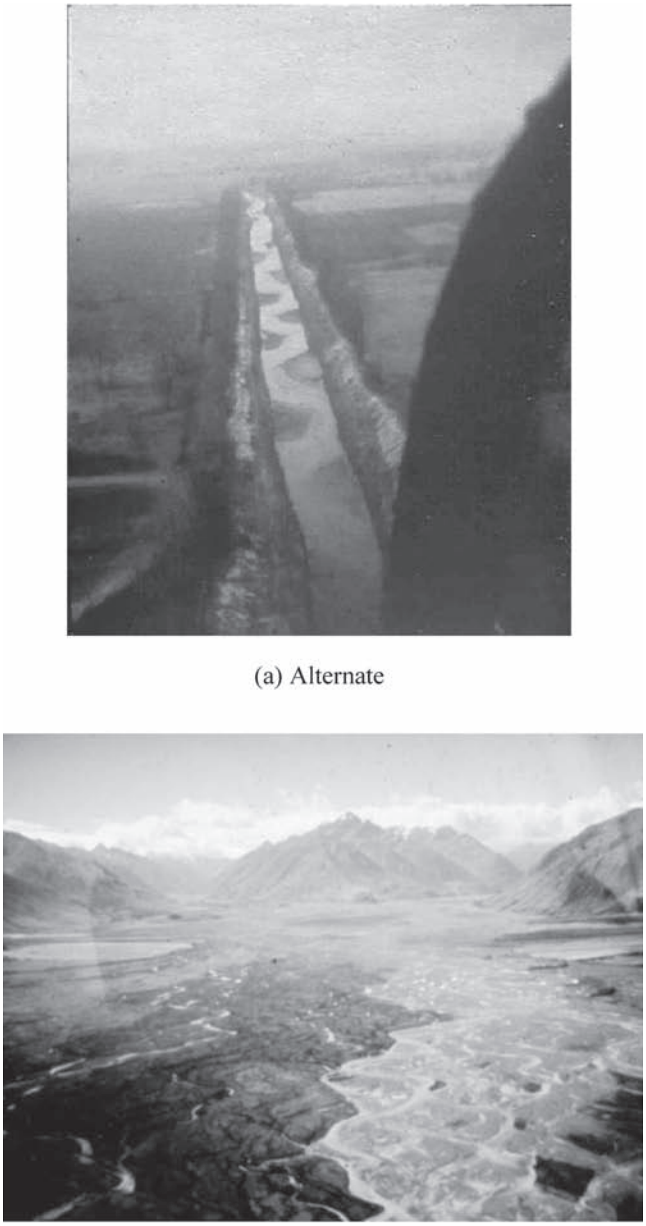

(a) Alternate

(b) Braided

Fig. 6-7. Typical bar patterns: (a) alternate, (b) braided.

Meander wavelength and amplitude are primarily dependent on water and sediment discharge, but are usually locally modified by spatial variation in the erodibility of the material in which the channel is formed. The effects of different bank materials are responsible for the irregularities found in the alignment of natural channels. In rare cases where the material forming the banks is practically homogeneous, meanders take a form that may be approximated by a sine-generated curve with a uniform meander wavelength. The *meander belt* is formed by and includes all the locations historically held by a stream due to meander development and migration. It should be noted that the width of the meander belt is usually greater than the meander amplitude and, in many cases, may include all of the active floodplain.

The *radius of curvature* (r_c) is the radius of the circle defining the centerline curvature of an individual bend, measured between the bend entrance and the bend exit (Fig. 6-8). The *arc angle* (θ) is the angle swept out by the radius of curvature. The ratio of radius of curvature to width (r_c/w) is a very useful parameter in the description and comparison of meander behavior and, in particular, bank erosion rates. The radius of curvature is dependent on the same factors as the meander wavelength and width. Meander bends generally develop a radius-of-curvature-to-width ratio (r_c/w) of 1.5 to 4.5, with the majority of bends falling in the range from 2 to 3. Nanson and Hickin (1986) examined the influence of r_c/w on bend migration rate and reported that maximum bank erosion rates occurred when the channel acquired an r_c/w between 2 and 3. This finding has been supported by many empirical studies, for example, Thorne (1991). Plots of erosion rate versus r_c/w do, however, display wide scatter and Biedenharn et al. (1989) showed that part of this scatter could be explained by variations in the erodibility of the outer bank material (Fig. 6-9).

River slope is one of the best indicators of the ability of a river to do morphological work. In general, rivers with steep slopes are much more active with respect to channel changes achieved through sediment movement, bed scour, bar building, and bank erosion. Slope can be defined in a number of

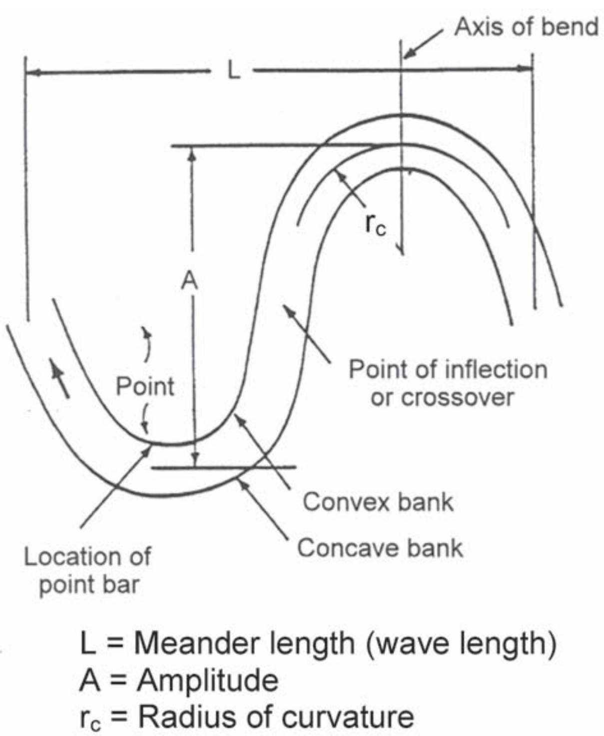

Fig. 6-8. Definition sketch for channel geometry (FISRWG 1998, with permission from the USDA).

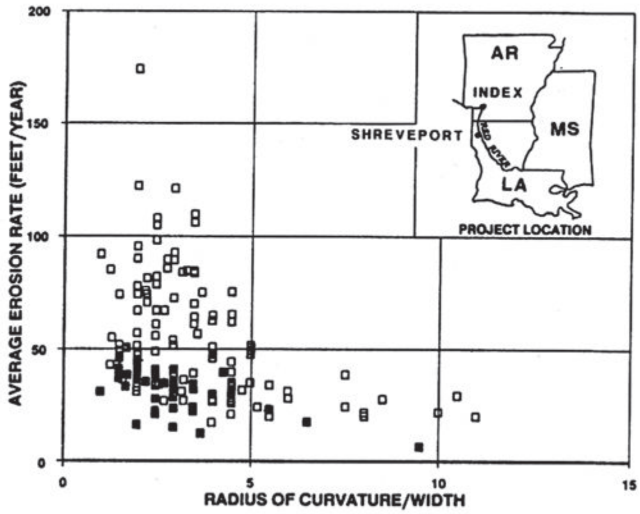

Fig. 6-9. Average annual erosion rate versus *r/w* for meander bends of the Red River. Open symbols represent free, alluvial bends and closed symbols, constrained bends (Biedenharn et al. 1989, with permission of ASCE Publications).

ways, however, leading to inconsistency in the way slope is used to represent the ability of a river to do morphological work. Ideally, energy slope should be used to calculate stream power, but the data required are seldom available. In gauged streams, water surface slope may be calculated using stage readings at consecutive gauging stations along the channel. However, many small streams are ungauged. In ungauged streams, thalweg slope is often used to calculate stream power. The thalweg profile not only provides a reasonable basis for calculation of stream power, but also may aid in locating bed controls due to geologic outcrops, other nonerodible materials, or inputs of relatively immobile sediments from steep tributaries. Repeat thalweg profiles are particularly useful in identifying bed-level adjustments through aggradation, degradation, local scour, and fill. When different slopes are used to calculate stream power, it must be kept in mind that the thalweg, water surface, and energy slopes are not necessarily equal.

6.3 SEDIMENT TRANSPORT

One aspect of river engineering that causes considerable confusion and misunderstanding is the terminology associated with sediment transport. In discussing the sediment transport, it is important to be familiar with the terminology adopted and the nature of the load being discussed. Over an extended period, a common terminology has emerged, and although it is not universally agreed upon or applied, it provides the basis for at least reducing inconsistency.

Total sediment load is the mass of granular sediment transported by a stream. It can be broken down by source, transport mechanism, or measurement status (Table 6-1). *Bed load* is a component of total sediment load made up of particles moving in continuous or frequent contact with the bed. Transport occurs at or near the bed, with the submerged weight of particles supported by solid-solid contact with the bed. Bed load movement takes place by processes of rolling, sliding, and saltation. *Suspended load* is a component of the total sediment load made up of sediment particles moving in continuous or semicontinuous suspension within the water column. Transport occurs above the bed, with the submerged weight of particles supported by anisotropic turbulence within the body of the flowing water. *Bed-material load* is the portion of total sediment load composed of grain sizes that are found in appreciable quantities in the streambed. The bed-material load is the bed load plus the coarser portion of the suspended load, that is, particles of a size that are found in significant quantity in the bed. *Wash load* is the portion of the total sediment load composed of grain sizes finer than those found in appreciable quantities in the streambed. *Measured load* is the portion of total sediment load that is sampled by conventional suspended load samplers. The sediment sampled in

Table 6-1 Classification of the Sediment Load

<table>
<tr><th>Measurement method</th><th>Transport mechanism</th><th>Sediment source</th></tr>
<tr><td>Unmeasured load</td><td>Bed load</td><td rowspan="2">Material load</td></tr>
<tr><td rowspan="2">Measured load</td><td rowspan="2">Suspended load</td></tr>
<tr><td>Wash load</td></tr>
</table>

deriving the measured load includes a large proportion of the suspended load, but excludes that portion of the suspended load moving very near the bed (that is, below the sample nozzle) and all of the bed load. *Unmeasured load* is that portion of the total sediment load that passes beneath the nozzle of a conventional suspended load sampler, moving in near-bed suspension and as bed load.

6.4 CHANNEL-FORMING DISCHARGE

Morphological studies have revealed that channel form depends on a delicate balance between the flows of water and sediment that shape the channel, the processes by which channel form is changed, and the ability of the boundary materials to resist change. Variability of water and sediment discharges is a characteristic of the watershed and, over a sufficiently long period, the morphology of the channel will adjust to accommodate the range of flow events responsible for regulating the balance between the erosive and resistive forces that mold the channel. Consequently, the shape and dimensions of an alluvial river channel are adjusted to and reflect the wide range of flows that entrain, transport, and deposit boundary sediments (Lane 1955). The concept that there is a single discharge that, if it prevailed all the time, would produce the same width, depth, slope, hydraulic roughness, and planform as those produced by the actual range of discharges is attractive, but viewed in this context it is clearly a gross simplification. The single discharge best able to represent the actual spectrum of sediment-transport events to yield the same bank-full morphology as that shaped by the natural sequence of flows is referred to as the channel-forming flow or the dominant discharge. Dunne and Leopold (1978) define channel maintenance flow as the most effective discharge for moving sediment, forming or removing bars, forming or changing bends and meanders, and generally doing work that results in the average morphological characteristics of channels. Their definition of channel maintenance flow is very similar to the concept of channel-forming discharge.

In a regulated canal system, the dimensions of the channel can appropriately be based on a single design discharge. Empirical analysis of the relationship between that discharge and the dimensions for a stable, unlined canal formed in alluvial materials produced the regime theory. Early work on regime theory stems from design of straight canals in the Indian subcontinent (Inglis 1941; 1947; 1949), and North America (Blench 1952; 1957). Later, flume experiments extended the regime approach to channels with meandering planforms (Ackers and Charlton 1970a; 1970b). However, for widely varying flows emanating from a natural watershed, the problem of identifying the single channel-forming discharge is both challenging and critical.

Soar (2000) recently reviewed the huge literature pertaining to the concept of channel-forming flow. This concept is closely related to the theory of dynamic equilibrium, which is characterized by fluctuations of channel form around an average condition that persists through time. In perennial rivers, recovery of equilibrium following a major event occurs relatively quickly, partly because rapid vegetation growth encourages sedimentation (Hack and Goodlett 1960; Gupta and Fox 1974). Hence, the long-term time-averaged condition is a valid representation of the channel form. Recovery in the ephemeral channels of semiarid regions tends to take longer, reflecting the influence of relatively wet and dry periods on vegetation growth (Schumm and Lichty 1965; Burkham 1972). In arid areas, infrequent floods impart long-lasting imprints on channels because more frequent flows do not have the power to restore a regime condition (Schick 1974). It has been concluded that the channel-forming flow concept may be inapplicable to ephemeral rivers that exhibit highly variable flow regimes, because there may not be a single discharge that can explain channel form (Stevens et al. 1975; Baker 1977). This is because channel morphology is likely to be perpetually in disequilibrium with the prevailing flows rather than fluctuating around an average state.

Channel-forming flow or dominant discharge is actually a geomorphological concept and not strictly a measurable parameter. However, a number of discharges that may be taken to represent the channel-forming flow can be defined and calculated using prescribed methodologies. The first approach is to identify a candidate flow based on channel morphology, such as the bank-full discharge. A second approach is to select a discharge based on a specified recurrence interval discharge, typically between the 1- and 3-year events in the annual maximum series. The third approach is analytical and involves calculating the effective discharge.

6.4.1 Bank-Full Discharge

Based on both theoretical and empirical arguments, bank-full discharge is generally recognized as being the moderate flow that best fits Wolman and Miller's (1960) dominant discharge concept for rivers in dynamic equilibrium. Leopold et al. (1964) proposed that the bank-full discharge was responsible for channel maintenance and form, and therefore that it was equivalent to the channel-forming discharge. Dury (1961) also suggested that the channel-forming discharge is approximately equal to the bank-full discharge and Dunne and Leopold (1978) concluded that their maintenance discharge corresponded to the bank-full stage. Field identification of bank-full discharge is, however, problematic (Williams 1978). It is usually based on identification of the minimum width-to-depth ratio (Wolman 1955; Pickup and Warner 1976), together with the recognition of some discontinuity in the nature of the channel, such as a change in sedimentary or vegetative characteristics. Nixon (1959) defined the bank-full state as the highest flood of a river that can be contained within its channel without spilling water on the river floodplain. Wolman and Leopold (1957) defined the bank-full stage as the elevation of the active floodplain. Woodyer (1968) suggested that bank-full discharge corresponds to the elevation of the middle bench of rivers having several overflow surfaces. Schumm (1960) defined bank-full stage as the height of the lower limit of perennial vegetation, primarily trees. Similarly, Leopold (1994) states that bank-full stage is indicated by a change in vegetation, such as herbs, grasses, and shrubs. Finally, the bank-full stage is also defined as the average elevation of the highest surface of the channel bars (Wolman and Leopold 1957). Harrelson et al. (1994) provide explanations of field methods for determining bank-full discharge using vegetation, gradation of bank materials, and elevation of sedimentary features. Although several criteria have been identified to assist in field identification of bank-full stage, ranging from vegetation boundaries to morphological breaks in bank profiles, considerable experience is required to apply these in practice, especially on rivers that have in the past undergone aggradation or degradation.

Table 6-2 Recommended Frequencies for Bank-Full Discharge (after Soar 2000)

Discharge frequency	Recommended by
1 to 5 years	Wolman and Leopold (1957)
1.5 years	Leopold et al. (1964); Hey (1975); Leopold (1994)
1.58 years	Dury (1973, 1976); Riley (1976)
1.02 to 2.69 years	Woodyer (1968)
1.01 to 32 years	Williams (1978)
1.18 to 3.26 years	Andrews (1980)
1 to 10 years, 2 years	USACE (1994)
2 years	Bray (1973, 1982)

6.4.2 Specified Recurrence Interval Discharge

Problems and subjectivity in the field identification of bank-full elevation and discharge make it attractive to use an objectively defined discharge such as a specific recurrence interval flow. This recurrence interval flow can, in turn, be related to the bank-full elevation (Table 6-2). Wolman and Leopold (1957) suggested that the bank-full frequency has a recurrence interval of 1 to 2 years. The most often quoted recurrence interval is 1.5 years. Dury (1973) concluded that the bank-full discharge is approximately 97% of the 1.58-year discharge, or the most probable annual flood. Hey (1975) showed that for three British gravel-bed rivers, the 1.5-year flow in an annual maximum series passed through the scatter of bank-full discharges measured along the course of the rivers. Richards (1982) suggests that, in a partial duration series, bank-full discharge equals the most probable annual flood, which has a 1-year return period. Leopold (1994) concludes that most investigations have found that the recurrence interval for bank-full discharge ranges from 1.0 to 2.5 years. However, there are many instances where the bank-full discharge does not fall within this range. For example, Williams (1978) showed that for 35 floodplains in the United States the recurrence interval of bank-full discharge varied between 1.01 and 32 years, and found that only about one-third of those streams had a bank-full discharge with a recurrence interval between 1 and 5 years. In a similar study, Pickup and Warner (1976) determined that bank-full recurrence intervals ranged from 4 to 10 years on the annual series.

If a specified recurrence interval flow is used to estimate the channel-forming discharge, a range of 1 to 3 years should be used. However, because of the uncertainties discussed above, it is recommended that discharges in this range be compared to the bank-full stage in the field to verify that they do have morphological significance.

6.4.3 Effective Discharge

The effective discharge is defined as the increment of discharge that transports the largest fraction of the annual sediment load over a period of years (Andrews 1980). The effective discharge incorporates the principle prescribed by Wolman and Miller (1960) that the channel-forming or dominant discharge is a function of both the magnitude of sediment-transporting events and their frequency of occurrence. An advantage of using the effective discharge is that it is a calculated value that integrates the discharge and sediment-transport regimes of the stream.

Equivalence between bank-full and effective discharges for natural alluvial channels that are in regime has been demonstrated for a range of river types (sand, gravel, cobble, and boulder-bed rivers) and in different hydrological environments, if the flow regime is adequately defined and the appropriate component of the sediment load is correctly identified (Andrews 1980; Carling 1988; Hey 1997). However, Benson and Thomas (1966), Pickup and Warner (1976), Webb and Walling (1982), Nolan et al. (1987), and Lyons et al. (1992) report that the effective and bank-full discharges are not always equivalent. This suggests that the effective discharge may not always be a direct surrogate for the channel-forming flow or the bank-full discharge.

Although the effective discharge is straightforward conceptually, and has been used for many years, many engineers have expressed concerns that the effective discharge calculations do not yield reasonable results in some instances. These problems may be attributable to data limitations, insufficient understanding of the morphology of the stream, or improper calculation procedure. To minimize these uncertainties a standardized procedure for the determination of the effective discharge has been developed and is outlined in the following paragraphs. This procedure is intended to help investigators avoid many of the potential problems that the authors have experienced in the calculation of effective discharge. Interested readers are referred to Biedenharn et al. (2000a) for a more detailed discussion of effective discharge calculation.

The method most commonly adopted for determining the effective discharge is to calculate the total load (tons) transported by the range of flows over a period of time by multiplying the frequency of occurrence of selected discharge classes (number of days) by the median magnitude of the sediment load (tons/day) transported by that class of flows. Although this approach has the merit of simplicity, the accuracy of the estimate of the effective discharge is clearly dependent on the calculation procedure adopted. The basic inputs required for calculation of effective discharge are (1) flow-duration data and (2) sediment transport as a function of stream discharge.

The first step in an effective discharge calculation is to group the discharge data into classes and determine the number of events occurring in each class during the period of record. This is usually accomplished from a flow-duration curve, which is a cumulative distribution function of measured discharges. A flow-duration curve shows the percentage of time a specific discharge is equaled or exceeded during the period of record, for which the curve was developed. From the flow-duration curve, the number of days that discharges within the specified class interval occurred can be calculated. The three critical components that must be considered in developing a flow-duration curve are the time base, the number of class intervals, and the period of record.

Conventionally, values of mean daily discharge are used to compute the flow-duration curve. Although this is convenient and uses readily available mean daily flow data that are published by the U.S. Geological Survey (USGS), it can, in some cases, introduce bias into the calculations. Mean daily values underestimate the influence of the high flows that occur within the averaging period and overestimate the significance of the low flows. On large streams such as the Mississippi River, the use of mean daily values is acceptable because differences between mean daily and daily peak discharges are negligible. However, on flashy streams, the time from the flood peak to base flow may be only a few hours, so mean daily flow cannot adequately describe the hydrograph. Missing flood peaks and associated high sediment loads can result in the effective discharge being underestimated. Rivers with a high flashiness index, defined as the ratio of the instantaneous peak flow to the associated daily mean flow, are most affected.

To avoid this problem it may be necessary to increase the temporal density from 24 h (mean daily) to 1 h, or even 15 min, especially on flashy streams. This will ensure that the hydrograph is adequately described, enabling a more representative effective discharge to be determined.

Class intervals should be arithmetic and must be of equal width. It has been demonstrated that the use of logarithmic or non-equal-width arithmetic classes introduces systematic bias into the calculation of effective discharge (Soar 2000; Soar and Thorne 2001). However, interested readers should review Holmquist-Johnson (2002) for guidance in calculating effective discharge for conditions under which equal-width class intervals are not usable. The selection of class interval may influence the calculated effective discharge. There are no definitive rules for selecting the most appropriate interval and number of classes. Yevjevich (1972) stated that the class interval should not be larger than $s/4$, where s is an estimate of the standard deviation of the sample. For hydrological applications he suggested that the number of classes should be between 10 and 25, depending on the sample size. Hey (1997) found that 25 classes with equal, arithmetic intervals produced a relatively continuous flow-frequency distribution and a smooth sediment-load histogram with a well-defined peak, indicating an effective discharge that corresponded exactly with bank-full flow. However, in the authors' experience, 25 classes may not always produce satisfactory results. It is recommended that in difficult cases the number of intervals be increased, but not to the extent that individual classes have zero events or only one event.

The period of record must be sufficiently long to include a wide range of morphologically significant flows, but not so long that changes in the climate, land use, or runoff characteristics of the watershed produce significant changes with time in the data. If the period of record is too short, there is a significant risk that the effective discharge will be inaccurate because of the occurrence of unrepresentative flows. A reasonable minimum period of record for an effective discharge calculation is about 10 years, with 20 years of record providing more certainty that the range of morphologically significant flows is fully represented in the data. Records longer than 30 years should be examined carefully for evidence of temporal changes in flow and/or sediment regimes.

The next step in the determination of the effective discharge is to develop a sediment-rating curve that relates the sediment transport and discharge. The sediment-rating curve can be developed from observed, measured sediment loads or using a computational procedure. Effective discharge is very sensitive to the slope of the sediment-discharge relationship.

The sediment load that is responsible for shaping the channel should be used in the calculation of the effective discharge. The suspended sediment load reported by USGS publications usually includes a portion of the bed-material load and most of the wash load. If measured suspended-sediment data are used for the effective-discharge calculation, then the fine sediment load, consisting of particles not found in appreciable quantities in the bed, should be omitted. If the bed load in the stream is only a small percentage of the total bed-material load, it may be acceptable to use only the measured suspended bed-material load in the effective discharge calculations. However, if the bed load is a significant portion of the load, it should be calculated using an appropriate sediment-transport function and then added to the suspended bed-material load to provide an estimate of the total bed-material load. If bed-load measurements are available, which seldom is the case, observed data may be used.

Once the fines have been removed from the data set, a sediment-rating curve is developed from the concentration data by plotting sediment load (concentration times discharge) against discharge, and then calculating a best-fit regression curve through the data, or, as required in some cases, multiple segments of best-fit regression.

The discharges used to generate the bed-material load histogram are the arithmetic mean discharges in each class of the flow-frequency distribution. The bed-material transport rate for each discharge class is found from the rating curve equation. This load is multiplied by the frequency of occurrence of that discharge class to find the total amount of bed material transported by that discharge class during the period of record. Care should be taken to ensure that the time units in the bed-material load rating equation are consistent with the frequency units for the distribution of flows. The results are plotted as a histogram. The bed-material load histogram should display a continuous distribution with a single mode (peak). If this is the case, the effective discharge corresponds to the mean discharge for the modal class (that is, the peak of the histogram). If the modal class cannot be identified readily, the peak of a smooth curve drawn through the tops of the histogram bars can be used to estimate the effective discharge by interpolation.

6.4.4 Overview

All three approaches to estimating the channel-forming flow or dominant discharge (bank-full estimate, discharge of a selected return period, and effective discharge) present challenges. The selection of the appropriate method will be based on data availability, the physical characteristics of the study stream, the level of study, and time and funding constraints. It is recommended that all three methods be used and the results cross-checked to reduce the uncertainty in the final estimate of the channel-forming flow. If the effective discharge method is used, then it is recommended that the standardized procedure presented here be followed.

6.5 RELATIONSHIPS IN RIVERS

Given the evident complexity of fluvial processes and their interactions with channel morphology, it is perhaps surprising that the characteristic forms adopted by alluvial rivers are limited in number and frequent in occurrence. For example, the planforms of meandering rivers display clear similarity in their proportions. Brice (1984) suggested that the similarity of meanders accounts for the fact that, if scale is ignored, all meandering rivers tend to look alike in plan view. It is the familiar and almost ubiquitous nature of the forms and features displayed by alluvial streams of different sizes, in widely varying landscapes, that makes these complex systems amenable to description by relatively simple empirical relationships. For example, relationships developed by Williams (1986) illustrate how Brice's recognition of the similarity of meanders may be expressed quantitatively through empirical relationships relating the geometric properties of channel meander to one another (Table 6-3).

Similarly, in regime theory the concept that the width, depth, slope, and planform of a river are adjusted to a channel-forming discharge is expressed numerically in simple power-law equations. The *Stream Corridor Restoration Manual* (FISRWG 1998) provides the selected summary of regime equations reproduced in Table 6-4.

Independent of regime theory, Leopold and Maddock (1953) compiled important statistical equations linking various channel dimensions to discharge using USGS gauging records. These equations, termed *hydraulic geometry relationships*, describe how width, depth, velocity, and other hydraulic characteristics vary both with stage at a station and with changing bank-full discharge downstream for some streams in the United States. The hydraulic geometry relationships are of the same general form as the regime equations of Kennedy (1895):

$$W = \mathrm{a}\,Q^{\mathrm{b}}$$
$$D = \mathrm{c}\,Q^{\mathrm{f}}$$
$$V = \mathrm{k}\,Q^{\mathrm{m}}$$

where W = channel width, Q = discharge, D = depth, and V = velocity. Later versions of these hydraulic geometry relationships (listed in Table 6-5) add the median bed sediment size (D_{50}) to improve the predictive power of the equations, and appear in the following format:

$$W = k_1\,Q^{k2}\,D_{50}{}^{k3}$$
$$D = k_4\,Q^{k5}\,D_{50}{}^{k6}$$
$$S = k_7\,Q^{k8}\,D_{50}{}^{k9}$$

The relationships presented here are only a small sample of those available in the literature. Regime relationships are empirical, which means that the relationships are derived from observed physical correlations and are strictly only applicable to the data sets from which they were derived. In this regard, Rinaldi and Johnson (1997) are correct to point out the inappropriateness of using simple regression equations in the design of meander restorations when fluvial processes and channel morphology in the project stream differ manifestly from conditions in the rivers used to develop the equations. In practice, hydraulic geometry and other empirical relationships may be widely and usefully applied, provided that conditions in the study watershed are similar to those in the watersheds for which the equations were developed. However, even under ideal conditions these equations remain incomplete representations of the factors that actually

Table 6-3 Derived Empirical Equations for River Meander and Channel Size (FISRWG 1998, with permission from USDA)

Equation number	Equation	Applicable range (meters)
	Interrelations between meander features	
2	$L_m = 1.25\ L_b$	$5.49 < L_b < 13{,}293$
3	$L_m = 1.63\ B$	$3.69 < B < 13{,}689$
4	$L_m = 4.53\ R_c$	$2.59 < R_c < 3{,}598$
5	$L_b = 0.8\ L_m$	$7.93 < L_m < 16{,}494$
6	$L_b = 1.29\ B$	$3.69 < B < 10{,}000$
7	$L_b = 3.77\ R_c$	$2.59 < R_c < 3{,}598$
8	$B = 0.61\ L_m$	$7.93 < L_m < 23{,}201$
9	$B = 0.78\ L_b$	$5.49 < L_b < 13{,}293$
10	$B = 2.88\ R_c$	$2.59 < R_c < 3{,}598$
11	$R_c = 0.22\ L_m$	$10.06 < L_m < 16{,}494$
12	$R_c = 0.26\ L_b$	$6.80 < L_b < 13{,}293$
13	$R_c = 0.35\ B$	$4.88 < B < 10{,}000$
	Relations of channel size to meander features	
14	$A = 0.0054\ L_m^{1.53}$	$10.06 < L_m < 23{,}201$
15	$A = 0.0085\ L_d^{1.53}$	$6.10 < L_d < 13{,}293$
16	$A = 0.0103\ B^{1.53}$	$4.88 < B < 11{,}616$
17	$A = 0.0669\ R_c^{1.53}$	$2.13 < R_c < 3{,}598$
18	$W = 0.0167\ L_m^{0.89}$	$7.93 < L_m < 23{,}201$
19	$W = 0.0228\ L_b^{0.89}$	$4.88 < L_b < 13{,}293$
20	$W = 0.0279\ B^{0.89}$	$3.05 < B < 13{,}689$
21	$W = 0.7108\ R_c^{0.89}$	$2.59 < R_c < 3{,}598$
22	$D = 0.0267 L_m^{0.66}$	$10.06 < L_m < 23{,}201$
23	$D = 0.0361 L_b^{0.66}$	$7.01 < L_b < 13{,}293$
24	$D = 0.0367 B^{0.66}$	$4.88 < B < 11{,}616$
25	$D = 0.0848\ R_c^{0.66}$	$2.59 < R_c < 3{,}598$
	Relations of meander features to channel size	
26	$L_m = 29.99\ A^{0.65}$	$0.04 < A < 20{,}914$
27	$L_b = 21.42\ A^{0.65}$	$0.04 < A < 20{,}914$
28	$B = 18.57\ A^{0.65}$	$0.04 < A < 20{,}914$
29	$R_c = 5.86\ A^{0.65}$	$0.04 < A < 20{,}914$
30	$L_m = 7.50\ W^{1.12}$	$1.49 < W < 3{,}963$
31	$L_b = 5.07\ W^{1.12}$	$1.49 < W < 2{,}134$
32	$B = 4.27\ W^{1.12}$	$1.49 < W < 3{,}963$
33	$R_c = 1.50\ W^{1.12}$	$1.49 < W < 2{,}134$
34	$L_m = 239.25\ D^{1.52}$	$0.03 < D < 18$
35	$L_b = 159.50\ D^{1.52}$	$0.03 < D < 18$
36	$B = 148.37\ D^{1.52}$	$0.03 < D < 18$
37	$R_c = 42.66\ D^{1.52}$	$0.03 < D < 18$
	Relations between channel width, channel depth, and channel sinuosity	
38	$W = 21.33\ D^{1.45}$	$0.03 < D < 18$
39	$D = 0.1492\ W^{0.89}$	$1.50 < W < 3{,}963$

(Continued)

Table 6-3 Derived Empirical Equations for River Meander and Channel Size (FISRWG 1998, with permission from USDA) *(Continued)*

Equation number	Equation	Applicable range (meters)
40	$W = 95.93\ D^{1.23}\ K^{-2.35}$	$0.03 < D < 17.99$ And $1.2 < K < 2.6$
41	$D = 0.08\ W^{0.05}\ K^{1.48}$	$1.49 < W < 3963$ And $1.2 < K < 2.6$

Note: A = bank-full cross-sectional area; B = meander belt width; D = bank-full mean depth; K = channel sinuosity; L_b = along-channel bend length; L_m = meander wavelength; R_c = loop radius of curvature; W = bank-full width. 1 ft = 0.3048 m.

Table 6-4 Limits of Data Sets used to Derive Regime Formulas (FISRWG 1998, with permission from the USDA)

Reference	Data source	Median bed-material size (mm)	Banks	Discharge (m^3/s)	Sediment concentration (ppm)	Slope	Bedforms
Lacey (1958)	Indian canals	0.1 to 0.4	Cohesive to slightly cohesive	2.37 to 237.3	< 500		
Blench (1969)	Indian canals	0.1 to 0.6	Cohesive	0.02 to 2,372.8	< 30[a]	Not specified	Ripples to dunes
Simons and Albertson (1963)	U.S. and Indian canals	0.318 to 0.465	Sand	2.37 to 9.5	< 500	0.000135 to 0.000388	Ripples to dunes
		0.06 to 0.46	Cohesive	0.12 to 2,095.2	< 500	0.000059 to 0.00034	Ripples to dunes
		Cohesive, 0.029 to 0.36	Cohesive	3.25 to 12.1	< 500	0.000063 to 0.000114	Plane
Nixon (1959)	U.K. rivers	Gravel		16.61 to 428.3	Not measured		
Kellerhals (1967)	U.S., Canadian, and Swiss rivers of low sinuosity, and lab	7 to 265	Noncohesive	0.03 to 1,675.2	Negligible	0.00017 to 0.0131	Plane
Bray (1982)	Sinuous Canadian rivers	1.9 to 145		4.60 to 3,284.0	"Mobile" bed	0.00022 to 0.015	
Parker (1982)	Single-channel Canadian rivers		Little cohesion	8.38 to 5,028.0			
Hey and Thorne (1986)	Meandering U.K. rivers	14 to 176		3.27 to 355.2	Qs computed to range up to 114	0.0011 to 0.021	

[a] Blench (1969) provides adjustment factors for sediment concentrations between 30 and 100 ppm. 1 ft^3/s = 0.0283 m^3/s.

influence channel form. For example, many popular hydraulic geometry equations express the stable width solely as a function of bank-full discharge. Intuitively, it would be expected that the width of a channel with sandy banks would be greater than that of an equivalent stream with clay banks. Indeed, Schumm's relationship between width-to-depth ratio (F) and the silt-clay weighted percentage in the channel perimeter (M) confirms this expectation empirically. If Schumm's relationship is valid, a width equation based only on discharge cannot fully account for observed width variability. Clearly, the generation of reliable results through application of simple and imperfect morphological relations

Table 6-5 Coefficients for Selected Hydraulic Geometry Formulas (FISRWG 1998, with permission from the USDA)

References	Data	Domain	k_1	k_2	k_3	k_4	k_5	k_6	k_7	k_8	k_9
Nixon (1969)	U.K. rivers	Gravel-bed rivers		0.5		0.545	0.33		$1.258n^{2b}$	−0.11	
Leopold et al. (1964)	Midwestern U.S.		1.65	0.5			0.4			−0.49	
	Ephemeral streams in semiarid U.S.			0.5			0.3			−0.95	
Kellerhals (1967)	Field (U.S., Canada, and Switzerland) and laboratory	Gravel-bed rivers with paved beds and small bed material concentration	1.8	0.5		0.33	0.4	−0.12[a]	0.00062	−0.4	0.92[a]
Schumm (1977)	U.S. (Great Plains) and Australia (Riverine Plains of New South Wales)	Sand-bed rivers	$37k_1^*$	0.38		$0.6k_4^*$	0.29	−0.12[a]	$0.01136k_7^*$	−0.32	
Bray (1982)	Canadian rivers	Gravel-bed rivers	3.1	0.53	−0.07	0.304	0.33	−0.03	0.00033	−0.33	0.59
Parker (1982)	Single-channel Alberta rivers	Gravel-bed rivers, banks with little cohesion	6.06	0.444	−0.11	0.161	0.401	−0.0025	0.00127	−0.394	0.985
Hey and Thorne (1986)	U.K. rivers	Gravel-bed rivers with									
		Grassy banks with no trees or shrubs	2.39	0.5		0.41	0.37	−0.11	$0.00296k_7^{**}$	−0.43	−0.09
		1-5% tree/shrub cover	1.84	0.5		0.41	0.37	−0.11	$0.00296k_7^{**}$	−0.43	−0.09
		Greater than 5-50% tree/shrub cover	1.51	0.5		0.41	0.37	−0.11	$0.00296k_7^{**}$	−0.43	−0.09
		Greater than 50% shrub cover or incised floodplain	1.29	0.5		0.41	0.37	−0.11	$0.00296k_7^{**}$	−0.43	−0.09

Notes: b_n = Manning n.
$k_1^* = M^{-0.39}$, where M is the percent of bank materials finer than 0.074 mm. The discharge used in this equation is mean annual rather than bank-full.
$k_4^* = M^{0.432}$, where M is the percent of bank materials finer than 0.074 mm. The discharge used in this equation is mean annual rather than bank-full.
$k_7^* = M^{-0.36}$, where M is the percent of bank materials finer than 0.074 mm. The discharge used in this equation is mean annual rather than bank-full.
$k_7^{**} = D_{54}^{0.84}\, Q_x^{\,0.10}$, where Q_x = bed material transport rate in kg s^{-1} at water discharge Q, and D_{54} refers to bed material and is in mm.
[a] Bed material size in Kellerhals' equation is D_{90}.

relies heavily on good insight and sound judgment on the part of the individual responsible for their application.

A misapplication of empirical relationships was lampooned by Mark Twain (1944) in *Life on the Mississippi.* Describing the Mississippi River cutoffs of which he had knowledge, he conceived a simple empirical relationship between river shortening and time, and then used it to predict the historical and future lengths of the Mississippi River, concluding that:

> Geology never had such a chance, nor such exact data to argue from! In the space of 176 years, the Lower Mississippi has shortened itself 242 miles. That is an average of a trifle over one mile and a third per year. Therefore, any calm person, who is not blind or idiotic, can see that in the Old Oölitic Silurian Period, just a million years ago next November, the Lower Mississippi River was upwards of 1,300,000 miles long, and stuck out over the Gulf of Mexico like a fishing rod. And by the same token, any person can see that 742 years from now the Lower Mississippi will be only a mile and three-quarters long, and Cairo and New Orleans will have joined their streets together, and be plodding comfortably along under a single mayor and a mutual board of aldermen. There is something fascinating about science. One gets such wholesale returns of conjecture out of such a trifling investment of fact.

The primary points of this passage are that, no matter what their basis in fact and observation, empirical relationships cannot be extrapolated either backward or forward in time, and engineers must avoid falling into the trap of designing a project based solely on "…wholesale returns of conjecture out of a trifling investment of fact."

6.6 CHANNEL STABILITY AND INSTABILITY

In designing river enhancement and channel rehabilitation projects the design engineer must recognize that rivers are dynamic systems, and must consider both the existing and possible future channel morphologies in the design. The problem is compounded when engineering interventions are planned, because the future morphology of the channel depends not only on the natural, or autonomous, evolution of the system, but also on channel response to construction, operation, and maintenance of the project. For this reason, it is important for the design engineer to acquire a broad understanding of the current stability status of the project reach and the extended channel network and to use this understanding to predict the type and extent of adjustments to the fluvial system likely to be triggered by the project. The capability to predict system response to the proposed works is vital to ensure that the selected enhancement or rehabilitation measures will work in harmony with both existing and future river conditions. The concept of channel stability status (which incorporates instability) builds on the basic geomorphic principles introduced previously and may be applied to the river at system and local scales.

6.6.1 System Stability

The geomorphic concept underpinning stability assessment in rivers is that over time the cross-sectional dimensions and longitudinal slope of the channel of an alluvial stream adjust so that the channel is able to convey the discharges of water and sediment supplied from upstream with no net change in hydraulic geometry or planform. On this basis, a stream may be classified as either *stable* or *unstable*, depending on whether the channel has adjusted or is still adjusting to the flow and sediment regimes. Mackin (1948) expressed the stability concept in his definition of the *graded stream*:

> A graded stream is one in which, over a period of years, slope is delicately adjusted to provide, with available discharge and with prevailing channel characteristics, just the velocity required for the transportation of the load supplied from the drainage basin. The graded stream is a system in equilibrium.

By definition, a graded stream does not have to have a channel that is static or fixed, and it may exhibit temporary morphological changes in response to the impacts of extreme events. Alluvial channel morphology is certain to be affected by major floods or protracted periods of low water, but provided that the time for moderate events to restore the graded morphology (termed the *recovery time*) is shorter than the return period for the extreme event (*recurrence interval*), the channel may be considered to be dynamically stable. The key attribute of a graded stream is that fluvial processes operating under formative flows tend to restore channel morphology to the graded condition following disturbance, rather than perpetuating or amplifying the changes imposed by the extreme event. A term commonly used for this type of stability is *dynamic equilibrium*.

The concept of dynamic equilibrium is inherent in a widely applied (and often misapplied), qualitative relationship for adjustment in alluvial streams proposed by Lane (1955):

$$QS \sim Q_s D_{50}$$

where Q = water discharge, S = slope, Q_s = bed-material load, and D_{50} = median size of the bed material. This relationship is commonly visualized as *Lane's balance* (Fig. 6-10). Mackin's explanation of how a graded stream responds to changes in the controlling variables is easily illustrated by Lane's balance, which shows how a change in any of the four driving variables will tend to produce a response in the others such that equilibrium is restored. When a channel is in dynamic equilibrium, it has adjusted these four variables so that the sediment transported into the reach is also transported out, without aggradation or degradation.

It should be noted that the map coordinates of a graded stream may change through time as the river reworks the