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An Approach for Estimating the Design Flood Flowline on Developed Rivers

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Abstract

The stages produced by the major floods on developed rivers (like the Rhine in Europe, the Mississippi and the Red rivers in the USA) indicate that existing stage-discharge relationships are usually several feet higher than the stage-discharge relationship if the rivers were natural or partially developed.

The objectives of this paper are to discuss the problem of estimating the flowline for the design flood on developed rivers, and in so far as is possible with existing data, point to a rational methodology for developing this estimate. The problem is approached in two stages. First, the stage-discharge relations in undeveloped and developed rivers are analysed and contrasted qualitatively to provide an indication of expected trends. Second, stage-discharge records on a developed river and an estimate of the energy gradient are combined with a computer solution of several methods for computing channel roughness to provide a plot of roughness (Manning's n) versus discharge. Extrapolation of this plot to the design discharge yields a range of roughness values that can be used to estimate flowlines associated with the design discharge.

Key words: River hydraulics, Developed rivers, Flood-stage relation, Maximum flood, Design discharge

Qualitative Analysis

The complexity of alluvial channel flow and the dynamic nature of river systems are reflected in the large number of interrelated variables necessary to describe flow in natural streams. The evolution of channel patterns and geometry together with continuous changes in bed forms in response to changing conditions of water and sediment discharge also add to the complexity of alluvial river systems. Because of this complexity, time or data limitations often preclude a detailed quantitative analysis of river response. As a result, the analysis of river response to natural change or human activity must be based, in many instances, on the indicators derived from a qualitative analysis, even where quantitative data are available for detailed studies of the long-term river response to engineering works. The qualitative approach constitutes a useful step in the analysis by establishing the general trends to be anticipated. This is particularly true in a study that involves the extrapolation of data beyond the range of field experience.

For this problem, the qualitative approach can be applied to reveal the most probable modification in hydraulic parameters such as stage, discharge and roughness as a result of changes imposed on a river system. The plan view of a typical meandering channel as shown in Figure 1 exhibits significant changes in the thalweg location between low-stage and high-stage flow. While the low-stage thalweg impinges on the concave bank of the bendway, the higher velocities and greater momentum of high-stage flows produce a thalweg that skirts the convex bank and cuts across the tip of the point bar, opening, in some cases, a chute channel across the bar.

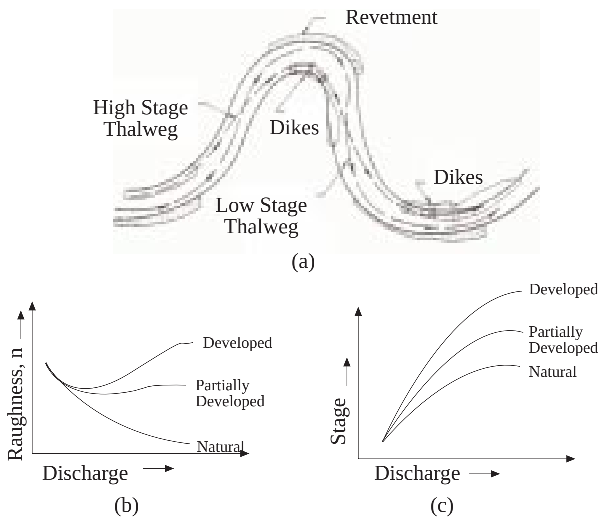

Figure 1. Change in hydraulic parameters in a developed reach

To stabilize the bendway and prevent the development of chute channels and divided flow reaches, it is common practice on developed rivers to construct dikes across the point bar as shown in Figure 1a.

To protect the concave bank from erosion by low-stage flows, a revetment is usually placed opposite the point bar dike field. These stabilization works are generally designed to confine and direct the low-stage flows so as to improve navigation conditions during the low-water season, and as a result are not generally in consonance with the high-flow condition of the river.

The Padma River in East Pakistan provides a classical example of the change in roughness (Manning's n) with increasing discharge in a natural, undeveloped river (Figure 2). Because the range of discharge in the Padma River approaches 85000 m^3/s, Figure 2 can be used to establish the general trend in roughness values to be expected in a large alluvial river system under natural conditions. The gradual decrease in roughness coefficient that can be anticipated as conditions change from low-stage to high-stage flow in an undeveloped bendway is sketched as the lower curve of Figure 1b.

In the developed bendway, however, roughness conditions are altered significantly. As the discharge increases from low flow to high flow, the main thread of the flow impinges on the contraction dikes at an angle that approaches 90 degrees (Figure 1a). The "shock" losses that result from the turbulence generated by this impact as the flow is diverted around or spills over the dikes radically increase the roughness characteristics of the reach. The concept of shock losses was advanced by Lacey (1930), in regard to the design of stable channels in alluvium to introduce the influence of "such discordant elements as alterations of section, enlargements, contractions, and turns and twists of a river". The concept of shock losses provides a convenient means of summarizing the impact of both natural irregularities and engineering structures on the resistance to flow in a given reach of river. As the number of structures such as contractions, dikes and revetments increases, the shock losses and the apparent roughness of a reach also increase. Thus, for increasing discharge in the developed reach, the trend in Manning's n can be sketched as the upper limb of Figure 1b. A partially developed reach with, perhaps, a single dike instead of a dike field would produce an intermediate roughness curve as shown.

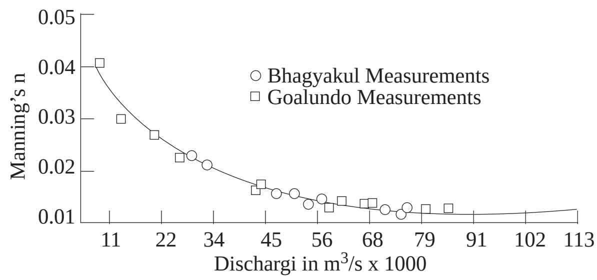

Figure 2. Decrease in Manning's n with discharge for the Padma River in East Pakistan

The response to this modification of the roughness characteristics of the reach as a result of constructing contraction works can be seen in Figure 1c. The qualitative stage-discharge relation for the natural river appears as the lower curve of Figure 1c. This increased roughness of the developed reach reduces velocity and increases the stage producing the modified stage-discharge relation approximated by the upper curve of Figure 1c. The intermediate stage-discharge curve reflects the diminished impact of partial development of the reach.

The critical problem here is that generally the data available for quantitative analysis cover only about one-third of the design discharge. The approach to the problem of developing an estimate of the design discharge flow line presented here suggests the use of the available field data as a base and a methodology for extrapolating this data to the design discharge range. The analysis is supplemented by the foregoing qualitative examination of the trend of parameters such as roughness and stage in the developed river.

Quantitative Analysis

An estimate of the design flood flowline can be obtained from the Manning equation in discharge form:

$$Q = \frac{1}{n} A R^{2/3} S^{1/2} \tag{1}$$

If section parameters for a given discharge are known and if the slope and roughness at the selected discharge can be estimated, the first task in the quantitative analysis then is to develop a working graph of slope versus discharge that can be extrapolated with reasonable reliability to the range of the design discharge. Next, the roughness versus discharge relations will be developed based on existing data. An estimate of roughness, will be calculated using methods like Richardson and Simons (1967), Shen (1962), Simons and Şentürk F. (1977) and Şentürk H. A. (1978) for the design discharge of the natural river. These relationships can then be combined to provide an estimate of the design discharge flowline at representative sections on the developed river.

Assuming that the slope-discharge relation in the range of existing data (up to 28000 m^3/s) is established from a plot of water surface elevation versus distance along the developed river between two representative sections about 300 km apart, namely Greenville, Mississippi (877 km) and Natchez, Mississippi (587 km). Discharge is included as a third variable, and by interpolation, flowlines in this reach for discharges of approximately 8500 m^3/s, 14000 m^3/s and 25500 m^3/s for an assumed uniform flow can be established. The slope of the water surface at these three discharges is calculated and plotted as in Figure 3 to establish the trend of the slope-discharge relation in the range of existing data. The trend line is essentially horizontal at the 7.08×10^{-5} level.

In extending the slope-discharge relation from the limits of existing data at approximately 28800 m^3/s to the range of the design flood at 85000 m^3/s qualitative reasoning based on an understanding of the physical processes at work in a large alluvial system must be employed. It is known that at higher discharges the main path of flow in large alluvial rivers tends to "short circuit" the meander loops of the system (Figure 1). Thus at higher discharges the length of the river along the flow-path through a given reach tends to decrease, which produces an increase in slope through the reach. The result of this process, if carried to the extreme, would be a straight river flowing at the valley slope. Hence, the valley slope represents an upper limit to the increase of slope with discharge. Here in the example, a value of 8.9×10^{-5} is taken as the valley slope for the reach to be studied.

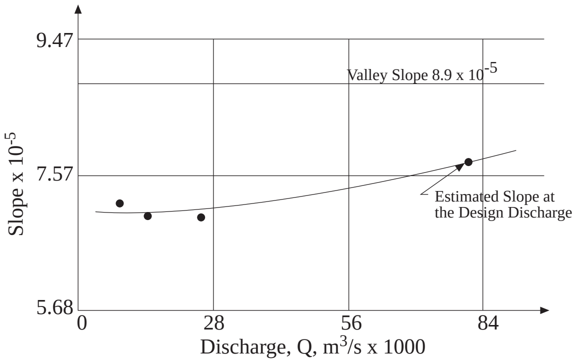

Figure 3. Water surface slope

To provide some quantitative guidance to the estimate of increasing slope with discharge, the potential within-bank shortening of the flow path in the study reach was measured. These measurements indicate a potential decrease of 5% percent in the flow path at the higher discharge, which in turn implies an increase of 5% in the uniform flow slope. It is reasonable to assume that a flood of the magnitude of the design flood (85000 m^3/s) will not remain within the channel. Overbank flow implies a further shortening of the flow-path and an increase in slope. For this estimate, the potential increase in slope at the design discharge can be taken as 10% of the slope value established by existing data in Figure 3. Accordingly, the slope at the design discharge is estimated at 7.78×10^{-5} ($1.10 \times 7.08 \times 10^{-5}$) which must be below the upper limit represented by the valley slope (Figure 3).

With the slope discharge relation established, the roughness (Manning's n) versus discharge plot can be developed. Roughness values (Figure 4) within the range of observed discharges (up to 28000 m^3/s) are calculated from the Manning equation (1) using measured geometric parameters (area and hydraulic radius) and slope values from Figure 3. The relatively large range of scatter in these roughness values does not indicate inaccuracy in either the measurements or calculations. This scatter represents the composite effect of a number of natural phenomena including the influence of sand waves moving through the system which on a large river can have amplitudes in the order of tens of feet, backwater effects, the influence of bends, water waves and the influence of man's activity. In extrapolating roughness values to the design discharge this natural scatter, in the order of 0.005 units, must be considered.

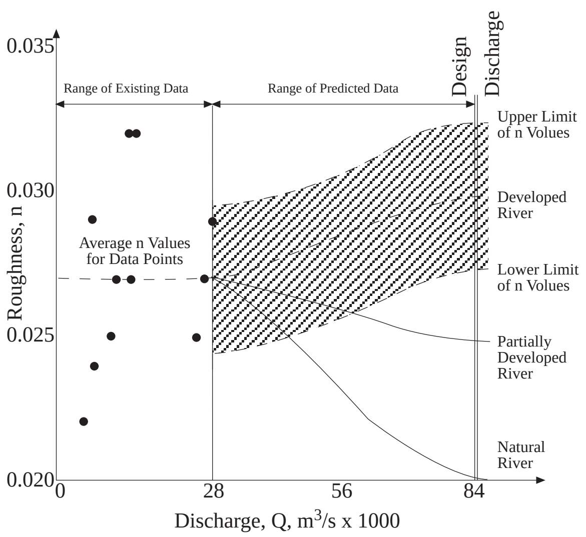

Figure 4. Roughness values at section 2

To assist in the extrapolation of roughness values to the design discharge a computer program (Şentürk, H. A., 1976; Simons, Şentürk, F., 1977; Shen, 1962;, Richardson and Simons, 1967) was developed utilizing several different methods to calculate river channel roughness (resistance to flow) in natural alluvial channels. The validity of these methods was checked against existing data for the Mississippi River, the Missouri River and Pakistan Chop Canal, and satisfactory agreement between observed and calculated resistance values was obtained.

These methods can be used to calculate roughness values (Manning's n) at the two chosen cross-sections of the 300 km study reach, section two being downstream of section one. The results of applying these methods namely, Shen's approach (Shen, 1962) (method 1), Simons-Richardson approach (Richardson and Simons, 1967) (method 2), and F. Şentürk's approach (Simons and Şentürk F., 1977) (method 3) to section 2 (Ajax Bar (780 km)) are shown in Figure 5 for a range of discharges to include the design discharge. These results can be assumed to represent roughness conditions on the natural river approach roughness values of 0.016 and 0.020. Here, the higher roughness value is selected to provide the most conservative estimate of natural river roughness at the design discharge.

To establish the variation of roughness with discharge in the present day, developed river, the calculated roughness values within the range of observed discharges from Figure 4 are replotted in Figure 5.

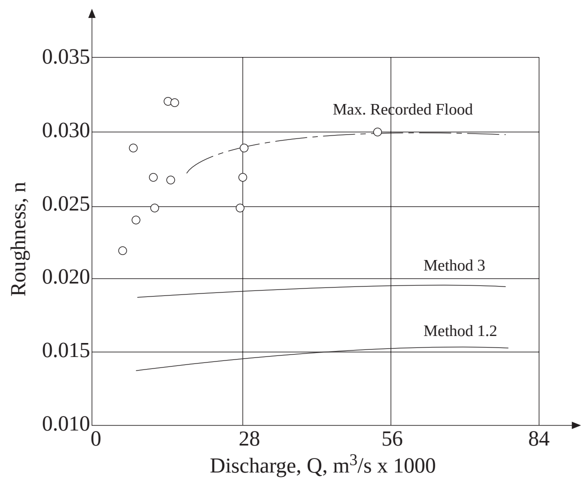

Figure 5. Actual and predicted roughness values at section 2

An estimation of roughness at the maximum recorded discharge, which is about 56000 m^3/s, is used to extend this data toward the design discharge of 84000 m^3/s. Noting the parallelism in the roughness versus discharge trend for methods 1, 2 and 3 (Figure 5), the roughness curve for the actual river can be extended with a similar trend to the design discharge. This extrapolation can still be made even if the value around 56000 m^3/s does not exist by making use of the parallel behavion in the roughness trend. This yields an estimated value of 0.030 for Manning's n under existing, developed conditions at section 2. For section 1 (Miller Bend (875 km)) the same approach also results in an estimate of 0.030 for roughness of the present day developed river.

Using a value for Manning's n of 0.020 on the natural river and a value of 0.030 on the developed river, Figure 4 is extrapolated to the range of the design discharge. The extension of the roughness discharge relation is guided by the qualitative analysis of Figure 1. An intermediate roughness value of 0.025 is selected to represent conditions on the partially developed river. Recalling the scatter of about 0.005 units that was noted in the calculated values of roughness (below 28000 m^3/s), an envelope of $\pm$ 0.0025 is drawn relative to the developed river curve. This yields for the developed river a range of roughness values between a low of 0.0275 and a high of 0.0325. The same range of predicted data results from the analysis of section 1 where the "average n value for data points" was also 0.027 (see Figure 4).

With the range of n values at the design discharge on the presentday, developed river estimated in Figure 4, the water surface elevation and thus the flowline at sections 1 and 2 can be estimated. The calculations are based on the Manning equation (1) and the use of Figure 3 for an estimate of the uniform flow slope at the design discharge. The geometric characteristics of the cross-sections at the design discharge were based on the maximum observed within-channel width at the maximum recorded discharge for a given section. Thus, the main flow for the design discharge is assumed to take place essentially within the banks.

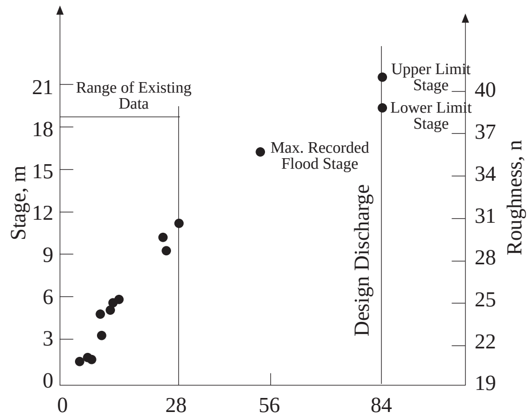

Figure 6. Stage-Discharge relation at section 2

This assumption is substantiated by observations made during floods of long duration. It was observed that on the average about 97% of the total flow passes through the main channel. For calculation of the water surface elevation at the design discharge, this same proportionality between within-channel and overbank flow was applied. In calculating the wetted perimeter at the design discharge, the water-water interface between within-channel and overbank flow was not considered.

With these assumptions, the predicted stages at the design discharge can be calculated. Stage-discharge data within the range of existing data are plotted in Figure 6 for section 2. The predicted lower limit stage and upper limit stage as calculated by the procedures outlined in this paper are also shown.

Besed on this stage data, water surface elevations and flowlines in the study reach can be calculated. These flowlines are shown in Figure 7. Flowlines at 8500 m^3/s, 14000 m^3/s and 25500 m^3/s were obtained from the working graph used to develop the slope-discharge relation in Figure 3. The upper limit and lower limit flowlines predicted by the procedures outlined in this paper are also shown.

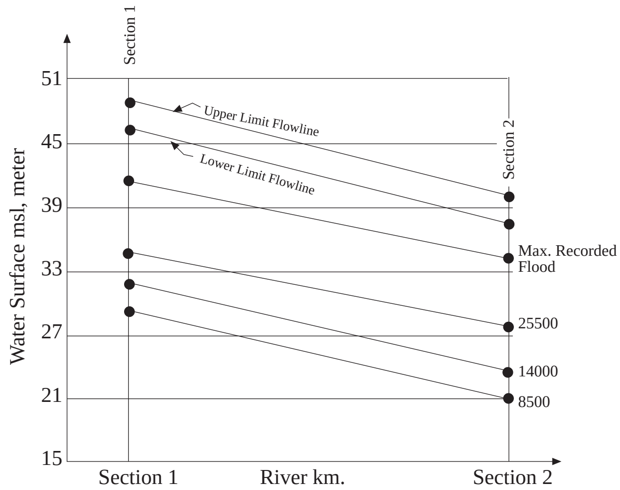

Figure 7. Flowlines in the study reach

The estimate of the design flood flowline developed in this paper represents an attempt to extend observed data on developed rivers to the range of the design flood. In moving from the range of known data into the range of predicted data several assumptions were required. Where possible, these assumptions were supported by quantitative data. Beyond this point it was necessary to rely on qualitative analysis based on field experience and observations of similar alluvial river systems. The refinement or modification of these assumptions can be expected to produce a more precise estimate of the design flood flowline. Additional data in the following specific areas would serve to refine the estimates of this approach.

1. Actual top width of the main channel at sections where calculations are to be made.
2. A better estimate of the proportion of flow that will pass through the main channel at the design discharge.
3. Actual sediment size distribution curves at the sections where calculations are to be made.
4. Actual specific gravity and concentration of sediment at sections where calculations are to be made.

It must also be recognized that the estimates of this approach are not supported by either a physical model study or a detailed mathematical model study for a specific problem. It is recommended, therefore, that the procedures outlined in this paper be used as an initial approach to roughness problems in developed or partially developed rivers.

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List of Symbols

A	cross-sectional flow area
n	manning's roughness coefficient
Q	discharge
R	hydraulic radius
S	slope of the water surface

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