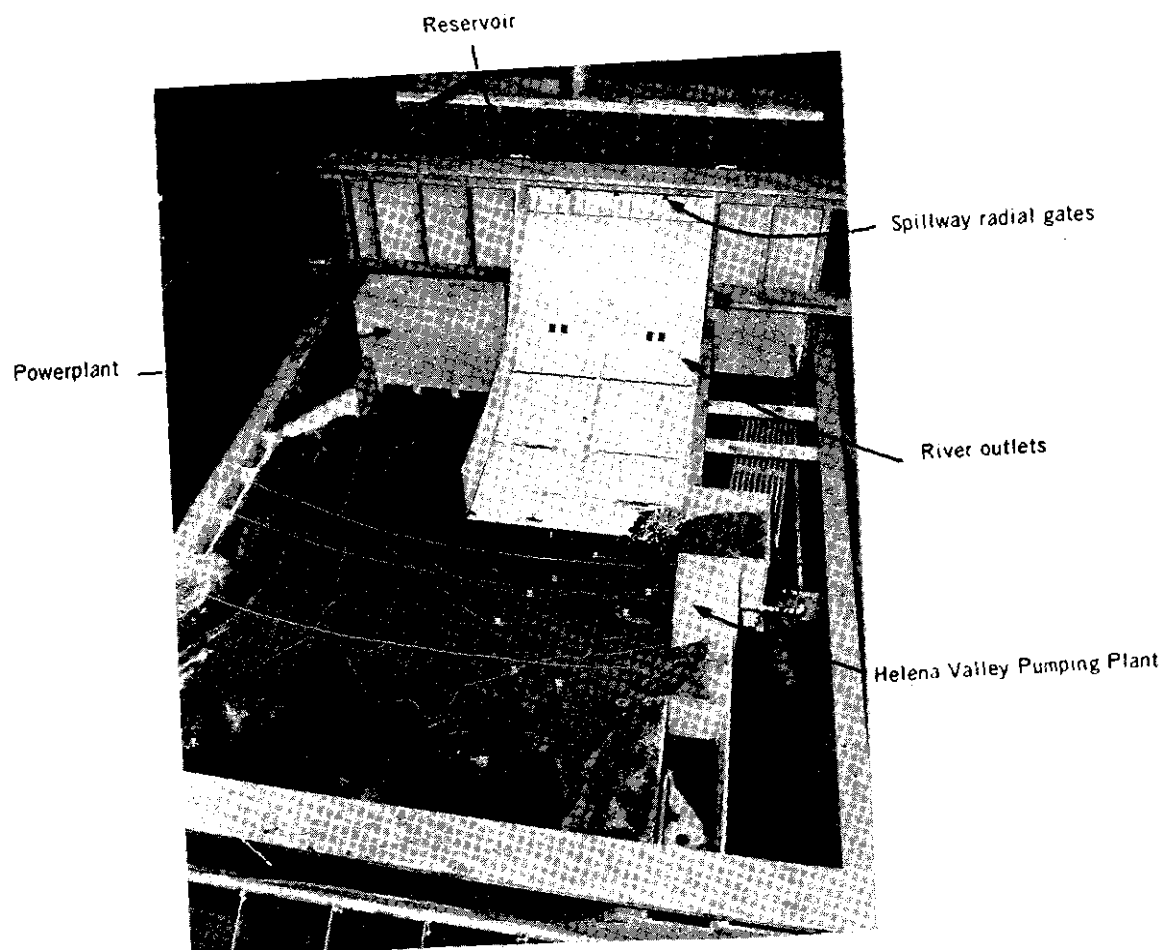


Figure 1. 1:48 Scale model layout of Canyon Ferry Dam. Photo P296-D-75729

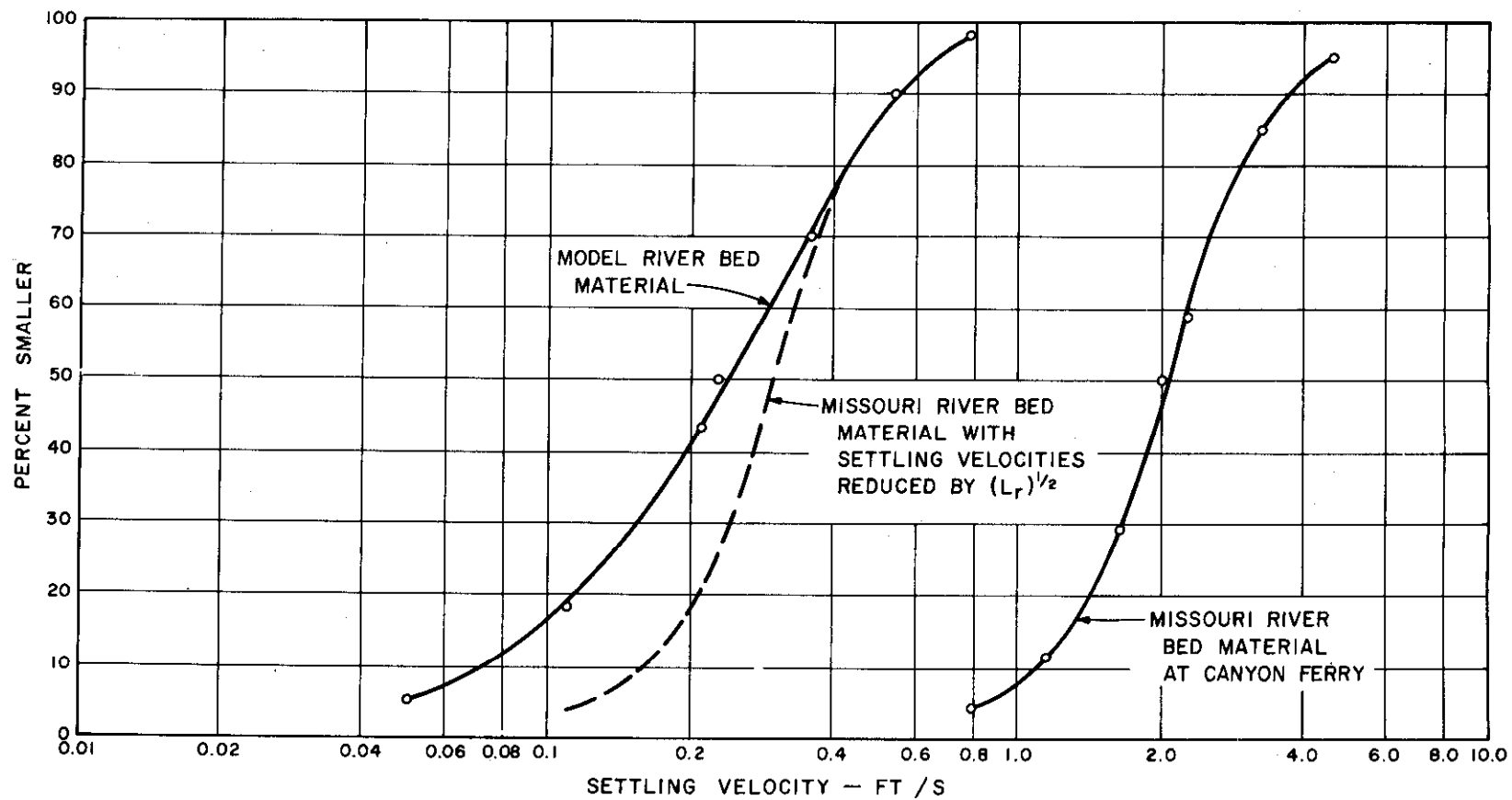
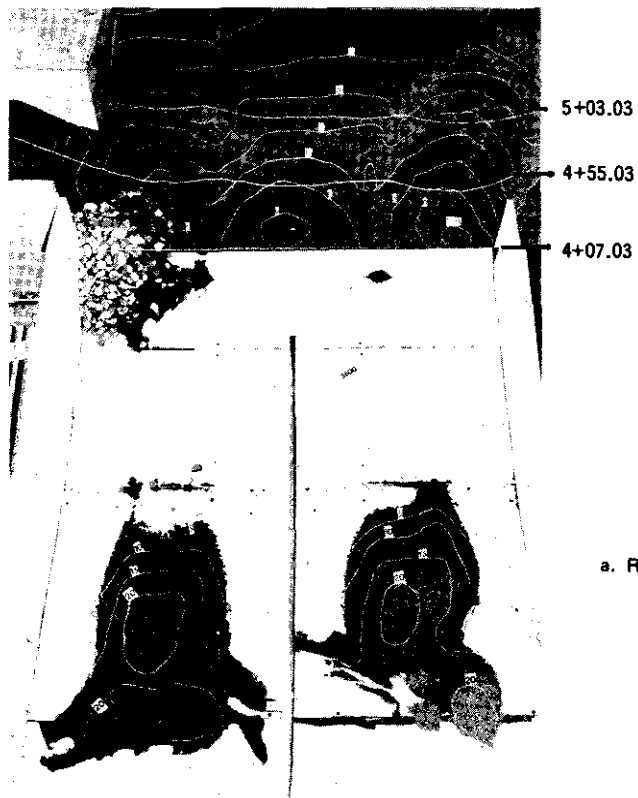
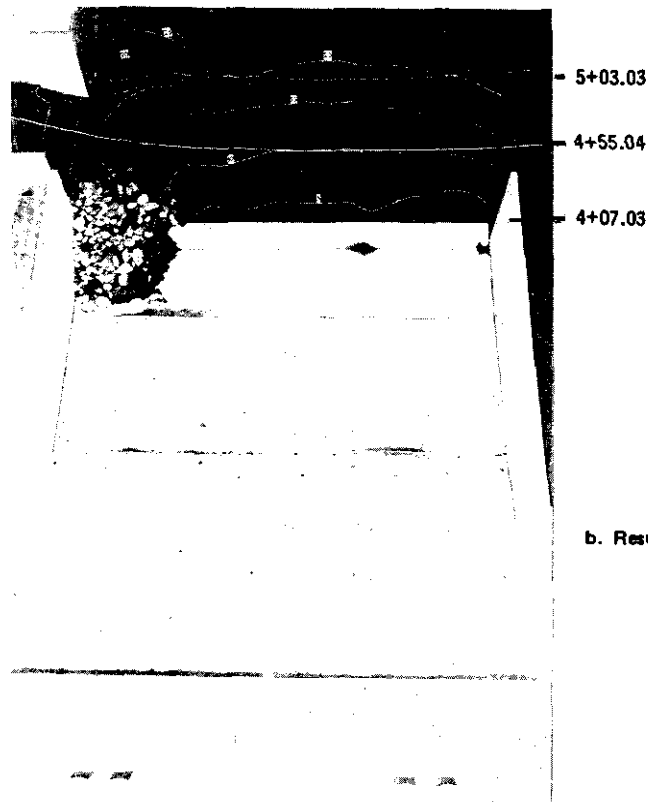


Figure 2. Comparison of settling velocities, Canyon Ferry model study.



a. Results of test 1, $Q = 9,500 \text{ ft}^3/\text{s}$.



b. Results of test 14, $Q = 3,000 \text{ ft}^3/\text{s}$.

Figure 4 Results of river outlet releases

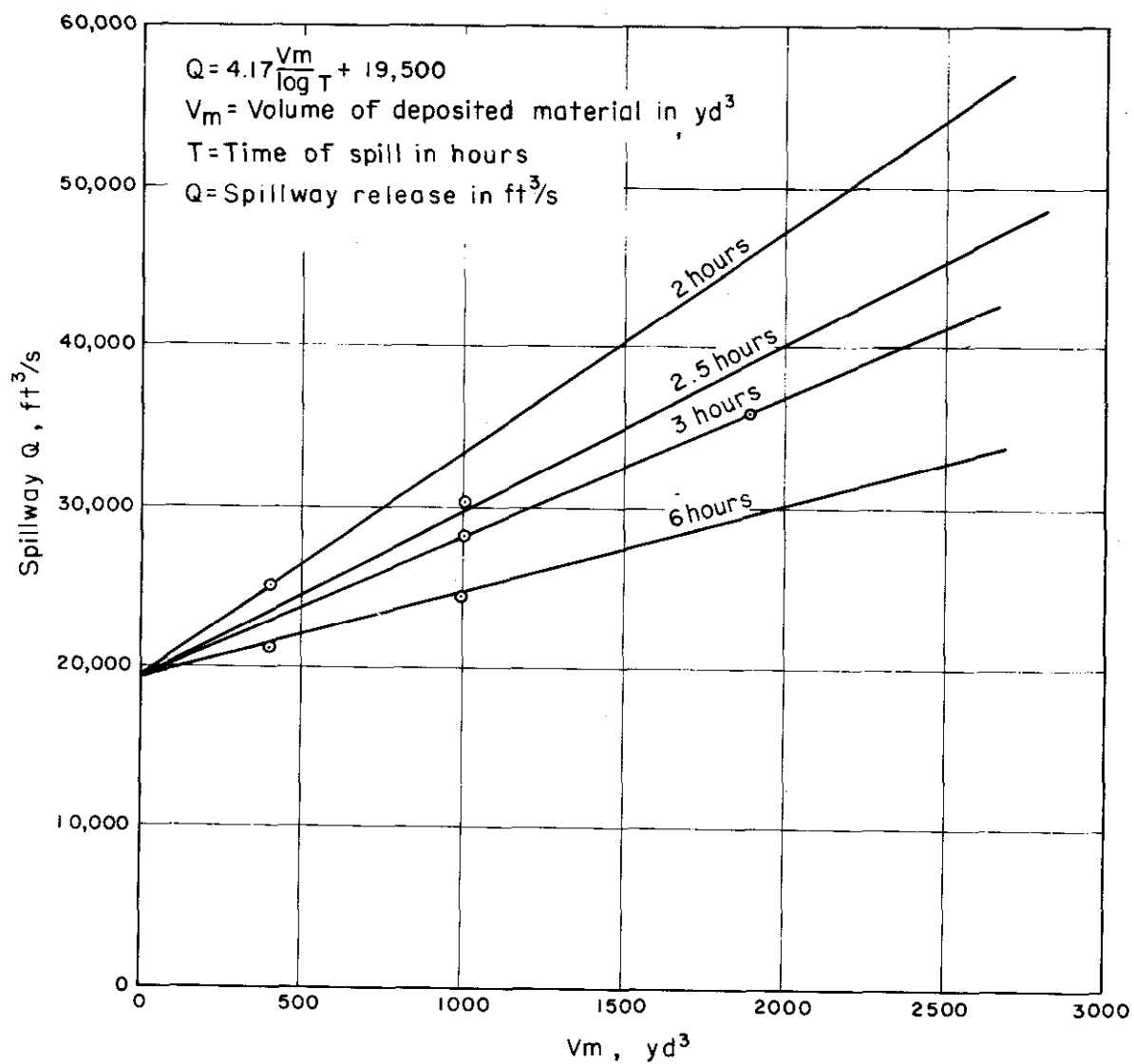


Figure 5 . Removal of riverbed material from stilling basin.

SEDIMENTATION IN COON CREEK VALLEY, WISCONSIN

by Stanley W. Trimble

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University of California, Los Angeles

Abstract

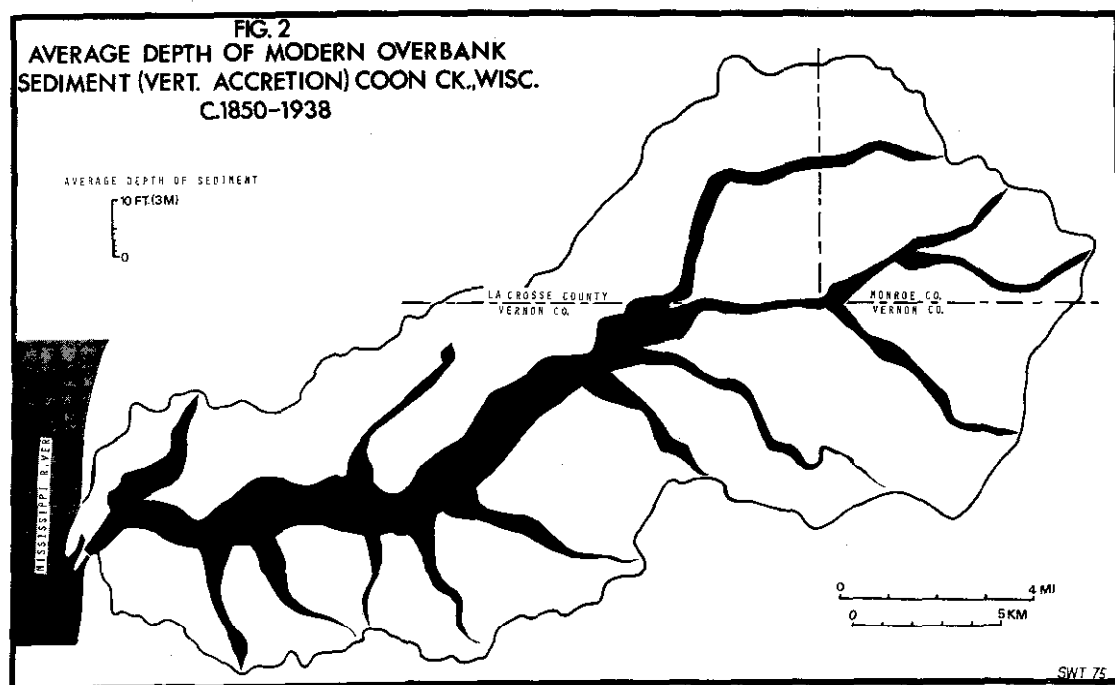
Coon Creek Valley is a severely eroded watershed in the Driftless Area of Wisconsin. Between c. 1850 and 1938, a net total of about $19,500 \text{ dkm}^3$ ($16,000 \text{ a.f.}$) of sediment was deposited within the system, a rate of about $56 \text{ dkm}^3/\text{km}^2$ (116 a.f./mi^2), or $0.7 \text{ dkm}^3/\text{km}^2/\text{yr}$ ($1.5 \text{ a.f./mi}^2/\text{yr}$) from the watershed exclusive of the main valley flood plains. These figures do not include colluvium nor material transported from the system by streamflow. Between 1938 and 1974, sedimentation continued, but the total represents only about half the previous rate. Within the system, however, there continued a partial transfer of material with net export from tributaries and net deposition in the lower-gradient main valley. Within the past few years, there has been little deposition and the present rate may be less than 10 percent of the high rates of the 1920's and 30's. This decrease is primarily attributed to changes of land use and land treatment.

The Coon Creek watershed (Fig. 1) is located in the Driftless Area of the upper Midwest, a sub-maturely dissected plateau with local relief of about 135 m (400 ft), steep valley sides, and narrow valleys. The main valley of Coon Creek generally does not exceed 1 km (0.6 mile) in width and is partially occupied by high terraces. The rolling uplands and valleys have been cultivated, but the steepest slopes have generally remained in woodland, although sometimes excessively grazed. Like most of the Driftless Area, Coon Creek valley has undergone severe man-induced soil erosion with consequent massive sedimentation of stream and valleys. The watershed was the first (1934) Soil Erosion Control Demonstration Area of the U.S. Department of Agriculture.

A comprehensive long-range study of the sedimentation phenomena was designed by S.C. Happ for the Soil Conservation Service in 1938. V.E. McKelvey made auger borings on more than a hundred valley cross sections, or ranges, mapped the sediment on air photos and analyzed the results (McKelvey, 1939). Supplemental boring and mapping was done by Happ and others, and W.F. Witzgall surveyed ground line profiles on the ranges. Although never completed as

planned, that study showed the general magnitude and distribution of modern sediment, and its relation to stream channel erosion.

Almost all of the dark gray to black natural flood plain soils, and some low alluvial terraces, had been covered by brown stratified sediment, mostly sandy silt. Artifacts in the sediment, and testimony of old residents, confirmed that the aggradation had occurred since European settlement, a period of about 90 years. Those modern vertical accretion deposits had an average thickness of about 2 m (6.5 ft) in the main valley and about 0.6 m (2 ft) in principal tributaries (Fig. 2). The main stream channel had also been aggraded but appearances suggested less than the overbank aggradation. It was uncertain whether tributary channels had been aggraded or degraded, on average, although degradation was obvious near the heads and in some reaches of streams crossing alluvial terraces.



The covered flood plains extend up tributary valleys to gorges, or trenched channels on steep slopes, below and across the outcrop zone of the cliff-forming Oneota dolomite formation. Remnants of covered plain were common in the larger gorges, but were rare along

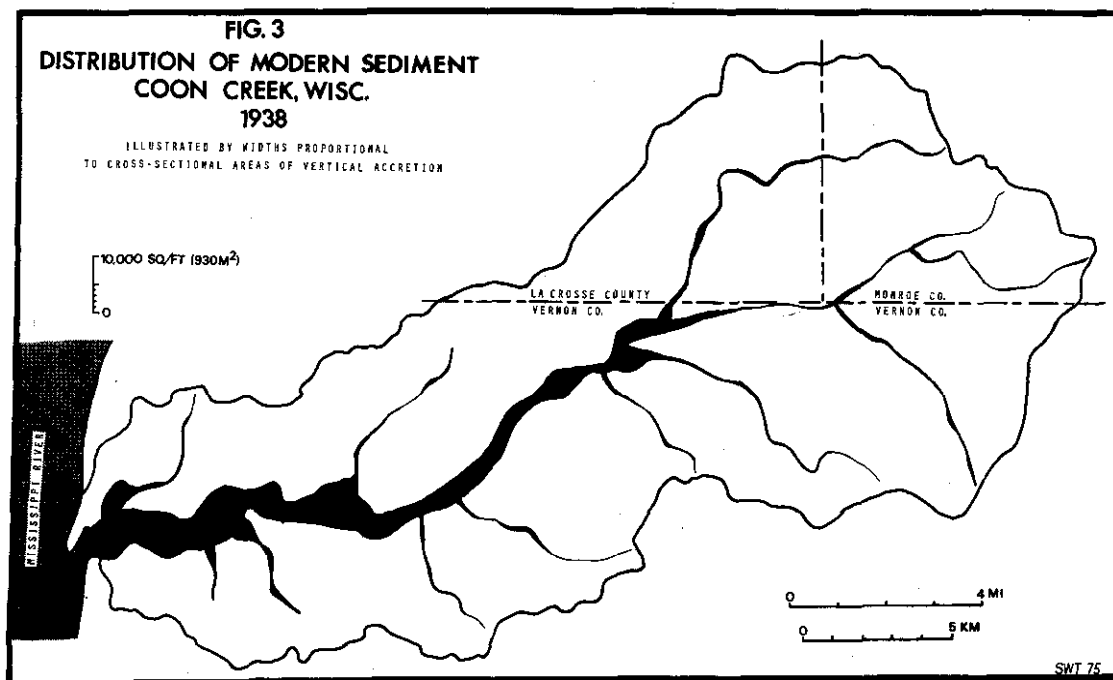
smaller trenched channels where they may never have been continuous. Vertical accretion deposits, commonly 0.3-0.6 m (1-2 ft) thick, had also covered the dark natural soils in many shallow upland valley heads, on the ridges above the gorges and trenched channels.

The tributary streams below the gorges had generally widened by bank erosion, forming a new "meander plain" on gravelly and bouldery lateral accretion deposits at lower level than the covered plain. The meander plains were 30-60 m (100-200 ft) wide in many places, and about one-third of the covered plains in tributaries had been destroyed. Channels had been enlarged so much that overbank flooding had become rare in the tributaries. Remaining modern vertical accretion deposits exceeded the volume removed by bank erosion, but it was uncertain whether net aggradation was still continuing or might have been reversed and bank erosion have become dominant in the tributaries.

Volumes of modern vertical accretion sediment were recently computed by a mean depth formula, between successive ranges. (Table 1, Fig. 3) Nearly 14,500 dkm³ (12,000 acre feet) were found in the main valley, and about 3,700 dkm³ (3,000 a.f.) in principal tributaries. Extrapolation from small samples indicated nearly 1,200 dkm³ (1,000 a.f.) more in smaller tributaries in the lower half of the drainage basin, and about 2,000 dkm³ (1,600 a.f.) in small upland valley heads on the bordering ridges. There was more modern sediment in small tributaries, in the upper part of the basin, but no measurements had been made in those small tributaries before the study was stopped; it appeared that deposition in such valleys might not exceed the volume of channel enlargement.

Table 1. Summary of modern sediment in Coon Creek and tributary valleys, 1938.

Valley	Vertical Accretion		Lateral Accretion			Channel erosion below projected old soil	Net sediment accumulation
	Area	Volume	Area	Volume			
				Total	Above projected old soil		
	ha (acres)	dkm ³ (ac. ft.)	ha (acres)	dkm ³ (ac. ft.)	dkm ³ (ac. ft.)	dkm ³ (ac. ft.)	dkm ³ (ac. ft.)
Main stream	728 (1,798)	14,500 (11,750)					14,500 (11,750)
Upper tributaries: (Bohemian, Timber, Bulland, Willsie, Spring)	322 (795)	2,328 (1,887)	172 (426)	1,044 (846)	146 (118)	700 (567)	1,774 (1,438)
Intermediate tributaries: (Bundahl, Dodson, Hasley, Chaseburg, Hohlfeld, Wing, Hunder)	132 (326)	980 (794)	64 (157)	301 (244)	12 (10)	391 (317)	601 (487)
Proksch Coulee & fan	34 (84)	468 (379)	9 (22)	36 (29)	22 (18)	151 (122)	339 (275)
Estimate for small trib- utaries (below Spring Coulee)	221 (546)	1,194 (968)	96 (237)	73 (59)	--	940 (762)	254 (206)
Estimate for upland valley heads	424 (1,047)	1,987 (1,610)	--	--	--	--	1,987 (1,610)
Total	1,861 (4,596)	21,456 (17,388)	341 (842)	1,454 (1,178)	180 (146)	2,182 (1,768)	19,455 (15,766)



Proksch Coulee, a tributary crossing old high terraces into the lower part of the main valley, was studied as a separate and unusual unit. It had been severely trenched, and together with smaller similar tributaries had built an unusually large alluvial fan, comprising nearly 500 dkm³ (400 a.f.) or 2 percent of the total sediment.

There was no way for accurate measurement of lateral accretion deposits in tributary valleys, but nearly 1,500 dkm³ (1,200 a.f.) were estimated from mapping and interpretation of cross sections. The bottom of the modern lateral accretion was taken as elevation of the stream bed, and sides were projected downward at the slope of the higher bank of the covered plain. (Fig. 4, Table 1 & 2) The old soil surface, as found by borings in the covered plain, was projected across the high water channel and meander plain, and lateral accretion deposits above that were considered net modern accumulation; those below the projected soil were considered replacement of older alluvium removed by bank erosion, representing both erosion and deposition volumes, but no net change.

FIG. 4

SCHEMATIC FOR ESTIMATION NET EROSION
AND SEDIMENT ON COON CREEK TRIBUTARY
RANGES

EXAMPLE: SPRING 6

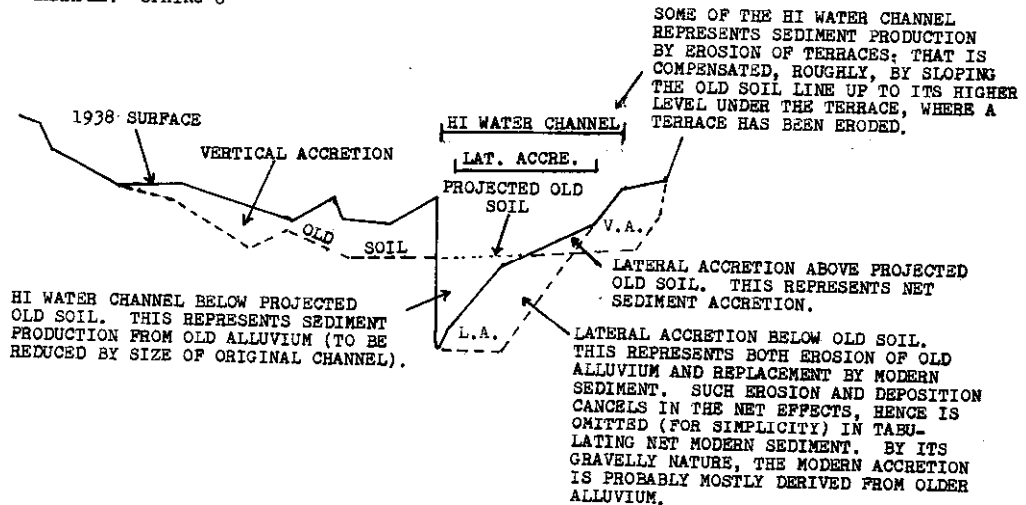


Table 2. Coon Creek Valley and Tributaries
Preliminary summary of 1938 net modern sediment
(negative values represent erosion)

	Vertical Accretion		Channel and lateral accretion		Lateral accretion above projected old soil	Sediment production by channel erosion below projected old soil	Net modern sediment
	Area	Volume	Area				
	ha (acres)	dkm ³ (ao.ft.)	ha (acres)		dkm ³ (ao.ft.)	dkm ³ (ao.ft.)	dkm ³ (ao.ft.)
Main valley	728 (1,798)	14,500 (11,750)					14,500 (11,750)
Upper tributaries (Bohemian, Rulland, Willis, Timber, Spring)	322 (795)	2,329 (1,887)	172 (426)		146 (118)	-700 (-567)	1,774 (1,438)
Intermediate tributaries (Rundahl, Dodson, Hasley, Chaseburg, Hohlfeld, Wing, Hunder)	132 (326)	980 (794)	64 (157)		12 (10)	-391 (-317)	601 (487)
Proksch Coulee and fan	34 (84)	468 (379)	9 (22)		22 (18)	-151 (-122)	339 (275)
Subtotal	1216 (3003)	18,276 (14,810)	245 (605)		180 (146)	-1,241 (-1,006)	17,214 (13,950)
Estimate for minor tributaries to main valley (86 valleys at least .83 km (0.5 mi.) long = 166 km (103 mi.) X 1.5 dkm ³ /km (2.0 a.f./mi.)							254 (206)
Estimate for upland valleys		91 km ² X 22 dkm ³ /km ² (35 sq. mi. X 46 a.f./sq. mi.)					1,987 (1,610)
Total							19,455 (15,766)

The part of the high water section below the projected old soil, and not occupied by lateral accretion, was taken as a measure of net channel erosion, after allowance for the old channel prior to accelerated erosion. Many old residents remembered, or had been told by others, that channels had been much smaller in the early years - typically, "so small that a man could jump across" - with low banks "which a wagon could cross almost anywhere." From such testimony, and observations in a few relatively stable reaches, the average "original" channel sections were estimated to have been 4.6, 1.9 and 0.9 m² (50, 20 and 10 ft²), respectively, in the larger, intermediate, and smaller tributaries. Those adjustments were applied to cumulative data for the tributaries, without attempting apportionment to individual ranges (Table 1).

Sediment in small valleys, 0.8 to about 5 km (0.5-3 mi) long was estimated from measurements in 5 small valleys chosen as representative of various observed conditions. The average from the samples was 2.5 dkm³ (2 a.f.) of net sediment accumulation per mile of valley length. The average was applied to 86 small tributaries entering the main valley, or its branches below the mouth of Spring Coulee. Tributaries less than 0.8 km (0.5 mi) long, and those entering Spring Coulee or farther upstream, were not believed to have appreciable net sediment accumulation after allowance for channel enlargement, except possibly in upland heads which were estimated separately.

Modern sediment in the upland valleys - mostly a few hundred meters to 0.8 km (0.5 mi) long - was measured by borings on 50 cross sections, but data from only 19 have been recovered. Those records, together with mapped areas, indicated about 57 dkm³ (46 a.f.) of sediment per square mile of drainage area. That average was applied to half of the 181 km² (70 mi²) of upland soils recorded by a watershed survey, for a conservative estimate of 1987 dkm³ (1610 a.f.), or about 10 percent of the net sediment found (Table 1).

The total measured and estimated sediment was about 22,830 dkm³ (18,500 a.f.) thought to be within about 80 percent of the total modern valley deposits, exclusive of colluvium on lower hillsides and terraces. About 3,450 dkm³ (2,800 a.f.) of channel erosion in the tributaries reduced net valley deposition to nearly 19,500 dkm³ (16,000 a.f.). That would be equivalent to 55.6 dkm³/km² (116 a.f./mi²) of drainage area outside the main valley floodplain, or 0.7 dkm³/km²/yr (1.5 a.f./mi²/yr) if 80 years be taken as the average period of accelerated erosion. The indicated rates were high, but not extreme for severely eroded watersheds. The current rate in 1938-39 was thought to be higher than the computed average, and perhaps well over twice as high according to some limited data from other parts of the Driftless Area.

THE PRESENT STUDY

The present study of Coon Creek, undertaken in 1974, has as its goals (1) the determination of changes of stream and valley morphology since the 1938 study by resurvey of as many old ranges as can be recovered (2) determination of past and present processes in streams and valleys and the rate of change of these processes, and (3) the causes for these changes as related to changes of watershed land use and land treatment. Thus far (August 1975) goal 1 is more than half completed. Goals 2 and 3 are by necessity long-range and data are being collected.

The major findings thus far are that since 1938 (1) overall sedimentation rates have decreased by almost half (Tables 3 and 4), (2) continued channel enlargement by bank erosion has been dominant in tributaries, with a net loss of material (Tables 3 and 4, Fig. 5), (3) neither meander plains nor overbank flood plains have been aggraded greatly in the bank erosion zones on tributaries, (4) general aggradation of the main valley had continued after 1938, and (5) the stream bed in the main valley has aggraded as much as the overbank flood plain.

Table 3. Comparison of 1938 and 1938-74 net sediment (and erosion) on 55 resurveyed valley ranges and 7 upland boring sections, Coon Creek and tributaries.
(negative values represent erosion)

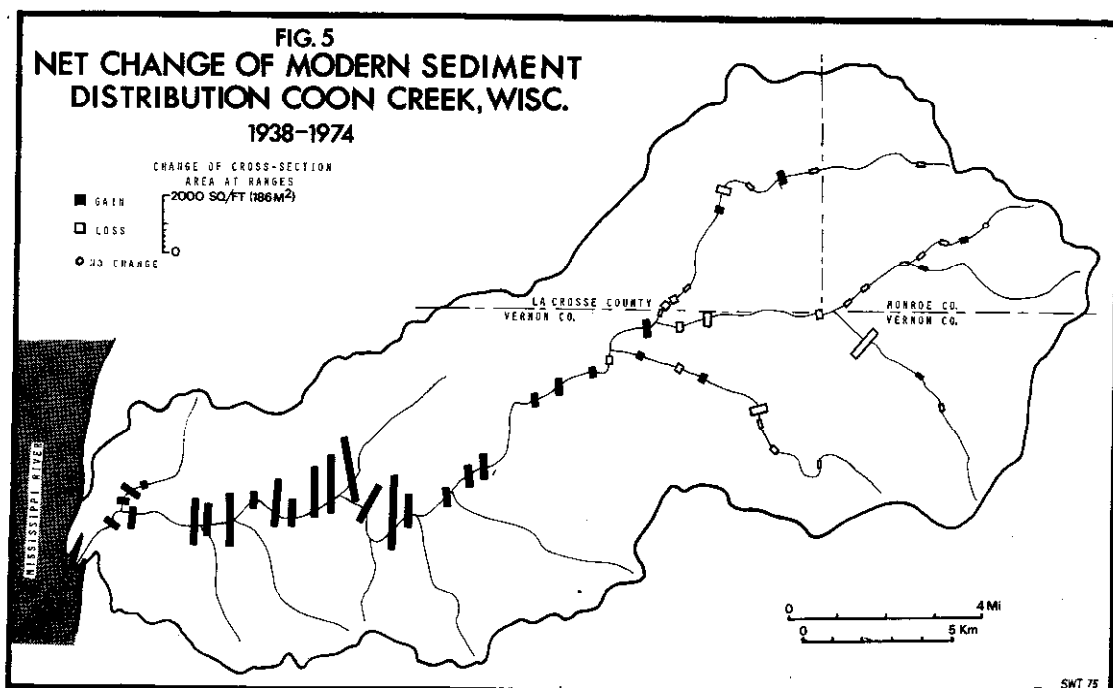
Valley	No. of ranges	1938				1938-1974		
		Vertical accretion	Lateral accretion above projected old soil	Channel erosion below projected old soil	Net sediment	Net Change from 1938		
		m ² (sq. ft.)	m ² (sq. ft.)	m ² (sq. ft.)	m ² (sq. ft.)	Sediment m ² (sq. ft.)	Av. depth m ² (feet)	Cross section %
Coon Creek	21	8,939 (96,125)			8,939 (96,125)	2,199 (23,640)	0.43 (1.4)	+25%
Upper tributaries (Timber, Bohemian, Bulland, Willsie)	24	1,411 (15,175)	54.4 (585)	306 (3,292)	1,159 (12,463)	-259 (-2,785)	-0.06 (-0.02)	-22%
Spring Coulee	7	317 (3,410)	3.3 (35)	96.7 (1,040)	224 (2,405)	-39.1 (-420)	-0.12 (-0.4)	-18%
Proksch Coulee	3	977.4 (10,510)	38.1 (410)		1,015.5 (10,920)	118.6 (1,275)	0.21 (0.7)	+12%
Total	55	11,608.8 (124,830)	95.8 (1,030)	408.4 (4,392)	11,296.2 (121,468)	2,019 (21,710)	0.12 (0.4)	+18%
Upland valleys from borings	7	75.8 (815)			75.8 (815)	37.2 (400)	0.24 (0.8)	+49%

Table 4. Comparison of average annual net deposition rates on 55 resurveyed valley ranges and 7 upland boring sections, Coon Creek and tributary valleys.

	1858-1938 ¹ (80 yrs.)	1938-1974 (36 yrs.)	Change in average rate
	m ² /yr. (sq. ft./yr.)	m ² /yr. (sq. ft./yr.)	(%)
Coon Creek	111.8 (1,202)	61.1 (657)	-45%
Upper tributaries (Bohemian, Timber, Rulland, Willsie)	14.5 (156)	-7.2 ² (-77)	-149%
Spring Coulee	2.7 (29)	-1.1 ² (-12)	-141%
Proksch Coulee & fan	12.7 (137)	3.3 (35)	-74%
Total	141.7 (1,524)	56.1 (603)	-60%
Upland valleys	0.9 (10)	1.0 (11)	+10%

¹Assumed average period of accelerated deposition

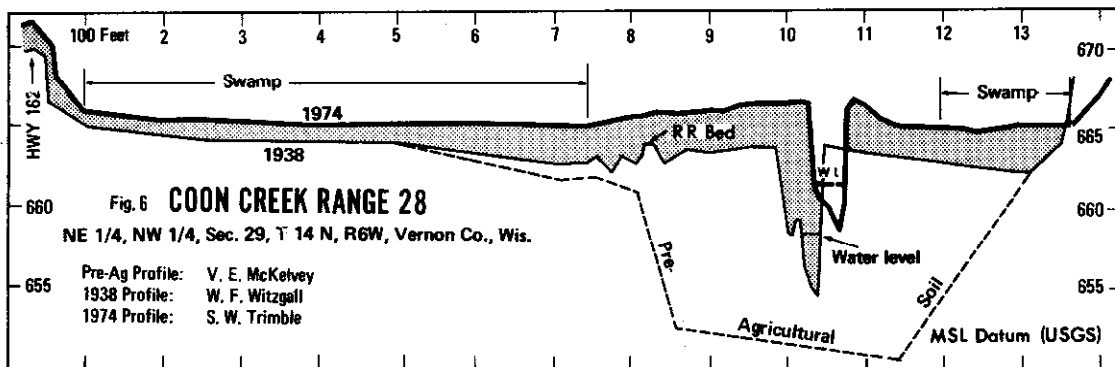
²Net erosion, by channel erosion exceeding overbank deposition



Main Valley

Flood plain aggradation has continued throughout the main valley below the Bohemian Coulee-Timber Coulee confluence (Fig. 5). Resurveys of 21 ranges indicate average aggradation of 0.4 m (1.4 ft) since 1938 (Tables 3 and 4), or only about 22 percent of the average thickness of pre-1938 modern sediment. The indicated 1938-74 rate of sedimentation has been only 55 percent of the average rate for the prior period of accelerated soil erosion (approximately 80 years), and only about one quarter of the rate which may have been prevailing in 1938 (twice the prior average). There appears to be little evidence of overbank deposition within the past few years. In view of the high rate believed to have prevailed in 1938, the present rate may be less than 10 percent of the 1938 rate.

Distribution of the 1938-74 sediment is similar to that found by 1938 borings, with the thickest deposits (over 0.6 m or 2 ft) near Chaseburg in the middle part of the valley (Figs. 5 and 6). Resurveys show that the main valley stream channel has generally widened slightly, but several channel cross-sections indicate less bankful discharge capacity than in 1939. At most ranges there has been apparent lateral channel shifting, up to about 9 m (30 ft), but bank erosion does not appear abnormally active. The channel bed has been generally aggraded as much as the immediately adjacent flood plain.



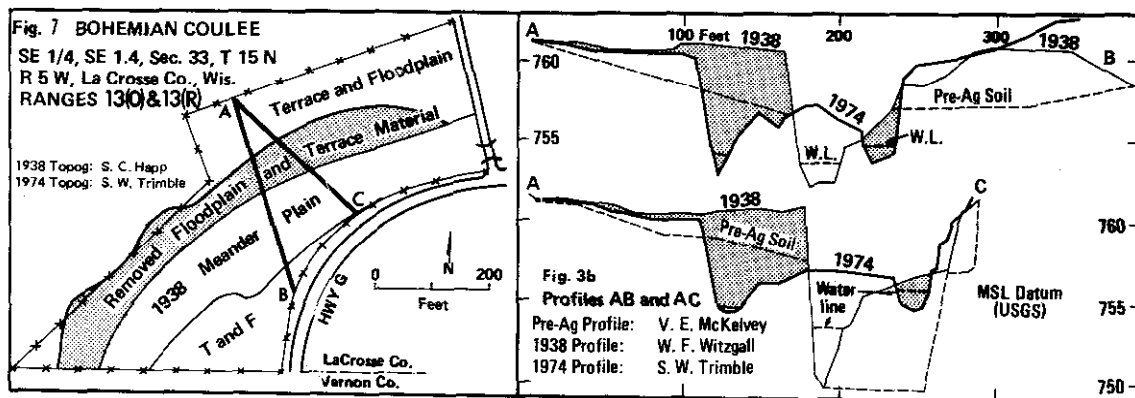
Bank Erosion Zone

Stream bank erosion has been the dominant process since 1938 throughout most of the principal tributary valleys below the "gorge" sections so that there has been a net loss of material (Tables 3 and 4, Figs. 4, 5, and 7). The meander plains, lying at a lower level than the old aggraded flood plains, appear to actually be newly-formed flood plains with some vertical accretion, leaving the old flood plains as terraces. Stream discharge measurements since 1974 suggest that storm flow has significantly decreased. If so, the lower level of these new flood plains may reflect the lower frequency to high-discharge events. The meander plains, raw, irregular, and barren in the 1930's, have grown in size and are now generally smoother, more level, well vegetated and most appear to be good grass pastures.

The low-water channels have shifted about considerably within the meander plains since 1938, but have begun to stabilize and erosion of the high-water banks has apparently decline greatly. High raw banks were prominent features in 1938, but are now sporadic and limited in extent. The higher rate evidently continued for some time after 1938, however, for the 1938-74 record shows considerable channel widening and cross section enlargement (Fig. 7). This decline of bank cutting may also be related to the decline of stream storm flow. Preliminary averages of the resurveys indicate about 35 percent increase in channel width with concomitant channel enlargement so that bankful discharge capacity (to the level of the covered flood plain) has been greatly increased.

Although there has been general channel enlargement, there has been an average rise in talweg elevations of about 0.2 ft at the ranges. However, there is considerable variation and the significance of the average is uncertain.

Local residents credit a dozen SCS detention dams, built 1960-62 in Bohemian, Rulland, Willsie and Timber Coulee watersheds for almost complete elimination of floods in those valleys and upper part of the main valley. This study has not yet identified any greater changes in the controlled valleys than in Spring Coulee where there are no retention dams; but data are not yet complete. Indications are that stream response to rain storm events has been everywhere reduced as a result of changes in land use and treatment, and the dams merely assist in reducing the already mitigated discharge peaks. Changes in stream regimen since the 1930's are an important objective of the present study and stream gages have been installed at Coon Valley and in a small tributary of Proksch Coulee (Fig. 1).



Trenched Valleys

Below the "upland" sections are relatively short, steep, narrow "gorge" sections where the tributary valleys descend over the Oneota dolomite. Channel trenching (or gullying) was the dominant process in 1938, although bordering remnants of flood plain showed there had been prior aggradation. Resurveys of two ranges show that the trenched channels have become slightly deeper and larger in cross sections since 1938. At one range there appears to have been some shallow overbank scouring, and at the other a small amount of colluvial accumulation.

Upland Valleys

Practically all of the tributary valleys have relatively short "upland" sections at their heads, above the Oneota dolomite formation. These sections are mostly less than 1.6 km (1 mi) long, with small, discontinuous channels. Accumulation of one to a few feet of sediment had been found by 1939-40 borings on many lines referenced to fence corners, etc. No monumented ranges have been resurveyed in the upland sections, but new borings have been made on seven such lines, and that small sample shows no reduction in the rate of sediment accumulation. This is difficult to explain in light of data which suggest that upland soil erosion has decreased by a magnitude since the early 1930's as a result of soil conservation measures (Lund, in preparation). A recent sediment survey of eight flood-retention reservoirs in the study area showed present average sediment yields of about $0.05 \text{ dkm}^3/\text{km}^2/\text{yr}$ ($0.1 \text{ a.f./mi}^2/\text{yr}$) from watersheds ranging in size from 86.6 to 614 ha (214 to 1517 acres) (S.W. Lund, personal communication).

Proksch Coulee Fan

Deposition has continued on the fan at mouth of Proksch Coulee, but at a slower rate. Resurveys of two ranges show 0.2 m (0.7 ft) of aggradation, only 12 percent of the prior thickness measured by borings in 1938. The small ditched channel across the fan has deepened about 0.5 m (1.5 ft), as has the larger channel at a range about a quarter mile above the fan and a similar distance below the SCS "soil saving dam," an early grade stabilization structure to retard trenching of the sandy Pleistocene terrace which occupies much of Proksch Coulee.

CONCLUSIONS

This study indicates the extraordinarily high rates of erosion and sedimentation which can exist under human disturbance, and conversely, it shows that these high rates can be greatly mitigated. The study also reinforces the idea that much, if not most eroded material may be "sunk" into the system as deposition and therefore not measurable at some downstream monitoring station (Happ, 1940, Trimble, 1975). Conversely, the continued erosion of tributary channels since 1938 indicates that stream sediment loads may be partially attributable to channel rather than upland erosion. Thus, stream sediment loads may be dubious indicators of upland processes even when sampled over more than a century.

ACKNOWLEDGEMENTS

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CHANNEL IMPROVEMENTS OF THE MISSOURI RIVER

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ABSTRACT

The lower Missouri River channelization is a classic example of engineers working with nature to change a liability to an asset. The Missouri was changed from an uncontrolled, sediment laden, unsightly, flooding and eroding river to an orderly well-behaved, clear and scenic river. The river was changed from an economic drain to an asset.

The Missouri River is about 2500 miles long. At Sioux City, Iowa it has an average annual flow of 31,700 cfs (73 years of record). Prior to river development the average annual sediment discharge was 164 million tons at Omaha, Nebraska (13 years of record). After development, average annual discharge decreased to 28 million tons (19 years of record).

INTRODUCTION

The Missouri River is one of the major rivers of the United States. It is about 2500 miles long and its drainage area is approximately 529,000 square miles. This makes it the third largest river in area on the North American continent. Only the MacKenzie and Mississippi rivers are larger. The drainage area encompasses all or parts of 10 states and contains about one-sixth of the total area of the contiguous states. The sediment discharge of the Missouri River ranks it as tenth of all the major rivers of the world (Todd, 1970). The estimated annual yield is 218 million metric tons. Only two streams of these ten largest in sediment discharge have lower annual average flows, the Ching and Yellow rivers in China.

The Missouri River can be divided into five sections. These are the uppermost or mountain streams which are from the headwaters to Great Falls, Montana; the mountain plains from Great Falls to the confluence of the Yellowstone; the high plains from the confluence with the Yellowstone to Sioux City, Iowa and the lower Missouri from Sioux City to the confluence with the Mississippi River just upstream of St. Louis, Missouri. From the confluence with the Yellowstone River to the mouth, the river was a wild, sediment-laden stream. Upstream of the confluence with the Yellowstone River the Missouri has less sediment load than the Yellowstone. But below, the Missouri River lives up to its name as "Big Muddy"- "Too thick to drink and too thin to plow." Although in truth, the saying is more apt for its high plains tributaries such as the Powder River in Wyoming or the White River in South Dakota where concentration larger than 30 percent by weight has been measured.

In spite of its wild nature, banks with caving, shifting channels, floating logs, "sawers" and general unsuitability for navigation, the Missouri River was navigated by canoe, keelboat, and steamboat. Lewis and Clark in their epic journey used a 55 ft keelboat and two large pirogues (Cours, 1893). Navigation of the Missouri was difficult and

more than 440 steamboats were sunk during the period prior to the railroads. The first steamboat to reach the mouth of the Platte was the Western Engineer in 1819. In 1832 the steamer Yellowstone reached the mouth of the Yellowstone River. In 1858, the banner year of early steamboats on the Missouri, there were "somewhat less than 100 boats in service" (Vestal, 1964). Vestal states that "in 1862 four boats reached Fort Benton, in '66 thirty-one, in '67 thirty-nine. But in 1874, two years after the railway reached Bismarck, North Dakota, only six boats reached the mountains." With the advent of the railroads, commercial traffic all but ceased on the Missouri River until the late 1930's when channelization work started. During World War II, work on the channel stopped. There was some traffic during the war, but only reached significant volume on the reaches below Kansas City after the war. In 1956 traffic resumed to Sioux City, Iowa, although at that time, reconstruction to repair and realign sections destroyed during the war was being carried out. Major alignment work was completed in the 1960s and traffic increased to its present volume of 6 million tons per year (all traffic).

The variable nature of the flow in the Missouri River, the periodic floods and droughts, the muddy silt-laden flow, and the shifting erosion of the valley led to channel improvements in the thirties and to the Flood Control Act of 1944 for multi-purpose development of the basin. The act was the result of combining two separate plans, one by the U.S. Bureau of Reclamation and the other by the U.S. Corps of Engineers into a coordinated plan for flood control, irrigation, hydroelectric power and navigation called the "Pick-Sloan Plan" named after W.G. Sloan and Gen. Lewis A. Pick. Other benefits were soil erosion and water quality control, conservation of fish and wildlife, improved municipal and industrial water supply, and outdoor recreation. For navigation and flood control the act provided for six main stem dams and a navigation channel from Sioux City, Iowa, to the Mississippi River. The navigation channel was to fix the river in space and prevent the wild shifting of the channel that occurred in the past. The channel of the Lower Missouri River was a braided channel that shifted from one part of the floodplain to the other. This shifting of the river channel often changed state boundaries in addition to destroying productive agricultural land.

This paper will describe the lower Missouri River channelization project from River mile 0 at the confluence of the Mississippi River to Sioux City, Iowa, mile 732. The flow, sediment discharge, channel design, methods of construction, and environmental changes will be described.

WATER AND SEDIMENT DISCHARGE

The average annual discharge at Hermann, Missouri (River mile 98, drainage area about 528,200 sq mi) for the period 1897 to September 1970 is 78,480 cfs or 56,860,000 acre-ft per year. Maximum discharge of record at the Hermann gage was 676,000 cfs on June 6, 1903. The April 1952 flood had a peak discharge of 368,000 cfs. The average annual discharge at Sioux City, Iowa (River mile 732, drainage area about 314,600 sq mi) for the period 1897 to 1970 is 31,700 cfs or 22,970 acre-ft per year. Maximum discharge of record was 441,000 cfs April 14, 1952.

The discharge of water and sediment in the lower reach has been altered considerably by the construction of the five main stem dams upstream of Sioux City, Iowa and river channelization. The dam's capacity, river mile location, date of completion, and elevation are given on Fig. 1.

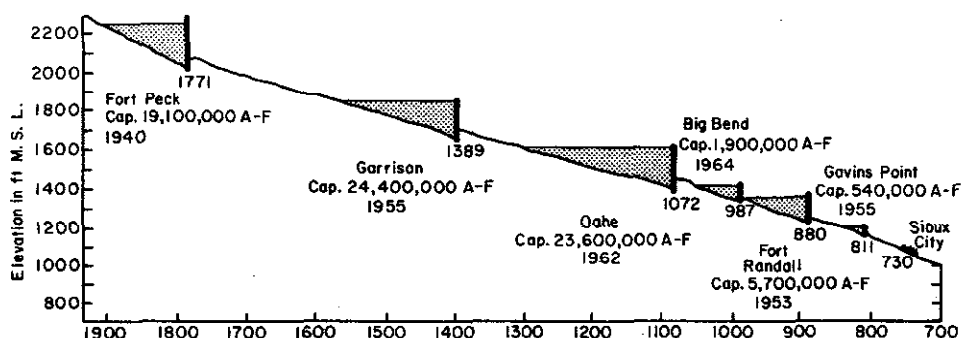


Fig. 1. Capacity, completion dates and distance upstream from mouth (1960 mileage) of main stem dams on the Missouri River.

Storage capacity of the five reservoirs total 75,240,000 acre-ft. This is 1.3 times the average annual flow of the Missouri River at Hermann, but is 3.3 times the average annual flow at Sioux City. These ratios illustrate how the relative influence of the storage decreases with distance down the river. The main stem reservoir can control all major floods arising upstream of Sioux City. But tributary inflow below the dams can cause floods downstream of Sioux City.

The construction of the dams and the use of the water for navigation, flood control, irrigation, and power production has changed the time history of the flows. To maintain adequate depth in the navigation channel during the summer months a minimum flow of 30,000 cfs at Sioux City is maintained during the eight month navigation season (April 1 to December 1). Flows during the navigation season may be larger depending on available water in storage, needs for power production, and evacuation of water from the exclusive flood zone of the reservoirs. For navigation and flood control flows will be lower than historic in the spring and larger than historic in late summer, fall, and winter. Winter flows are maintained larger than historic to increase the amount of water available for wastewater dilution. Ice problems, such as ice jams and damage to docks, intakes, outlets, banks, etc., restrict the magnitude of winter flows. Changes in historic discharge patterns are illustrated by the flow duration curves for flows at Sioux City, Iowa (Fig. 2). The flow duration curve indicates that whereas two percent of the time historic flows exceeded 100,000 cfs they now only exceed 40,000 cfs. Flows now exceed 25,000 cfs 60 percent of the time whereas historically they exceeded 20,000 cfs. Thus, peak flows have been decreased and medium flows are sustained for longer periods of time. The changes in suspended sediment load is given in Table 1.

The median particle size of the bed material has also changed at Sioux City (getting coarser) but doesn't appear to have changed at Omaha,

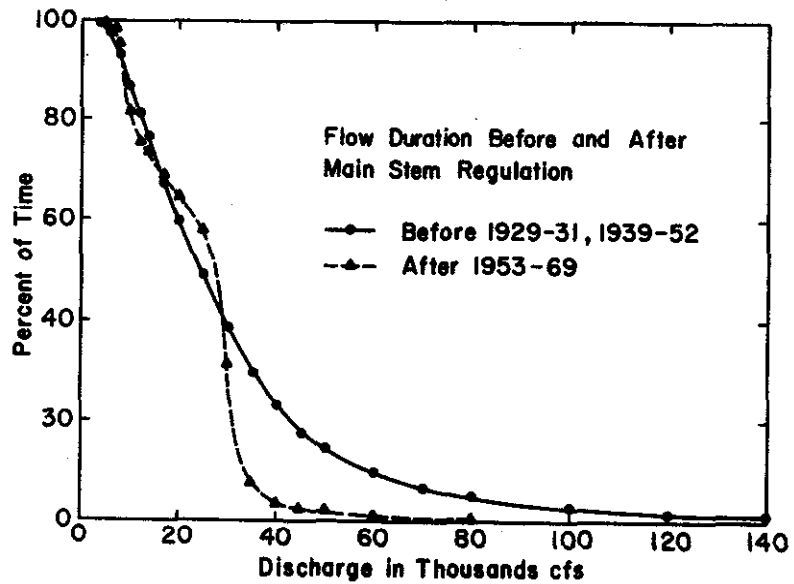


Fig. 2. Flow duration curves for Missouri River at Sioux, City, Iowa.

Table 1. Yearly suspended sediment load and average water year bed material size at Yankton, South Dakota, Sioux City, Iowa, and Omaha, Nebraska.

	Yankton tons	D ₅₀ (mm)	Sioux City tons	D ₅₀ (mm)	Omaha tons	D ₅₀ (mm)
1940	49,708,160				78,339,700	
41	139,040,400				139,469,200	
42	201,335,530				230,421,700	
43	149,372,550				179,617,700	
44	230,300,900				251,678,300	
45	89,846,800				156,537,800	
46	92,324,530				109,101,600	
47	169,289,570				176,173,920	
48	138,191,700				150,074,600	
49	102,229,930				120,558,450	
1950	147,674,220	0.359			159,551,910	0.718
51	108,940,000	.348			219,458,870	.313
52	174,534,500	.456			157,962,850	.782
53	58,811,400	.359			80,535,400	.219
54	26,752,870	.321			37,292,590	.302
55	8,942,470	.966	12,134,530	0.244	25,492,410	.217
56	4,389,480	.802	14,148,090	.263	23,312,190	.257
57	1,740,960	1.003	8,047,150	.295	27,992,230	.228
58	1,258,600	.421	7,306,160	.271	19,104,800	.217
59	1,611,350	.509	10,642,000	.277	28,803,650	.225
1960	910,220	.586	14,073,900	.275	30,889,590	.289
61	1,113,600	.491	6,522,800	.266	25,623,900	.215
62	847,753	.556	12,408,400	.287	43,527,450	.226
63	1,072,475	.523	6,957,190	.293	30,158,970	.213
64	1,326,536	.457	7,949,770	.292	26,254,400	.233
65	1,071,280	.450	9,226,770	.329	36,602,400	.248
66	1,232,170	.418	11,791,220	.296	25,651,100	.250
67	1,009,850	.430	13,336,260	.308	31,876,000	.237
68	1,132,420		11,376,700	.311	20,687,300	.248
69	710,580		17,839,970	.296	38,830,100	.280
1970			14,871,960	.321	20,887,300	.287
71			23,682,700	.318	27,975,560	.239
72			16,265,800	.341	28,488,300	.243
73			8,842,530		20,338,690	

Nebraska (Table 1). A major factor in the size of the bed material in the lower Missouri River is the Platte River. Upstream of the Platte the bed material is considerably finer than downstream. Bed material size as a function of distance is given in Fig. 3.

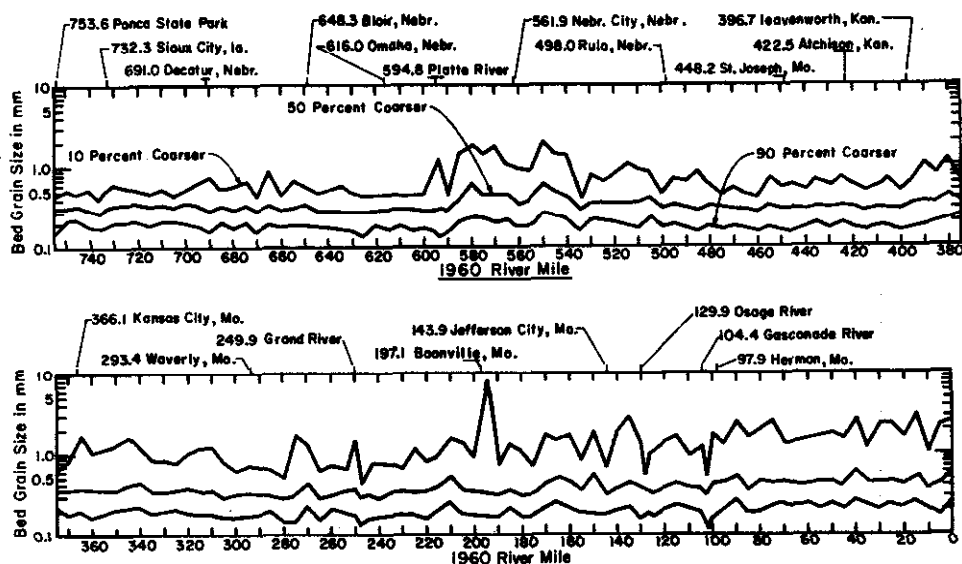


Fig. 3. Bed material size as a function of distance upstream of mouth, (1971, 1972, 1973 sampling observations and a composite of 5 to 7 verticals).

NAVIGATION CHANNEL

To control erosion and provide for navigation, the lower Missouri River was changed from a braided multi-channel stream with constantly shifting thalweg to a sinuous single channel stream. The width of the natural braided channel ranged from about 1000 ft where the river valley was narrow to as wide as 10,000 ft where the valley was wide. In addition the braided channel would shift its position thousands of feet from year to year in the broad alluvial valley. The river valley, bluff to bluff, ranges in width from 1/4 mile to 15 miles. The change from a braided channel to sinuous channel is illustrated in Fig. 4.

The river's width was established at about 600 ft in the upper reaches to around 1000 ft in the lower reaches near the mouth. Within, these widths a navigation channel 300 ft wide and 9 ft deep was established. River width was established and maintained by dikes and revetments constructed along both banks of the river. Velocities in the navigation channel segment range from 3.5 to 7.2 ft per second depending on river mile, discharge, and time of year (Fig. 5). The change and control of the river was made by working with the flow. Pervious pile dikes were constructed out from the banks, some as long as 3,000 or 4,000 ft to the predetermined new bank line. The dikes slowed the flow so that the silt was deposited behind them forming a new bank which soon became vegetated. To hold the

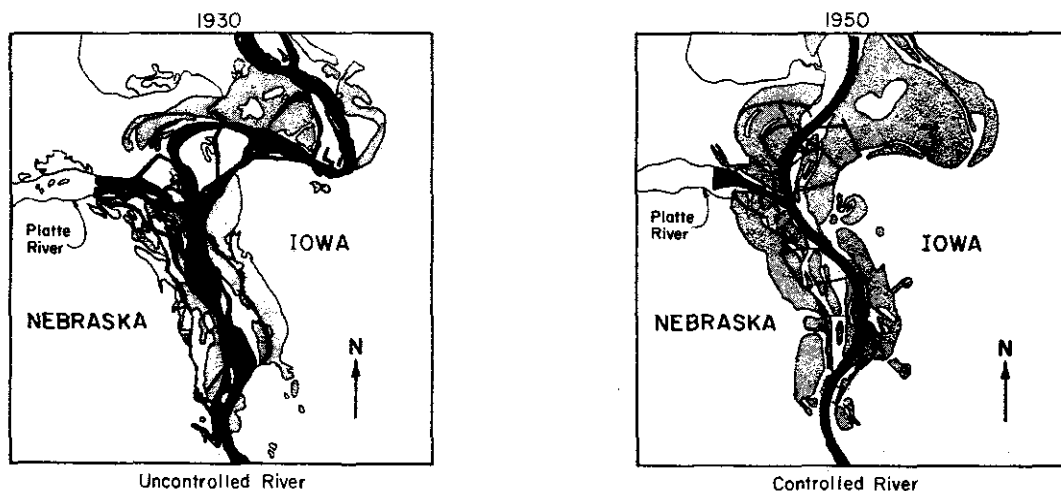


Fig. 4. Missouri River near Plattsmouth before and after improvement.

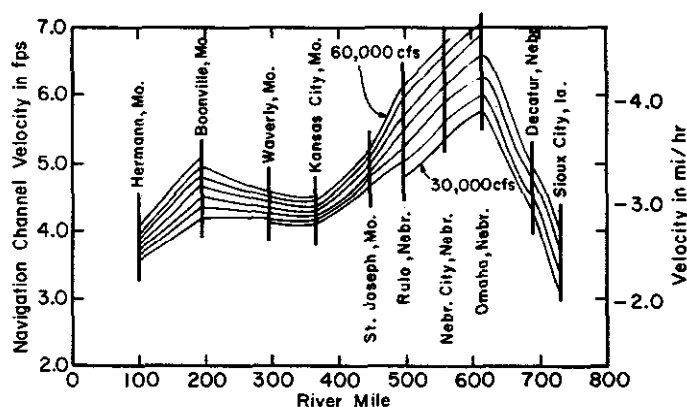


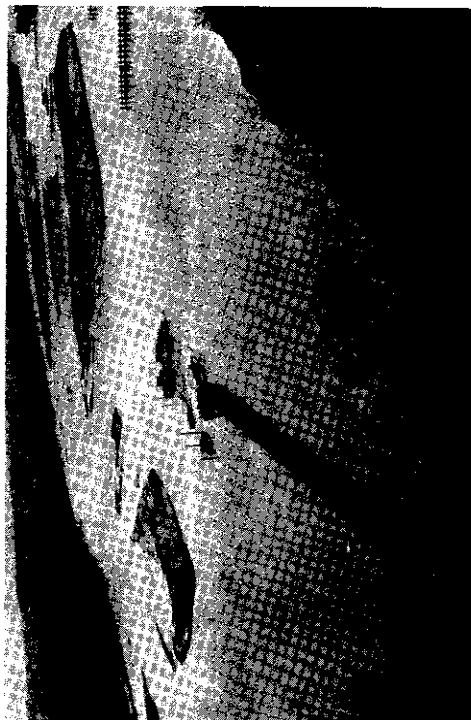
Fig. 5. Mean velocity in the Missouri River navigation channel, (1960-1969 average, June through August period).

predetermined width and alignment where velocities were large, stone dikes were used. To protect the concave banks on the bend from erosion continuous revetment of asphalt, pile, or stones were used, with stone revetment the predominant material. A typical channel alignment and control with dikes and revetment is illustrated in Fig. 4. How the channel changes were made by working with the river is illustrated in Fig. 6.

The desirable width of channel is a compromise between a satisfactory depth and velocity for navigation, satisfactory depth, width, and velocity to transport the sediment without scour or deposition, and finally a channel that will transport the flood flows expected. The width must provide sufficient confinement of the flow to prevent low-water meandering of the flow and sufficient velocities to transport the sediment. But too large a contraction can cause scour, high velocity, and overbank flooding. Fortunately, for the Missouri, the main stem dams provide control of the high and low flows, specially in the reach upstream of Kansas City.



September, 1934



November, 1934



August, 1936



May, 1946

Fig. 6. Establishment of erosion control and navigation channel on the Missouri River.

Considerable study was made of the natural channel to establish the proper width and in fact adjustments are made in selected reaches as experience dictates. In 1881 Col. C.R. Suter recommended a low water width of 650 ft and high water width of 820 ft at Sioux City increasing to low and high water widths of 1020 ft and 1160 ft at Kansas City, maintaining these widths to River mile 110 and then 1100 ft and 1240 ft to the mouth (Freeman, 1946). In 1908 Captain E.H. Schultz stated that channel 10 ft deep could be obtained below Kansas City with a width of 858 ft. A special board of engineers in 1910 recommended a width of 2000 ft but this was entirely unsatisfactory because of bar formation at low flows. Finally by studying the natural widths at the most restricted points on various sections of the river the following widths, which increased as drainage area increased, were suggested (Freeman, 1946).

<u>River Mile</u>	<u>Contracted Width</u>
760 to 514.4	700 feet
514.4 to 381.9 (Kansas River)	800 feet
381.9 to 251.8 (Grand River)	900 feet
251.8 to 129.0 (Osage River)	1,000 feet
129.0 to 0.0	1,100 feet

These widths are wider than what was finally established. Channel widths at main gages and how they have slowly become narrower with experience is illustrated in Fig. 7.

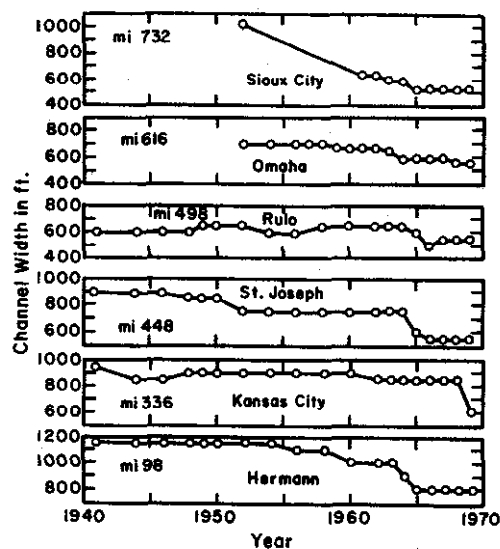


Fig. 7. Minimum channel width at selected gates on the Missouri River (channel survey was 1/2 mile up and downstream of the gage).

Changing a braided channel to a sinuous channel in general will decrease channel length. Change in thalweg length for selected places are as follows:

	1960	1941	1890
Sioux City, Iowa	732.3	760.0	807.5
Omaha, Nebraska	615.9	632.0	659.8
Rulo, Nebraska	498.0	514.4	537.5
St. Joseph, Missouri	448.2	460.0	478.7
Kansas City, Missouri	366.1	373.5	386.7
Jefferson City, Missouri	143.9	143.0	151.3

Of the total change in thalweg length from 1890 to 1960 63 percent occurred prior to 1941 and resulted from natural cutoffs, constructed cutoffs and general river alignment activities.

Slopes in selected reaches in ft/ft are as follows:

Reach	1943	1950	1957	1973
Sioux City to Omaha	0.000171	0.000171	0.000174	0.000180
Nebraska City to St. Joseph	.000184	.000184	.000182	.000192
Kansas City to Boonville	.000156	.000156	.000156	.000167
Boonville to Herman	.000163	.000159	.000159	.000158

The steepest slope is in the reach of the river downstream from the Platte river (Nebraska City to St. Joseph). Bed material in this reach is coarser than the reach upstream, Fig. 3.

The channel changes and decreased sediment inflow into the Lower Missouri River has, as anticipated by engineers, caused some degradation of the channel bed. The changes in stage resulting from the degradation are illustrated in Fig. 8 for the Sioux City, Iowa gage. For the period 1930 to 1974 stage at Sioux City has decreased from 7 to 10 ft depending on the discharge. Gages below Blair, Nebraska, which is 84 miles downstream of Sioux City, do not indicate any significant degradation of the bed. It is interesting to note in Fig. 8 that after closure of Gavins Point Dam there was an increase in stage for a few years followed by the decrease. The increase in stage probably resulted from the deposition of material scoured from the channel immediately downstream of Gavins Point Dam. The size of the bed material is increasing in the Sioux City reach as the river degrades; see Table 1.

The degradation of the bed from 7 to 10 ft over the past 40 years has not caused any serious problems to date. It has decreased flood stages. Nevertheless the changes in bed elevation and stages in the Missouri River are being monitored and studied to determine the ultimate change in bed elevation and what if any remedial action will be necessary.

It is well known that water temperature has a significant effect on the bed configuration, resistance to flow and sediment transport (Straub, 1954; Simons and Richardson, 1966; Hubbel and Al-Shaikh-Ali, 1961; and others). Temperature by changing water viscosity and thus the fall velocity has significantly affected the stage and depth of the lower Missouri River. This is illustrated in Fig. 9 (U.S. Corps of Engineers, 1968b). With a constant or even a slight increase in discharge the stage at selected gages in the Sioux City, Iowa to Rulo, Nebraska reach

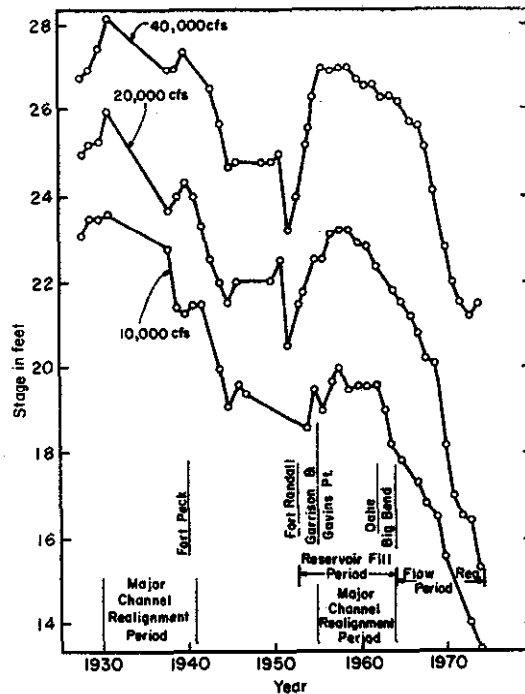


Fig. 8. Change in stage with time for the Sioux City, Iowa gage.

decreases in the fall. The temperature decrease is from 30° to 40° from late August to September. This temperature change causes the bed form to change from dunes to washed out dunes or plane bed. Consequently Manning's n has decreased from approximately 0.020 to 0.015, average velocity increased from 4.5 to 5.3 ft/sec and depth decreased. The change in bed form from dunes to washed out dunes or plane bed has been verified by sonic sounding of the bed configuration. Average depth decreases about 1 ft in the fall. However minimum depths (water surface to top bed forms) remains about the same or slightly increases. Changes in bed forms during low water periods does cause navigation depth problems in a few crossings that have poor efficiencies. When additional water is available it is released during the fall to increase depth or is used to extend the navigation season if weather is favorable.

SUMMARY

The Lower Missouri River from Sioux City, Iowa (mile 732) to its confluence with the Mississippi River, was changed from a multi-channel, constantly shifting, unstable, erosive, braided river to a single-channel, stable, sinuous river. A virtually useless river--useful only for local water supply and to transport domestic waste--was stabilized for erosion control, navigation and flood control. The development of the lower river was aided immensely by the construction of six main stem dams with a total storage capacity of 75,240,000 acre-feet. This storage capacity is 3.3 times the average annual flow at Sioux City, Iowa and 1.3 times the flow at the mouth. The dams and development of the lower river significantly decreased the sediment load of the river.

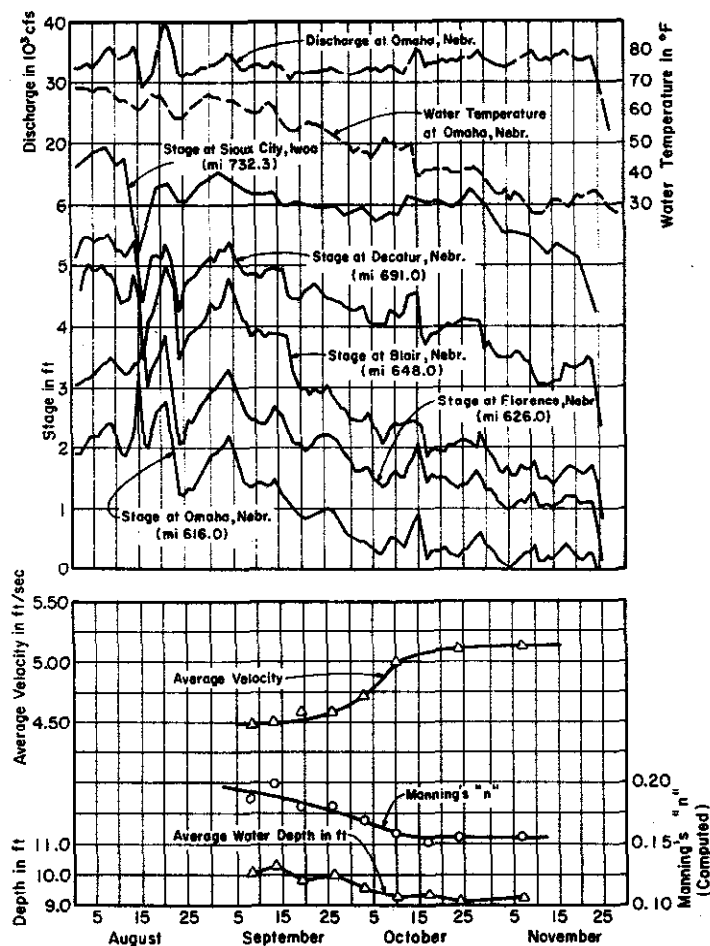


Fig. 9. Change in stage, velocity, Mannings n and depth with temperature and discharge. Velocity, Mannings n and depth are for reach of river from mile 609 to 616.

The channel changes were accomplished by working with the river. Pervious pile dikes, some longer than 4000 ft, were built in the braided channel from both major banks to obtain the desired new channel width and alignment. The silt in the water deposited behind the dikes forming new land which became vegetated. Stone dikes and bank revetments were used to control erosion where the velocities were large.

Channel alignment and width were determined by study of the natural river and observing its response to change. The channel between dikes has a width of around 600 ft at the upper end and around 1000 ft at the mouth. The change in width increasing with drainage area. The navigation channel is 9 ft deep and 300 ft wide.

The thalweg distance from Sioux City, Iowa to the confluence was decreased 75 miles (mile 807.5 in 1890 to mile 732.3 in 1960). Slope consequently increased but there is no single river slope from Sioux City to the mouth. Tributaries, changes in bed material, river discharge, degradation, etc. all cause the slope to be different from

reach to reach. The steepest slope in the lower Missouri River is downstream of the Platte River (Nebraska City, Nebraska to St. Joseph, Missouri reach slope was 0.000192 ft/ft, 1973) and the flattest slope is downstream of Kansas City (Kansas City to Herman, Missouri reach the slope was 0.000164 ft/ft, 1973).

The construction of the dams and channel control have caused the channel to degrade from below Gavin's Point Dam (mile 811) to below Sioux City (mile 732). The decrease in stage resulting from this degradation is between 7 and 10 ft for discharges between 10 and 40,000 cfs. This degradation has occurred over a 40 year period (1933-1973) and appears to be continuing. Downstream of Blair, Nebraska (mile 648) significant degradation of the channel is not apparent.

Temperature changes of the river water have a significant affect on the bed configuration. A temperature decrease of between 30° to 40° F occurs every fall. This decrease changes a dune bed to washed out dunes and almost to a plain bed. This decreases Manning's n ($n \approx 0.02$ to $n \approx 0.015$), decreases depth (1 to 2 ft) and increases average velocity (4.5 to 5.3 ft/sec) for a constant discharge between 30 and 35,000 cfs. These changes occur in the reaches of the river upstream of Rulo, Nebraska (mile 498) and require an increase in flow in the fall of the year to maintain navigation depths.

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PATTERNS OF SCOUR AND FILL IN POOL-RAPID RIVERS

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ABSTRACT

Because the change in head on a critical control (rapid or weir) is less than the change in equilibrium depth in the pool upstream, a pool will eventually scour with an increase in flow and fill with a decrease in flow. However, because the supply of sediment to a pool is dependent on the conditions at the outlet of the pool just upstream, initially there may be either scour or fill in the downstream pool with an increase in flow. Indeed, a pool following several other pools may behave in a seemingly erratic manner because its sediment supply is affected by what happens in each of the upstream pools.

INTRODUCTION

The pool-rapid river consists of short and medium length reaches at subcritical flow separated by rapids where the flow rate is supercritical. The Colorado River through the Grand Canyon is a prime example of this kind of river, and there are many others.

Since each reach has a downstream control, a natural first impression is that each reach should be hydraulically independent except for the attenuation of the hydrograph by the reservoir effect—assuming, of course, that the controls are not drowned out at high water. However, because sediment is transported in a real river, the reaches are not independent; the behavior of any reach is dependent on what happens in (and, therefore, on the characteristics of) all upstream reaches.

The characteristics of the imaginary pool-rapid river studied in this investigation were completely arbitrary—but not unlike the Colorado River. All reaches were 16 kilometers (10 miles) long. The cross sections were assumed rectangular with widths of 120 or 180 meters (400 or 600 feet). The banks were considered to be inerodible rock walls, as is true in much of the Grand Canyon. The erodibility of the bank in general is a factor determining the width of a stream, and widths can change during extreme floods if the banks are erodible. These effects were put aside in this investigation. A slope of 0.00005 and an n value of 0.03 were used as these gave velocities and depths which seemed reasonable.

The controls were approximated by either a parabolic or triangular weir giving different, but reasonable, head-discharge relationships.

$$Q = 55.8 h^2 \quad (1)$$

$$Q = 28.4 h^{2.5} \quad (2)$$

where Q , the discharge, is in cubic meters per second, and h , the head, is in meters.

SOME CHARACTERISTICS OF SEDIMENT-TRANSPORTING STREAMS

Alluvial streams transport sediment as well as water. Figure 1 illustrates some of the resulting stream behavior for a bed material typical of the Colorado River, several widths and slopes and an n value of 0.03. The Laursen (1958) relationship was used in the preparation of this figure but with the hydraulic radius R used to evaluate the shear velocity term and in the Manning equation. The width was found to be surprisingly ineffective as a determining factor in the load-discharge relationship. Indeed, this implies that the slope and sediment composition (and probably the Manning n) together with the "dominant" discharge might be sufficient to estimate the watershed sediment yield of bed material. An estimate of the change in bed material due to "wash load" during storms would be needed to guess at the total sediment yield.

Figure 1 also indicates that the slope in a long contraction is not much different than the slope of the uncontracted stream. Using the DuBoys and Manning equations and the approximations that hydraulic radius equals depth ($R = y$) and critical tractive force is much smaller than the total shear ($\tau_c \ll \tau_0$), Straub (1940) obtained an expression for the depth ratio as a power function of the width ratio of the uncontracted and contracted reaches.

$$y_2/y_1 = (B_1/B_2)^{9/14} \quad (3)$$

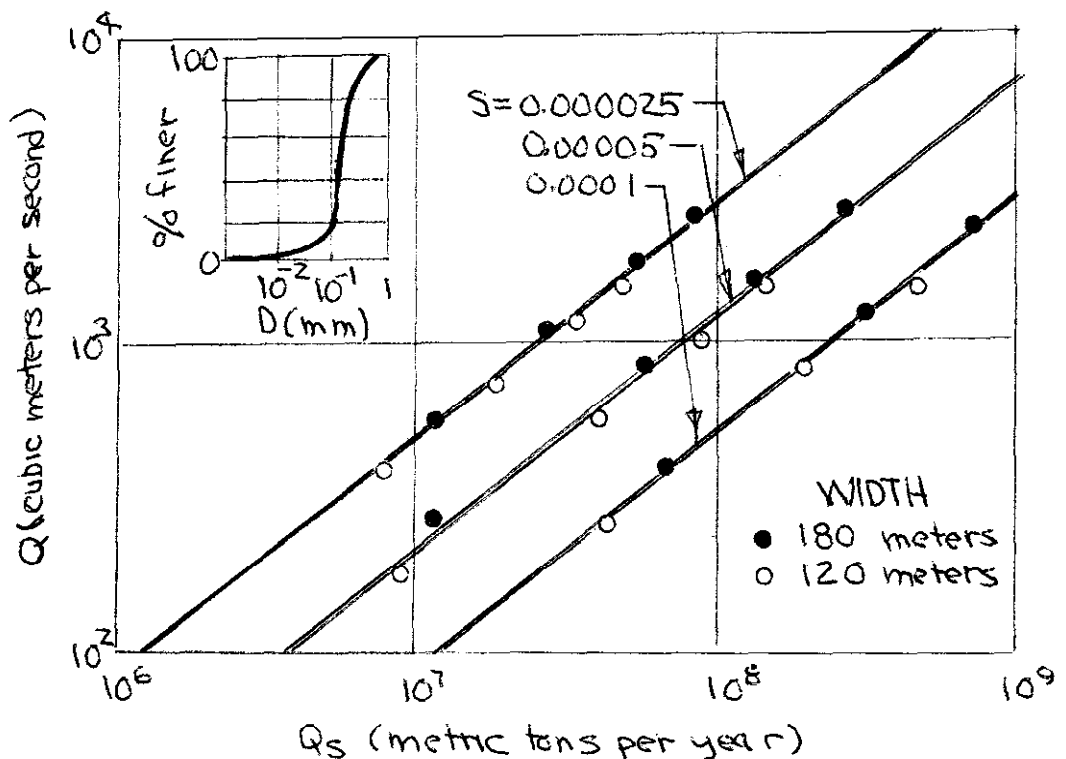


FIGURE 1. LOAD-DISCHARGE RELATIONSHIPS

This analysis can be extended to find a similar expression for the slope ratio, assuming n is the same in the two reaches,

$$S_1/S_2 = (B_1/B_2)^{1/7} \quad (4)$$

Letting $R = Ky$ and $(\tau_0'/\tau_c) - 1 = C(\tau_0'/\tau_c)$ and approximating the shear velocity/fall velocity function in the Laursen relationship by power functions with the exponents 1/4, 1, and 9/4 when that velocity ratio is $<1/2$, $=1$, and >2 , similar expressions result: (Note that τ_0' is the particle shear, not the total shear τ_0).

$$\sqrt{gRS}/w < 1/2 \quad S_1/S_2 = (C_2/C_1)^{0.88} (K_2/K_1)^{1.30} (n_1/n_2)^{1.78} (B_1/B_2)^{-0.02} \quad (5)$$

$$y_2/y_1 = (C_2/C_1)^{0.26} (K_2/K_1)^{-0.01} (n_1/n_2)^{-0.07} (B_1/B_2)^{0.59} \quad (6)$$

$$\sqrt{gRS}/w = 1 \quad S_1/S_2 = (C_2/C_1)^{0.71} (K_2/K_1)^{1.21} (n_1/n_2)^{1.29} (B_1/B_2)^{0.14} \quad (7)$$

$$y_2/y_1 = (C_2/C_1)^{0.21} (K_2/K_1)^{-0.04} (n_1/n_2)^{-0.21} (B_1/B_2)^{0.64} \quad (8)$$

$$\sqrt{gRS}/w > 2 \quad S_1/S_2 = (C_2/C_1)^{0.54} (K_2/K_1)^{1.13} (n_1/n_2)^{0.78} (B_1/B_2)^{0.31} \quad (9)$$

$$y_2/y_1 = (C_2/C_1)^{0.16} (K_2/K_1)^{-0.06} (n_1/n_2)^{-0.37} (B_1/B_2)^{0.69} \quad (10)$$

K and C values are shown in Figure 2. Although the exponents of the C , K , and n ratios are in the order of 1, the ratios themselves should not be too different from 1. Moreover, $C_2 > C_1$ while $K_2 < K_1$ and probably $n_1 > n_2$ (unless they are equal), so in general the slope in the contraction is slightly less than in the uncontracted (normal) stream. The difference in the slopes is small, and the difference in the difference with changing discharge should be even smaller. Slope changes are probably seldom observable except during the active period of scour and fill before equilibrium depths and velocities are attained.

Width changes have a considerably greater effect on depth; and not only is the depth greater in the contraction, but with an increase in discharge the contraction scours and the expansion beyond fills (and vice versa with a decrease in discharge). The exponents on the K , C , and n ratios are such that the effects of these ratios are generally negligible.

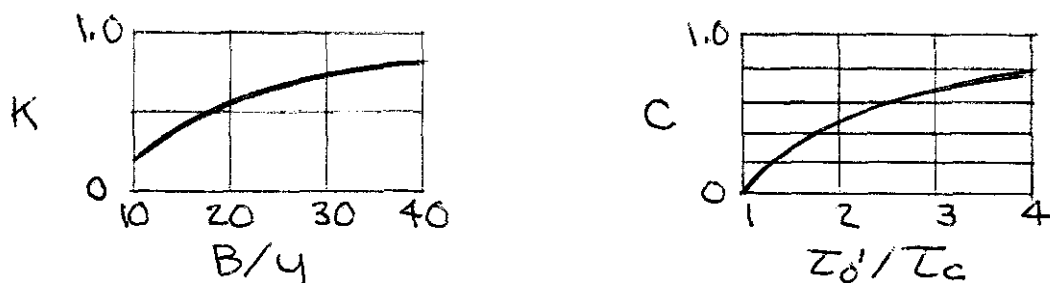


FIGURE 2. K & C VALUES

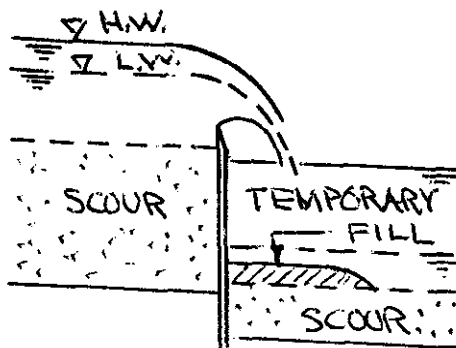


FIGURE 3. POOL-RAPID-POOL

The special characteristic of the pool-rapid reach is that the increase in head is not as great as the increase in equilibrium depth when there is an increase in discharge. As shown in Figure 3, the pool eventually scours out with an increase in flow (and fills with a decrease in flow). However, if the control or the width of the next pool downstream is different, the next pool may fill instead of scour, or scour to an interim equilibrium depth while the upstream pool is actively scouring and, therefore, delivering a non-equilibrium supply of sediment to the downstream pool. If the particular reach in

question follows several pool-rapid reaches, the bed can fill and scour in what seems to be an inexplicable fashion without further change in the discharge. If the hydrograph goes up and down faster than the reaches can scour and fill to equilibrium, the pattern of scour and fill can be even stranger because residual holes and humps will tend to move through the system.

The time for a flood wave to pass through the system is approximately the quotient of the length of the system divided by the velocity of flow; minutes, hours, or days depending on the particular river. The time to reach equilibrium as a result of scouring or filling is approximately the quotient of the total volume of scour or fill divided by the difference between the supply of sediment from upstream and the capacity to transport sediment in the reach or reaches under examination; days, weeks, months, or even years depending on the particular situation.

In order to better understand the behavior of the pool-rapid river, several simpler cases were studied, each illustrating some facet of the more complex problem.

CASE I. A SET OF INERODIBLE LONG CONTRACTIONS AND EXPANSIONS

To test the degree of nonuniformity of flow after the flood wave has passed but before much of the scour and fill has taken place, the set of contractions and expansions shown in Figure 4 was examined as if the bed and bank were fixed in the equilibrium configuration associated with the previous low water flow. The reaches were assumed 16 km (10 miles) long except the first and last which were very long. With a low flow rate of 140 cubic meters per second (5000 cfs), a slope of 0.00005 in the expansions and an n value of 0.03, the depth in the 180 meter (600 ft) wide expansion reaches was 2.1 meters (6.8 ft) and in the contractions 2.5 meters (8.3 ft). At a flow of 700 m^3 /sec (25,000 cfs) the normal depth of flow in the expansions is 5.5 meters (18 ft) and in the contractions 7.1 meters (23.3 ft).

The normal depth of flow in the last long expansion would be the control for the system. The losses at the transitions between reaches and the changes in velocity head are small enough to be neglected. Therefore, the depth at the end of the upstream contraction would be 5.9 meters and would rise in a drop-down backwater curve to 6.4 meters at the beginning of the contraction—less than the normal depth. In the middle expansion the depth at the lower end

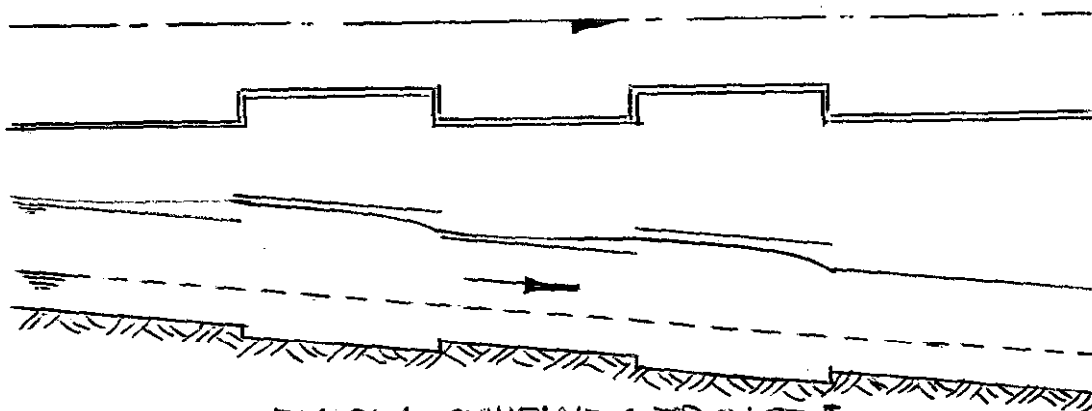


FIGURE 4. SCHEMATIC FOR CASE I.

would be 5.9 meters which would reduce to 5.8 meters at the beginning of the reach. In the next contraction upstream, the depths would be 6.2 at the tail end and 6.6 at the head end. At the tail end of the first expansion, the depth would be 6.2 meters and there would be a long backwater curve reducing the depth to the normal depth of 5.5 meters. Note that only in the downstream, control reach is the flow uniform. Otherwise, everywhere in the system there are differences in sediment-transporting capacity and a tendency to scour or fill.

CASE II. A SET OF ERODIBLE LONG CONTRACTIONS AND EXPANSIONS

If the bed of Case I is erodible and the stream is carrying a sediment from upstream, when the flow is increased there will be scour and fill until the depths and velocities are such that the same capacity to transport sediment exists everywhere. Because of the nonuniformity of the flow, there will be some reworking of the bed; however, this effect will disappear in the general scour in the contractions and fill in the expansions. The equilibrium depths would be 5.5 meters and 7.1 meters (23 ft) in the expansions and contractions. As shown in Figure 5, the material scoured out of the contractions would fill the next expansions. The fill in the last long expansion would gradually be carried on downstream, getting thinner and thinner so that it practically disappears. This would lower the control and all other expansions and

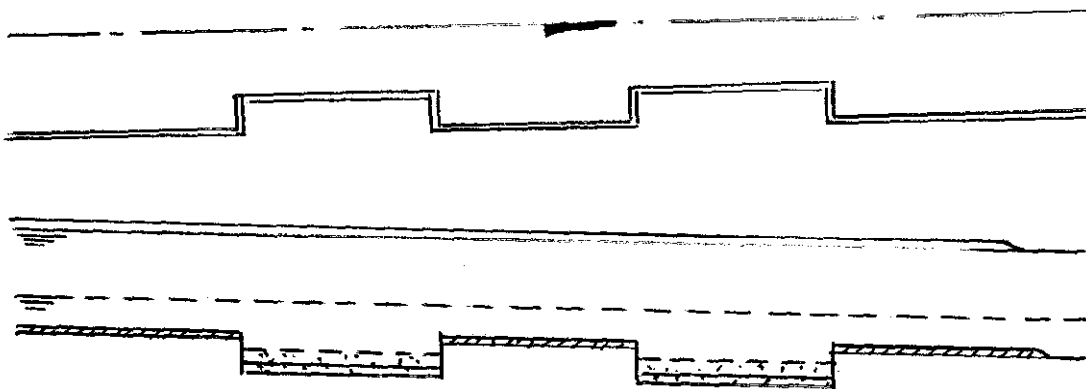


FIGURE 5. SCHEMATIC FOR CASE II.

contractions would gradually lower. At first there would be a scour of about 0.7 meters and a fill of about 0.5 meters. Eventually the scour would be 1.2 meters and there would be little or no fill.

If the bed material is typical of the Colorado River, the sediment supply from upstream at a discharge of $700 \text{ m}^3/\text{sec}$ would be 50×10^6 metric tons per year. Because of the backwater, initially the supply rate to the first contraction would be only 27×10^6 metric tons per year. The initial sediment-transporting capacity at the head of the first contraction would be 86×10^6 metric tons per year. It would therefore take something more than 14 days for the contraction to scour out. Values of supply and capacity for the other reaches would be about the same and scour and fill would be occurring in all reaches simultaneously. For all the adjustments to occur, probably a month rather than two weeks would be needed.

CASE III. A SET OF POOLS WITH THE SAME WIDTHS BUT DIFFERENT CONTROLS

Consider first the simplest set of pools, all the same width (say 120 meters), the same length (16 km) except the first pool which is very long and all with the same control (parabolic weir). At a flow of $140 \text{ m}^3/\text{sec}$, the head on the control is 1.5 meters and the equilibrium depth is 2.5 meters, so the bed is 1.0 meters below the control crest. At a flow of $700 \text{ m}^3/\text{sec}$, the head on the weir is 3.4 meters and the equilibrium depth is 7.1 meters, so the bed ultimately will be 3.6 meters below the crest, thus the scour in each pool will be 2.6 meters (8.5 ft). Shortly after the flow has changed, the depth of flow at the tail end of each pool will be 4.4 meters while, because of backwater, the depth of flow at the head end of the pools will be 5.9 meters. Each pool will deliver more sediment than the next downstream pool can handle. Therefore there will be deposition at the head of each pool. Within each pool initially the capacity to transport sediment will be greater at the tail end than the supply coming from the head end. Therefore there will be scour at the tail end of each pool. The wedge shape of fill and of scour will result in a greater slope, uniform flow at a lesser depth and greater velocity, and a greater capacity to transport sediment than the ultimate equilibrium conditions. This will happen simultaneously in all pools and a temporary equilibrium will have been established. However, in the first long pool, there is no deposition and the sediment supply from upstream is that of the final, ultimate equilibrium condition. Scour will continue at the tail end of this long pool and the sediment supply to the second pool will decrease. The head end of the second pool will then scour also, and the scour hole will get deeper and longer until the entire pool has scoured to the equilibrium condition. Nothing will happen in the next downstream pool until the scour hole in the upstream pool reaches the control at the tail end; then the next downstream pool will begin to scour in the same way. Each downstream pool will be delayed until the pool just upstream has scoured out. Thus even for the simplest geometry, the essential character of pool-rapid river behavior is evident, and if the hydrograph changes before the entire system has attained equilibrium, the behavior will be even more complex.

Case III, as shown in Figure 6, is changed from this simple case in that the first and third pool controls are triangular weirs, whereas the second and fourth pools are controlled by parabolic weirs. For a flow of $1400 \text{ m}^3/\text{sec}$ (50,000 cfs), the equilibrium depth for all pools is 11.0 meters; the head on

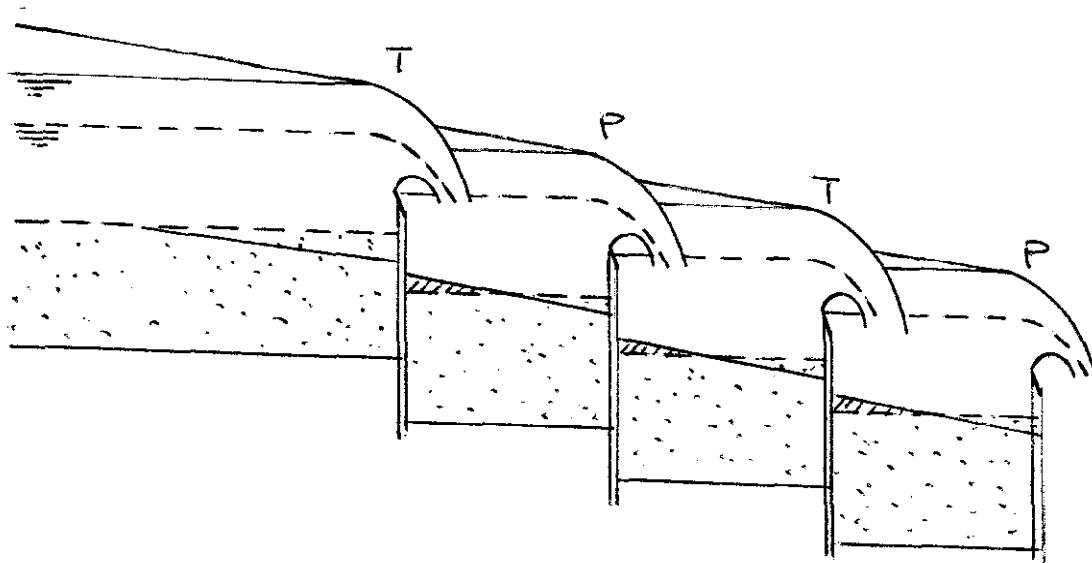


FIGURE 6. SCHEMATIC FOR CASE III.

the parabolic weirs is 4.8 meters, on the triangular weirs is 4.9 meters. The bed is 6.2 meters below the crest of the parabolic weir and 6.1 meters below the crest of the triangular weir. For a flow of $5600 \text{ m}^3/\text{sec}$, the equilibrium depth in all pools will be 27.1 meters; the head on the parabolic weir 9.1 meters and on the triangular weirs 9.7 meters. Ultimately the pools with the parabolic weirs will scour out 11.8 meters, and the pools with the triangular weirs will scour out 11.3 meters.

Immediately after the high flow is established, the depth at the tail end of the first long pool will be 15.8 meters and the depth at the head end of the next pool will be 18.3 meters. Therefore the head of the second pool will fill as the tail of the first pool scours. The initial depth at the tail of the second pool will be 15.3 meters; therefore the tail of the second pool scours. The heads of the third and fourth pools will also fill and the tails scour. Because the controls are different, the amount of scour and fill in each pool will not be quite the same and the time needed to attain this temporary equilibrium condition will vary a little. As in the simple case, the first pool continues to scour toward equilibrium and, as the supply to the second pool decreases, the head end begins to scour towards equilibrium. With only slight variations, the progress of the scour through the successive pools and from pool to pool will be the same as in the simple case. It is conceivable that the controls could be different enough so the depth at the head of one pool would be equal to or less than the depth at the tail end of the next upstream pool. In this circumstance, the head of the pool would not first fill but would hold steady for a while or immediately begin to scour. It is also conceivable that the controls could be so different that a downstream pool would exhibit very erratic behavior as the sediment supply originating in various upstream pools reached it.

CASE IV. A SET OF POOLS WITH DIFFERENT WIDTHS AND THE SAME CONTROLS

This case differs from the previous one in that the heads on the controls for the pools are the same, but the equilibrium depths are different because the widths of the pools are different. The situation is shown in Figure 7: in

the 120 meter width pools, the equilibrium depth at $1400 \text{ m}^3/\text{sec}$ is 16.0 meters and 27.1 meters at $5600 \text{ m}^3/\text{sec}$. In the 180 meter width pools, the equilibrium depth at $1400 \text{ m}^3/\text{sec}$ is 8.5 meters and 19.8 meters at $5600 \text{ m}^3/\text{sec}$. The head on the controls at the low flow is 4.9 meters and at high flow is 9.7 meters. In the narrow pool, the bed is 6.1 meters below the crest at low flow and 17.4 meters at the high flow; therefore the ultimate scour is 11.3 meters. In the wide pool, the bed is 3.6 meters below the crest at the low flow and 10.1 meters below at the high flow; so the ultimate scour in the wide pool is 6.5 meters.

At the high flow rate the supply of sediment from upstream is 900×10^6 metric tons per year, and this would finally be the sediment transport rate throughout the system when equilibrium is established. Initially at the tail of the first long pool, the transport rate is 5760×10^6 metric tons per year; therefore, scour would occur, deepening and lengthening upstream until a depth of flow of 19.8 meters is attained. At the head of the next narrow pool, the capacity initially is 6660×10^6 metric tons per year, and this section also scours. The tail end of this second pool has a capacity of 9000×10^6 metric tons per year; the entire pool scours and in the process steepens. The head of the next wide pool has a capacity of 3060×10^6 metric tons per year and it fills, while the tail end with a capacity of 5760×10^6 metric tons per year scours. The next narrow pool has a capacity of 6660×10^6 metric tons per year at the head and 9000×10^6 metric tons per year at the tail and fills at the head and scours at the tail.

After temporary equilibrium is approximately established, the capacity in the second pool would be less than it was initially, about 5500×10^6 metric tons per year, in the third pool about 4400×10^6 metric tons per year and in the fourth pool about 7800×10^6 metric tons per year.

The supply rate from the first pool would be less than initially and less than the capacity in the second pool; therefore, the second pool should continue

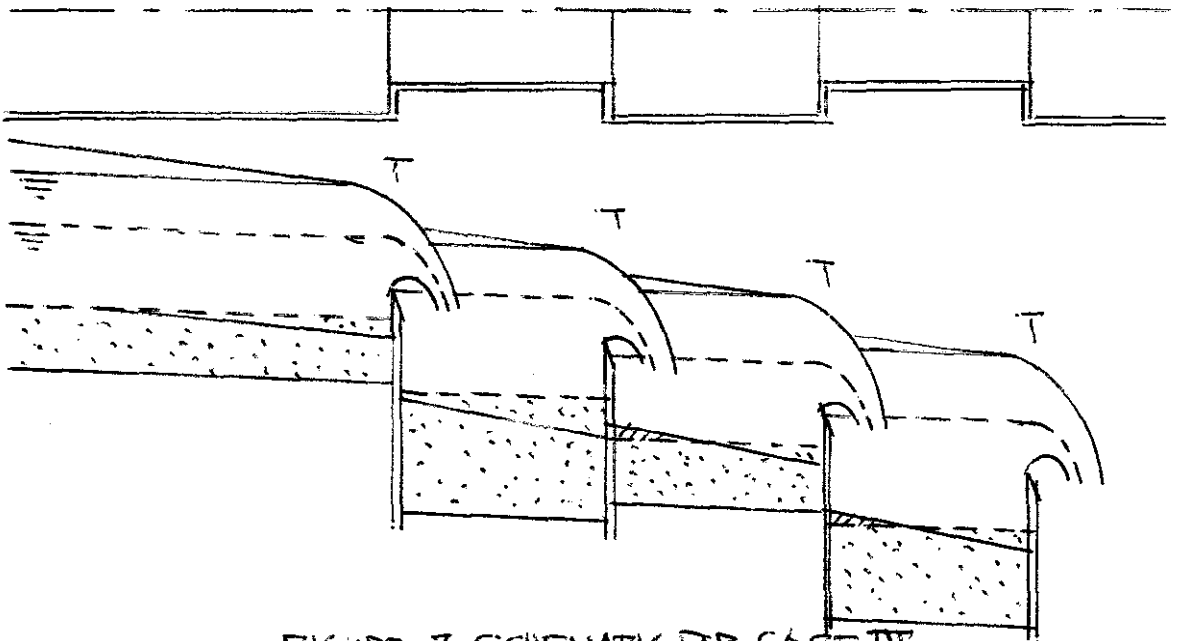


FIGURE 7. SCHEMATIC FOR CASE IV.

scouring. The capacity in the third pool would be less than the supply from the second pool and it would fill. The fourth pool would have a greater capacity than the supply and it would scour.

CASE V. A SIMPLE POOL-RAPID RIVER

The pool-rapid river shown in Figure 8 is a combination of the two previous cases; the wide (180 meter) pools have a triangular control, the narrow (120 meter) pools have a parabolic control. At a flow of $1400 \text{ m}^3/\text{sec}$, the head on the triangular control is 4.9 meters, the equilibrium depth in the wide pool is 8.5 meters, and the bed is 3.6 meters below the crest. At that low flow the head on the parabolic weir is 4.8 meters, the depth in the narrow pool is 11.0 meters, and the bed is 6.2 meters below the crest.

At a flow of $5600 \text{ m}^3/\text{sec}$ the head on the triangular weir is 9.7 meters, the equilibrium depth in the wide pool is 19.8 meters, and the bed will finally scour 6.5 meters to be 10.1 meters below the crest. The equilibrium rate of sediment transport would be 900×10^6 metric tons per year, but initially just above the control the depth will be 13.3 meters and the capacity will be 5760×10^6 metric tons per year. At the head end of the pool the depth will be 15.0 meters initially and the capacity will be 3060×10^6 metric tons per year.

At the high flow the head on the parabolic weir will be 9.1 meters, the equilibrium depth in the narrow pool will be 27.1 meters, and the bed will scour 11.8 meters to be 18.0 meters below the crest. The initial depths at the head and tail of the pool will be 18.3 meters and 15.3 meters, and the capacities will be 6300×10^6 and 17100×10^6 metric tons per year.

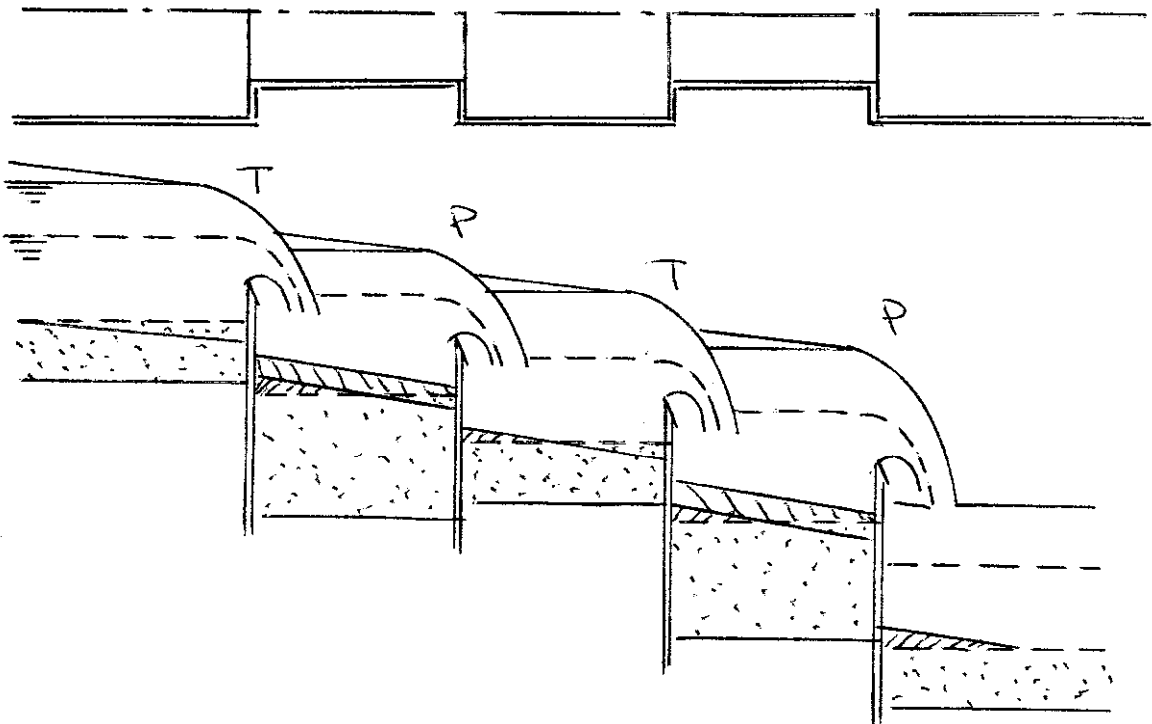


FIGURE 8. THE POOL-RAPID RIVER

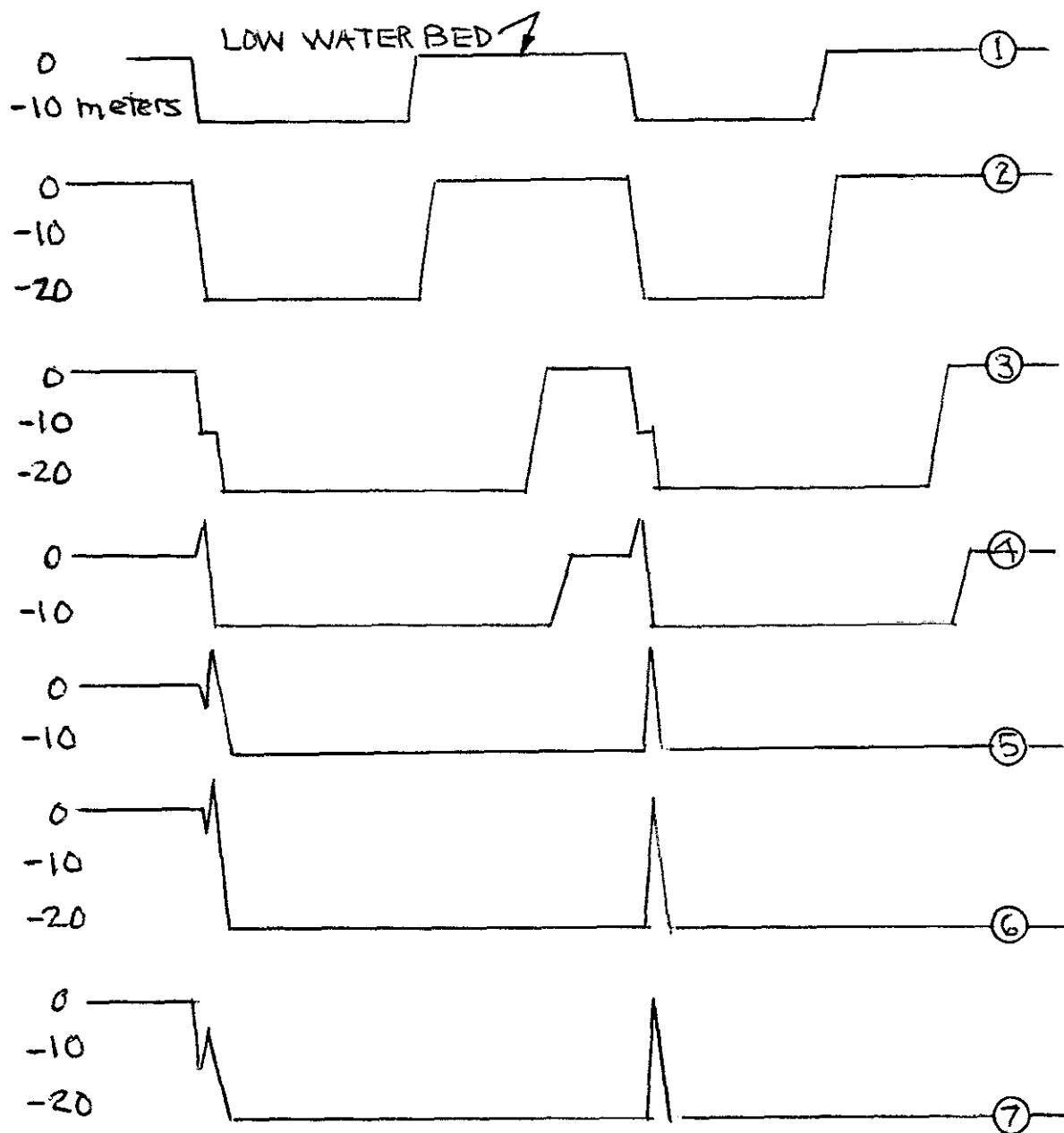
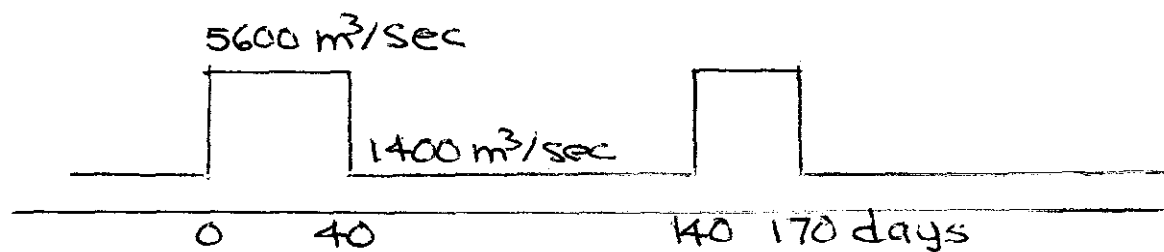


FIGURE 9. SCOUR & FILL VS. TIME

Figure 9 shows the bed elevations just above and just below the controls as a result of a hydrograph of two floods of $5600 \text{ m}^3/\text{sec}$ of 40 days and 30 days duration, separated by a low flow period of 100 days at $1400 \text{ m}^3/\text{sec}$ and that same low flow preceding and following the floods.

The first long pool scours out above the control and head cuts back. The scour hole fills from upstream and therefore there is a delay before the bed rises at the control. The second pool scours out both from the head and the tail. The lower scour hole enlarges faster but does not scour out to equilibrium depth as more than the equilibrium transport rate is being supplied to it. The upper scour hole goes down to the limit and when the two holes meet, the upper one proceeds on downstream. The second pool must wait until the first pool is refilled before it can fill.

The third pool first fills at the head, then scours; at the tail it first scours, then fills, then scours again. At the intermediate low flow only part of the pool fills; as a result on the second flood at the tail end a short-lived delayed hump appears and disappears. These humps, slightly delayed, appear and disappear at both the head and the tail of the fourth pool during the second flood. The head of the fourth pool scours, fills, and scours just like the tail of the third pool. Similar action occurs at the tail of the fourth pool but the fill does not come up to the low water bed level.

With a more complex river and a more complex hydrograph, it is easy to imagine very erratic patterns of scour and fill with time and space.

QUALIFICATIONS

Real pool-rapid rivers are more complex than considered herein. Pools can vary in width along their length; head-discharge relationships are not necessarily simple power functions and controls can be drowned out at high discharges. Tributaries can add flow at intermediate points; and can change the relative concentration and the composition of the sediment load.

Rock falls, cobbles and boulders can effectively riprap the bed at some elevation so scour cannot proceed and so an increased sediment load cannot be obtained. Analysis in such a case would have to consider the characteristics of clear-water scour.

Above and below the rapids the flow would be quite nonuniform, the supercritical flow leaving the rapids is likely to form a jet plunging to the bottom of the stream. Especially after the rapid, a local scour hole is to be expected, perhaps followed by a stationary dune. All such considerations have been ignored in this study.

CONCLUSIONS

The seemingly erratic behavior of pool-rapid rivers is subject to rational explanation. Because with increasing discharge, the equilibrium depth increases faster than does the head on the control, there will be scour in all pools if the flood lasts long enough. With decreasing discharge, of course, the opposite ultimate result—fill—is to be expected. Fill, however, takes

much longer to occur than scour because the difference between sediment supply and capacity to transport sediment is much smaller.

During the active phase of scour (or fill) the seemingly erratic behavior becomes apparent. Sections that will ultimately scour may first fill because scour upstream supplies a temporary unusual high load of sediment. Temporary differences in capacity for transport can be due to differences in pool width or to differences in controls. A pool which follows several pools may fill and scour several times before scouring to equilibrium depth, because no pool can make its final move to equilibrium until all pools above it have reached equilibrium.

If the hydrograph is not long enough, the scour or fill to the equilibrium condition will be interrupted, and holes or humps will be left in the system. These holes and humps will tend to move through the system.

The normal river with contractions and expansions but without periodic controls (rapids) behaves similarly except that the reaches are linked hydraulically and the scour and fill takes place simultaneously throughout the system.

The combinations of sediment supply and discharge (and sediment composition) impose a requirement for a certain slope to the stream. Width, therefore bank erodibility, has only a slight influence on the slope. The roughness, or Manning n , has a considerable influence but is measurable (or estimable). Therefore, slope and flow should allow an estimate of watershed sediment yield of bed material.

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