CHAPTER 7

Streambank Erosion and River Width Adjustment

James E. Pizzuto and the ASCE Task Committee on Hydraulics, Bank Mechanics, and Modeling of River Width Adjustment

7.1 INTRODUCTION

Many different methods are available to describe river channel morphology and morphological adjustments for river engineering purposes. Available approaches range from equations that predict the regime or graded morphology of equilibrium channels to mathematical models that simulate channel changes in time and space. Most mathematical models, however, neglect time-dependent channel-width adjustments and do not simulate processes of bank erosion or deposition. Although changes in channel depth caused by aggradation or degradation of the riverbed can be simulated, changes in width cannot. For prediction of the behavior of natural streams, this is a significant limitation, because channel morphology usually changes with time, and adjustment of both width and depth (in addition to changes in planform, roughness, and other variables) is the rule rather than the exception (Leopold et al. 1964; Simon and Thorne 1996). As a result, our ability to model and predict changes in river morphology and their engineering impacts is limited. This is unfortunate, because width adjustments can seriously impact floodplain dwellers, riparian ecosystems, and bridge crossings, bank protection works, and other riverside structures through bank erosion, bank accretion, or bankline abandonment of the active river channel.

In this chapter, methods for assessing processes of bank erosion and river width adjustment are reviewed. Most of this chapter was originally written by the ASCE Task Committee on Hydraulics, Bank Mechanics, and Modeling of River Width Adjustment (1998a; 1998b), which was chaired by Dr. Colin R. Thorne.

7.2 GEOMORPHIC CONTEXT OF RIVER WIDTH ADJUSTMENT

River width adjustments have varied causes and occur in different geomorphic settings (Figs. 7-1 and 7-2). Widening can occur by erosion of one or both banks without substantial incision (Fig. 7-1a) (Everitt 1968; Burkham 1972; Hereford 1984; Pizzuto 1994). Widening in sinuous channels may occur when outer bank retreat exceeds the rate of advance of the opposite bank (Fig. 7-1b) (Nanson and Hickin 1983; Pizzuto 1994). In braided rivers, bank erosion by flows deflected around growing braid bars is a primary cause of widening (Fig. 7-1c) (Leopold and Wolman 1957; Best and Bristow 1993; Thorne et al. 1993). In degrading streams, widening often follows incision of the channel when the increased height and steepness of the banks cause them to become unstable (Fig. 7-1d). Bank failures can cause very rapid widening under these circumstances (Thorne et al. 1981a,b; Little et al. 1982; Harvey and Watson 1986; Simon 1989). Widening in coarse-grained, aggrading channels can occur when flow acceleration due to a decreasing cross-sectional area, coupled with current deflection around growing bars, generates bank erosion (Fig. 7-1e) (Simon and Thorne 1996).

Processes of channel narrowing are equally diverse (Fig. 7-2). Rivers may narrow through the formation of in-channel berms, or benches at the margins (Fig. 7-2a) (Pizzuto 1994; Moody et al. 1999) (Fig. 7-3). The growth of berms or benches often occurs when bed levels stabilize following a period of degradation and can eventually lead to the creation of a new, low-elevation floodplain and establishment of a narrower, quasi-equilibrium channel (Woodyer 1975; Harvey and Watson 1986; Simon 1989). Encroachment of riparian vegetation into the channel often contributes to the growth, stability and, in some cases, to the initiation of berm or bench features (Hadley 1961; Schumm and Lichty 1963; Harvey and Watson 1986; Simon 1989). Narrowing in sinuous channels occurs when the rate of alternate or point bar growth exceeds the rate of retreat of the cut bank (Fig. 7-2b) (Nanson and Hickin 1983; Pizzuto 1994). In braided channels, narrowing may result when a marginal anabranch is abandoned (Fig. 7-2c) (Schumm and Lichty 1963). Sediment is deposited in the abandoned channel until it merges into the

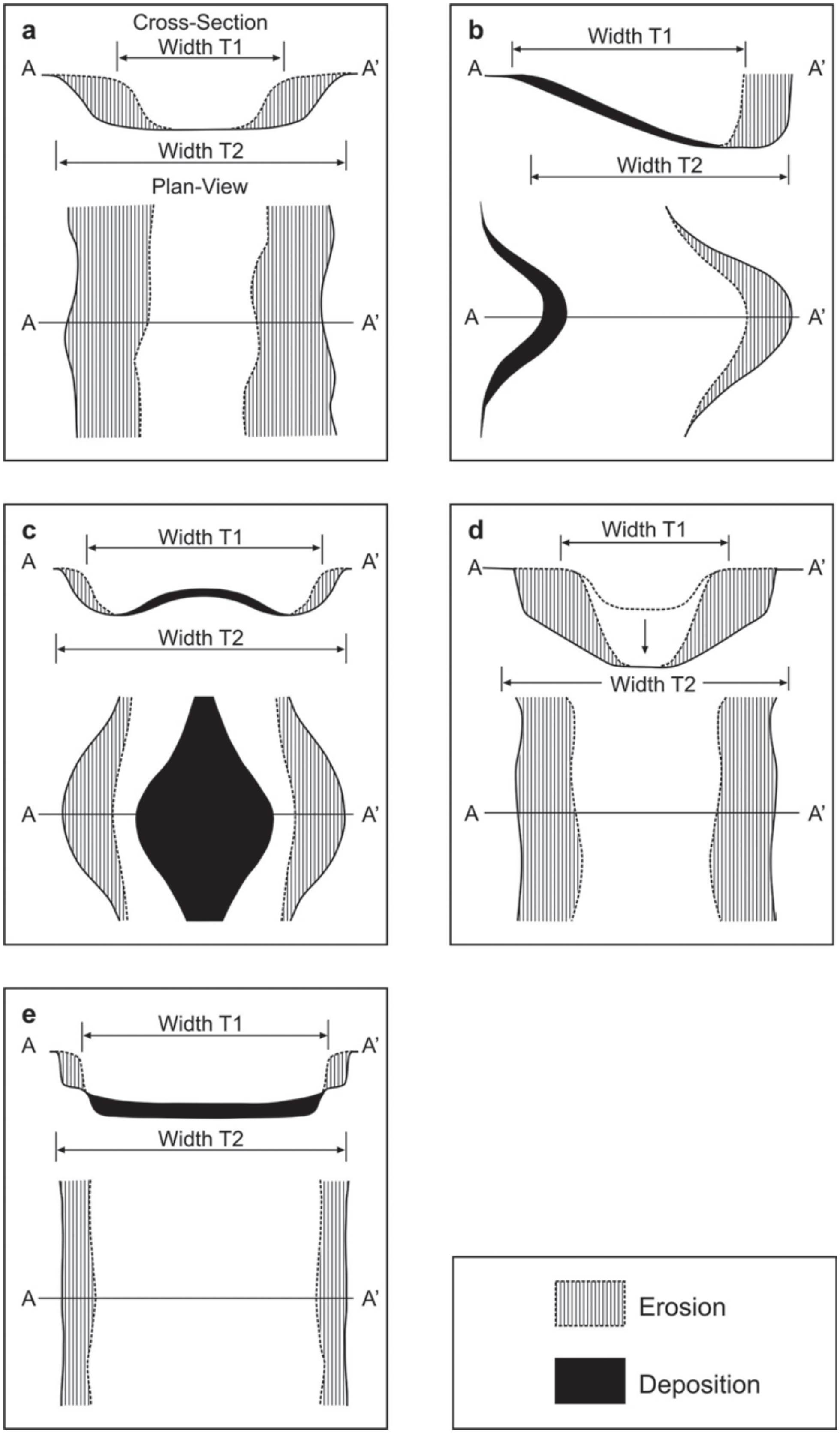

Fig. 7-1. Geomorphology of channel widening: (a) channel enlargement by bank erosion without incision; (b) erosion of outer bank in sinuous channel at faster rate than accretion on bar opposite; (c) deflection of flows by growing braid bar; (d) bank failure and retreat due to mass instability following channel incision; (e) bank erosion due to flow acceleration and deflection in aggrading channel (ASCE Task Committee on Hydraulics, Bank Mechanisms and Modeling of River Width Adjustment 1998a, with permission from ASCE).

floodplain. Also, braid bars or islands may become attached to the floodplain, especially following a reduction in discharge (Fig. 7-2d). Island tops are already at about floodplain elevation and attached bars are built up to floodplain elevation by sediment deposition on the surface of the bar, often in association with the establishment of riparian vegetation. Attached islands and bars may, in time, become part of the floodplain bordering a much narrower, often single-threaded channel (Williams 1978; Nadler and Schumm 1981).

If the flow regime and sediment supply are quasi-steady over periods of decades or centuries, then a river may adjust its morphology to create a metastable equilibrium form (Schumm and Lichty 1965). Such rivers are described as being graded or in regime (Mackin 1948; Leopold and Maddock 1953; Wolman 1955; Leopold et al. 1964; Ackers 1992). Although the width of an equilibrium stream may change due to the impact of a large flood or some other extreme event, the stable width is often eventually recovered following such perturbations (Costa 1974; Gupta and Fox 1974; Wolman and Gerson 1978). Unfortunately, predicting the time-averaged morphology of equilibrium channels remains a difficult problem, despite years of effort

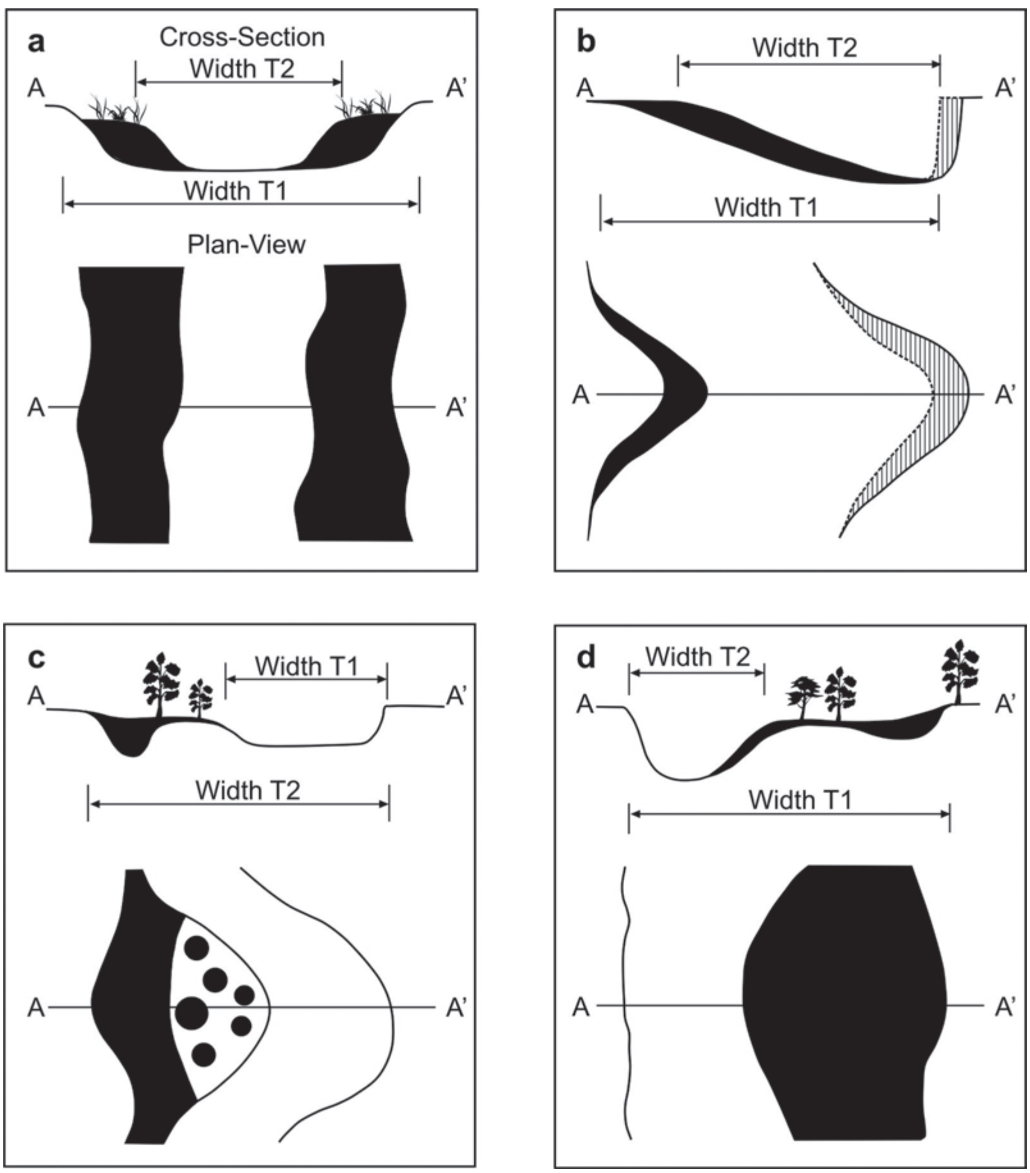

Fig. 7-2. Geomorphology of channel narrowing: (a) channel reduction by berm or bench formation; (b) accretion on advancing bar at faster rate than erosion of bank opposite; (c) abandonment of marginal anabranch in braided channel; (d) closure of marginal channel when braid bars or island becomes attached to floodplain (ASCE Task Committee on Hydraulics, Bank Mechanisms and Modeling of River Width Adjustment 1998a, with permission from ASCE).

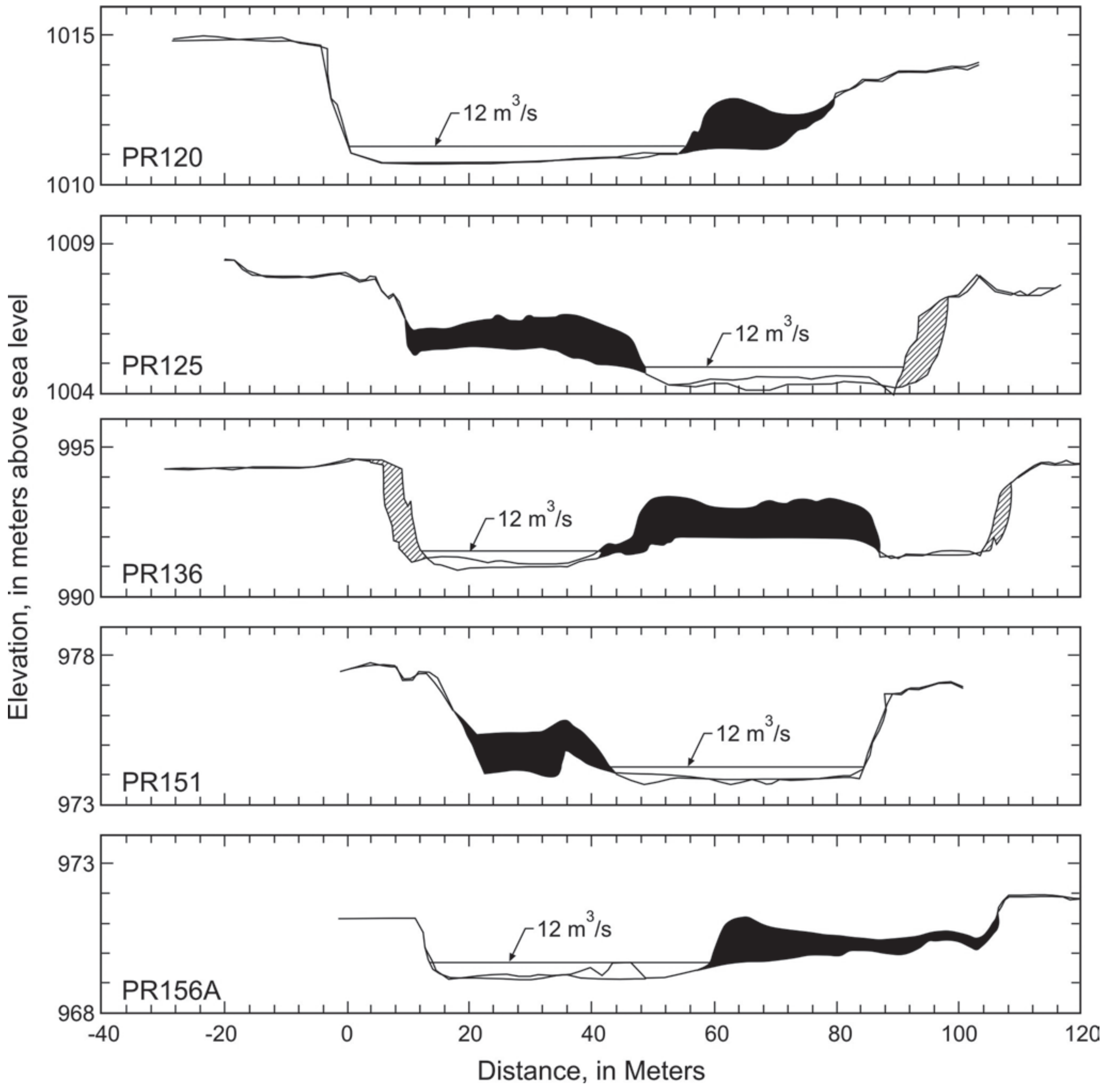

Fig. 7-3. Five examples of channel narrowing by floodplain formation along Powder River in southeastern Montana based on surveyed cross sections in 1978 and 1996 (solid lines). Surveys in 1978 were done after a 25–50-year flood that widened the channel. The hachured areas represent net erosion, and the solid black areas represent net deposition (from Moody et al. 1999).

(Ackers 1992; White et al. 1982; Ferguson 1986; Bettess and White 1987).

Many rivers, however, cannot be considered to have equilibrium channels, even as an engineering approximation. These rivers display significant morphological changes, including width adjustments, when viewed over decades or centuries. For example, some rivers in arid and semiarid regions of the American West change their morphologies drastically as the volume of annual precipitation, frequency of flood events, and other factors vary stochastically (Schumm and Lichty 1963; Everitt 1968; Burkham 1972; Osterkamp and Costa 1987). Because these streams vary so dramatically, they cannot be considered as graded or regime channels (Stevens et al. 1975) but are perpetually enlarging rapidly in response to a period of relatively high discharges, or contracting during periods of less than average runoff (Schumm and Lichty 1963; Stevens et al. 1975; Pizzuto 1994).

Other nonequilibrium rivers may be actively adjusting to changes in flow regime and sediment supply (Andrews 1986; Madej 1977; Smith and Smith 1984), changing valley slope (Patton and Schumm 1975), succession of riparian vegetation (Hadley 1961; Graf 1978), climate change (Schumm 1968; Knox 1983; Hereford 1984), watershed land-use change (Hammer 1972), neotectonic valley floor tilting (Burnet and Schumm 1983; Schumm and Winkley 1994), or sea-level rise (Brammer et al. 1993). The resulting width adjustments can occur at various rates and in different temporal sequences. For example, Hammer (1972) suggested that rivers of southeastern Pennsylvania adjust to the impacts of urbanization in less than 5 years, but Andrews (1986), Jacobson and Coleman (1986), and other researchers have documented disruptions in river morphology that persisted for more than a century.

Width adjustments not only encompass a variety of time scales; they are also accomplished by a wide range of fluvial processes and geotechnical mechanisms associated with varying discharge, climatic, and environmental conditions. Bank erosion processes provide a useful example. Wolman (1959) noted that significant bank erosion on Watts Branch in the Maryland Piedmont occurred more than 10 times per year during relatively small but frequent flow events. However, scientists working elsewhere report that significant bank erosion has been caused mostly by large floods with recurrence intervals of decades or centuries (Williams and Guy 1973; Costa 1974; Gupta and Fox 1974; Gardner 1977; Osterkamp and Costa 1987). In other cases, bank retreat has been found to be almost entirely unrelated to flow stage and intensity but correlated with precipitation events and ground-water levels that generate erosion through sapping or piping (Brunsden and Kesel 1973; Ullrich et al. 1986; Hagerty 1991). Thus, identifying the dominant erosion processes and failure mechanisms and selecting the appropriate discharge or climate events to be included in either conceptual or mathematical models of width adjustment remain very difficult tasks.

These examples indicate that channel changes involving width adjustment occur in a wide variety of geomorphic contexts, that width adjustment will usually be accompanied by changes in other morphological parameters such as channel depth, roughness, bed-material composition, riparian vegetation, energy slope, and channel planform, and that the processes responsible for width adjustments are diverse. Furthermore, adjustment processes display a variety of spatial patterns and operate over a wide variety of time scales. Because of this diversity, it is unlikely that a single method can be developed to predict the trends and rates of width adjustment for all rivers. Therefore, engineers must establish the morphological context of width adjustment and identify the major processes and mechanisms involved before selecting appropriate methods for analysis, modeling, and solution of width adjustment problems. This is best achieved through systematic field observation and monitoring.

7.3 FACTORS INFLUENCING BANK EROSION AND WIDTH ADJUSTMENT

7.3.1 Cause and Effect: The Influence of Scale

Causes of bank erosion and width adjustments can be viewed at several different spatial scales. Bank erosion, for example, may occur because high discharges cause increased shear forces on the banks. However, the high discharges themselves may be a result of changes in land use or climatic changes that are controlled by processes external to a particular river reach. Channel enlargement caused by urbanization provides a useful example (Hammer 1972). Impervious surfaces throughout the watershed generate increased runoff (Leopold 1968), which in turn causes bank erosion and increases in channel width and cross-sectional area.

It is important for engineers to understand both local and larger-scale causes of bank erosion and width adjustment. In some cases, bank protection or other small-scale engineering structures may provide the best solution to a bank erosion problem. In other cases, however, trying to mitigate erosion at a particular reach may be futile and wasteful because the problem is ultimately caused by land use practices throughout a watershed.

The following discussion of factors influencing bank erosion and width adjustment focuses on local processes that may affect individual river cross sections. However, the broader context of the entire watershed should always be considered in trying to understand and solve bank erosion and width adjustment problems.

7.3.2 Fluvial Hydraulics

7.3.2.1 Introduction The flow of water and sediment in rivers is described by Newton's laws, which are straightforward until turbulence is encountered. The simplified, one-dimensional (1D) St. Venant equation is commonly applied in river engineering because the flow can be considered to occur predominantly in the downstream direction. An appropriate rigid boundary resistance law is then usually adopted to relate the conveyance capacity to the geometry (see, for example, Keulegan 1938; ASCE Task Committee on Friction Factors in Open Channels 1963; Chow 1959; Cunge et al. 1980; and Yen 1993). For alluvial channels the resistance law must also take into account the additional energy losses arising from bed forms and sediment transport (Engelund 1966; Alam and Kennedy 1969; Garde and Ranga-Raju 1977; White et al. 1982; van Rijn 1984). For meandering channels, additional resistance terms are required for form drag due to channel curvature (Nelson and Smith 1989a).

A 1D representation is simple but is inadequate to define the processes and the mechanics of river width adjustment.

A three-dimensional (3D) formulation is better, but 3D models are so complex to solve that they are of little value except in the most well-funded of projects. A two-dimensional (2D) representation (depth integration of 3D equations) can be solved more readily, but it is still limited because it is not strictly applicable in the near-bank zone and does not give any motion in the vertical direction. Vertical motion is particularly important at river bends, where most bank erosion and bar deposition occur. However, a 2D formulation is still frequently used because it yields useful information about the lateral variation of most of the important hydraulic parameters that impact bank erosion and accretion. A detailed review of aspects of channel hydraulics that directly influence width adjustment is given here. A broader river of river and floodplain hydraulics is given elsewhere by Knight and Shiono (1995).

7.3.3.2 Cross-Sectional Shape The shape of a river cross section influences the isovel, secondary flow, and boundary shear stress distributions in a number of ways. A typical example (Fig. 7-4) is a rectangular cross section in a straight river with vertical banks. The data are from a rectangular duct experiment with an equivalent open-channel width/depth ratio of 20. Even in this relatively wide case the isovels and boundary shear stress distribution indicate the presence of secondary flow cells and 3D effects in the near-bank zone, which in this case is 15% of the channel width. Fig. 7-5 shows isovels and boundary shear stresses for flow in a narrow trapezoidal channel with an aspect ratio of 1.5. In this case the narrowness

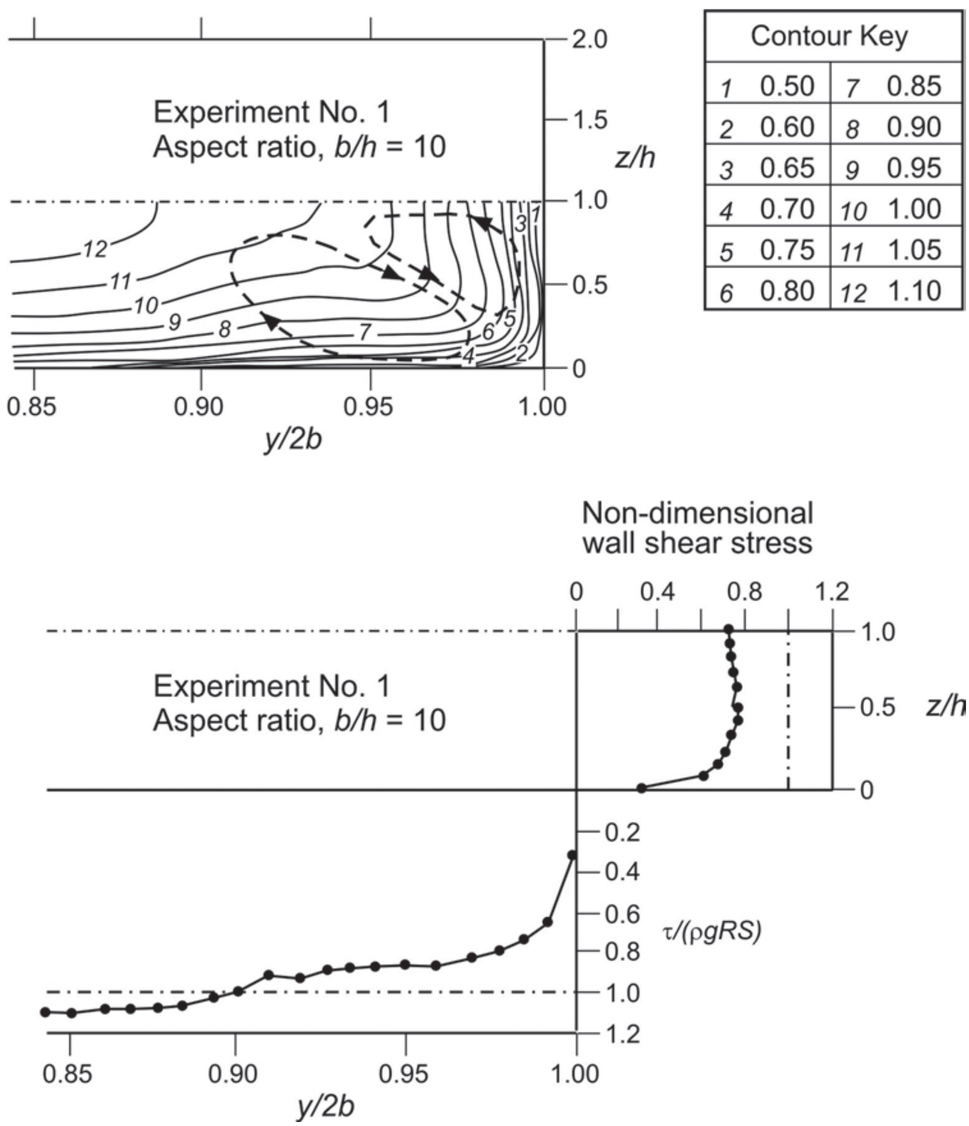

Fig. 7-4. Typical influence of vertical riverbank on velocity and boundary shear stress in wide rectangular channel (Rhodes and Knight 1994, with permission from ASCE).

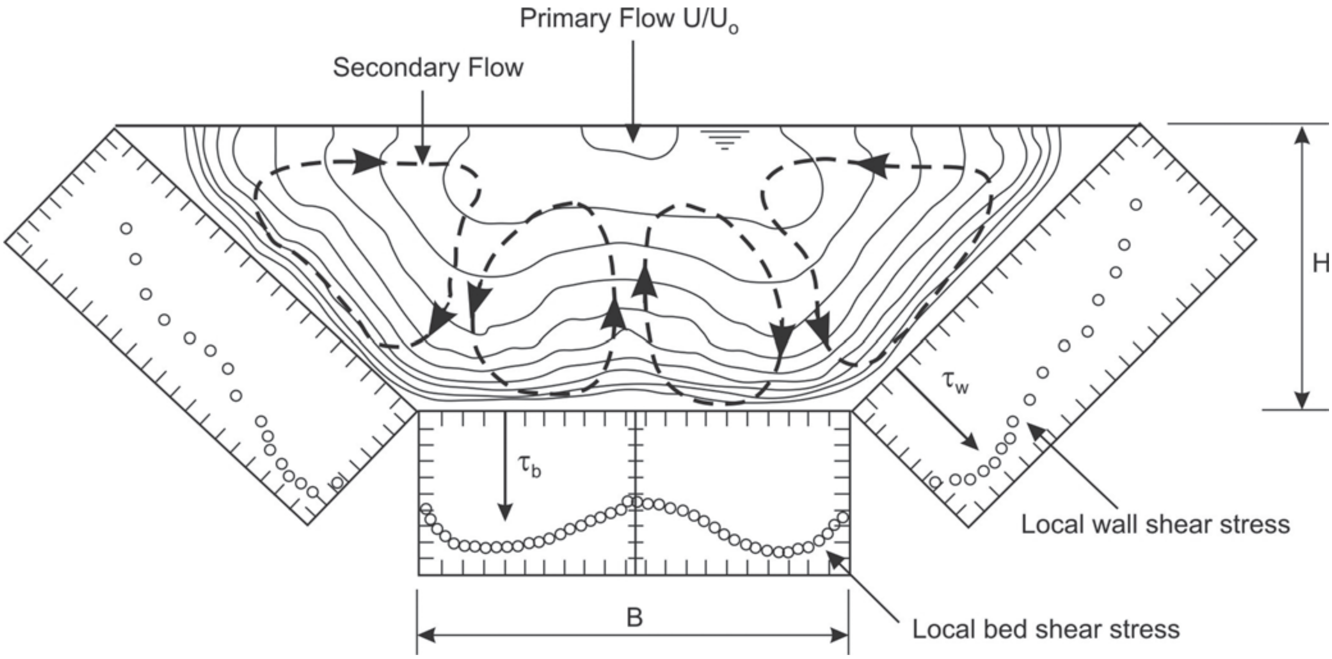

Fig. 7-5. Typical relationship between boundary shear stress distribution, secondary currents, and primary velocities in trapezoidal channel. The Froude number $\mathsf{F} = 3.24$, width/depth $= 1.52$. (Adapted from Knight et al. 1994. Copyright John Wiley & Sons Limited. Reproduced with permission.)

of the channel causes the flow in the entire cross section to be influenced by 3D flow structures, unlike the case shown in Fig. 7-4. In the wide-channel case, flow in the central region is almost 2D, provided that $y/2b < 0.85$, and for this condition standard boundary layer distributions may be assumed for velocity and Reynolds stress. However, even in a wide ($B/H > 20$) channel, although it may be acceptable to ignore bank effects for many hydraulic and geomorphic analyses, the bank still influences the flow in the near-bank zones sufficiently to require that the resulting 3D flow structures be accounted for in models of width adjustment.

Secondary flow cells may be generated by anisotropic turbulence (stress-induced secondary currents) or streamwise curvature (skew-induced secondary currents) and are always present in any turbulent flow along a channel with a noncircular cross section, such as a natural river channel (Einstein and Li 1958; Liggett et al. 1965; Tracy 1965; Perkins 1970; Melling and Whitelaw 1976; Chiu and Hsiung 1981; Naot and Rodi 1982; Nezu 1993; Meyer and Rehme 1994). In straight channels, stress-induced secondary velocities are usually small, typically being 1 to 2% of the primary velocity. Modeling these weak motions is especially difficult in complex cross sections such as those of natural rivers. In meandering channels, skew-induced secondary velocities may be as great as 10 to 20% of the primary flow, and they are known to affect the distributions of primary velocity and bed-shear stress significantly (Bathurst et al. 1979; Ikeda and Parker 1989; Nelson and Smith 1989a; 1989b; Shiono and Muto 1993; Knight and Shiono 1995).

River engineers are often concerned with the parameters at the channel boundary. A depth-averaged form of the streamwise equation of motion of flow in a straight channel is given by Shiono and Knight (1988; 1991) as

$$\rho g H_{S_0} - \frac{1}{8}\rho f U_d^2\left(1+\frac{1}{s^2}\right)^{1/2} + \frac{\partial}{\partial y}\left(\rho \lambda H^2\left(\frac{f}{8}\right)^{1/2} U_d \frac{\partial U_d}{\partial y}\right) = \frac{\partial}{\partial y}[H(\rho UV)_d] \quad (7\text{-}1)$$

where

ρ = water density;
g = acceleration due to gravity;
H = water depth;
U = streamwise velocity;
V = cross-stream velocity;
y = lateral distance across the channel;
S_0 = streamwise channel slope;
s = local channel side slope of the banks; and
U_d = depth-averaged mean velocity, defined by

$$U_d = \frac{1}{H}\int_0^H U dz \quad (7\text{-}2)$$

Three coefficients, f, λ, and Γ are introduced to deal with the local friction factor, dimensionless eddy viscosity, and secondary flow parameter, defined respectively by

$$\tau_b = \frac{f}{8}\rho U_d^2 \tag{7-3a}$$

$$\overline{\tau}_{yx} = \rho \overline{\varepsilon}_{yx} \frac{\partial U_d}{\partial y} \tag{7-3b}$$

$$\overline{\varepsilon}_{yx} = \lambda U_* H \tag{7-3c}$$

$$\frac{\partial \mathrm{H}(\rho \mathrm{UV})_{\mathrm{d}}}{\partial \mathrm{y}} = \Gamma \tag{7-3d}$$

where

$\overline{\varepsilon}_{yx}$ = depth-averaged eddy viscosity; and
τ_b = local boundary shear stress.

Equation (7-1) governs lateral distributions of U_d or τ_b across the channel width, provided that appropriate values of f, λ, and Γ are specified for each boundary element. Applications of this model to both in-bank and over-bank flow are described in Shiono and Knight (1990) and Abril (1995).

The depth-mean apparent shear stress τ_a acting on a vertical plane in the streamwise direction of a river cross section may be determined by laterally integrating (7-1) to give

$$\tau_a = -\frac{1}{H}\int_0^y \left[\rho g H S_0 - \tau_b\left(1+\frac{1}{s^2}\right)^{1/2}\right] dy \tag{7-4}$$

A comparison between (7-1) and (7-4) illustrates that the depth-averaged apparent shear stress has two distinct components, one arising from depth-averaged secondary flow motion and the other from turbulent Reynolds stresses. It is this apparent shear stress that is often required in sediment transport models, because it links the local boundary shear stress at a point on the wetted perimeter with the streamwise resolved weight force and the resultant net stress. Some values of τa are shown in Fig. 7-6 for trapezoidal channels with roughened banks and width/depth ratios between 4 and 10.

Significantly, the right-hand side of (7-1) contains a depth-averaged secondary flow term. This term is often ignored in stream-tube models and in some eddy viscosity/width-adjustment models (Wark et al. 1990; Darby and Thorne 1992; James and Wark 1994; Kovacs and Parker 1994). However, this term is, in fact, important, as is well illustrated by the examples of Knight and Abril (unpublished paper 1996) using benchmarked experimental data from the U.K. Flood Channel Facility (Knight and Sellin 1987; HR Wallingford 1992).

From the point of view of sediment transport, although it is known that the entrainment and motion of grains may be correlated with turbulent bursts and sweeps (Jackson 1976b; Raudkivi 1995), inclusion of burst and sweep phenomena in a practical model of width adjustment is at this stage premature, owing to our lack of knowledge concerning all the details of coherent structures in the boundary layer close to the bed (Tehrani 1992; Ashworth et al. 1996). As a result, one of the flow parameters still most closely associated with sediment motion is the local time-averaged boundary shear

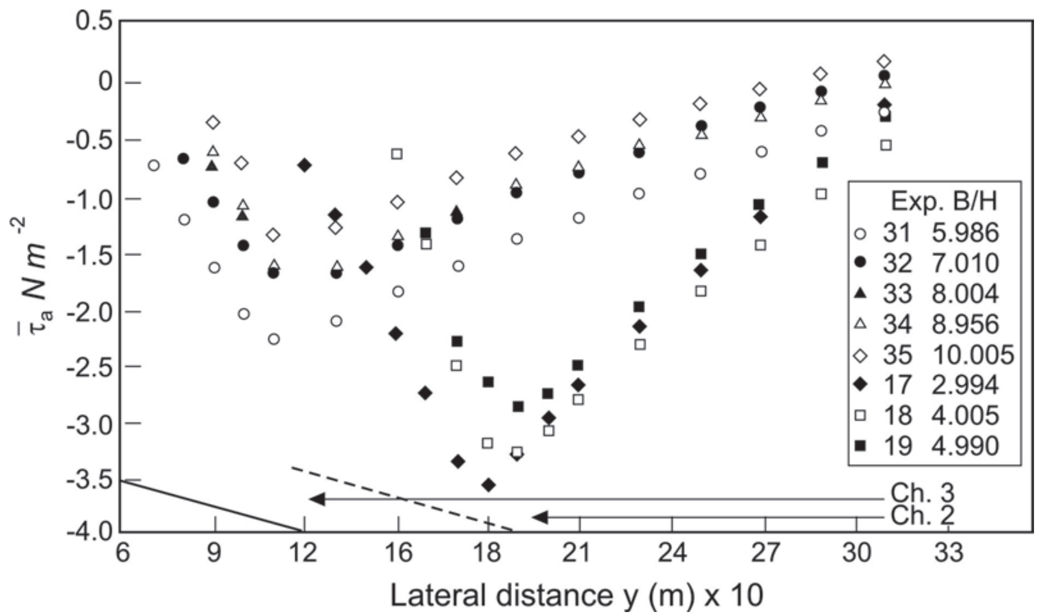

Fig. 7-6. Lateral variation of depth-averaged apparent shear stress τ_a for trapezoidal channels with roughened walls and smooth bed (ASCE Task Committee on Hydraulics, Bank Mechanisms and Modeling of River Width Adjustment 1998a, with permission from ASCE).

stress. Many researchers have attempted either to predict or to measure the lateral distribution of local time-averaged boundary shear stress around the wetted perimeters of channels of various shapes and the longitudinal distribution of local boundary shear stress over sand dunes or in channel reaches. These studies have usually been conducted at laboratory scale using, for convenience, rectangular, trapezoidal, or lenticular cross sections (Engelund 1964; Lundgren and Jonsson 1964; Knight et al. 1994; Rhodes and Knight 1994), and only a few studies have been undertaken in the field at full scale (Bathurst et al. 1979; Dietrich and Whiting 1989; Nece and Smith 1970).

Although the applicability of laboratory-based work to field situations is limited, most previous studies of the relevant hydraulic processes have been carried out under carefully controlled laboratory conditions. Figures 7-7 and 7-8 illustrate how the average wall or bank stress τ_w and bed shear stress τ_b vary for uniformly roughened trapezoidal channels (side slope angles of 45°, 68°, and 90°) and how they compare with simple exponential equations. These data

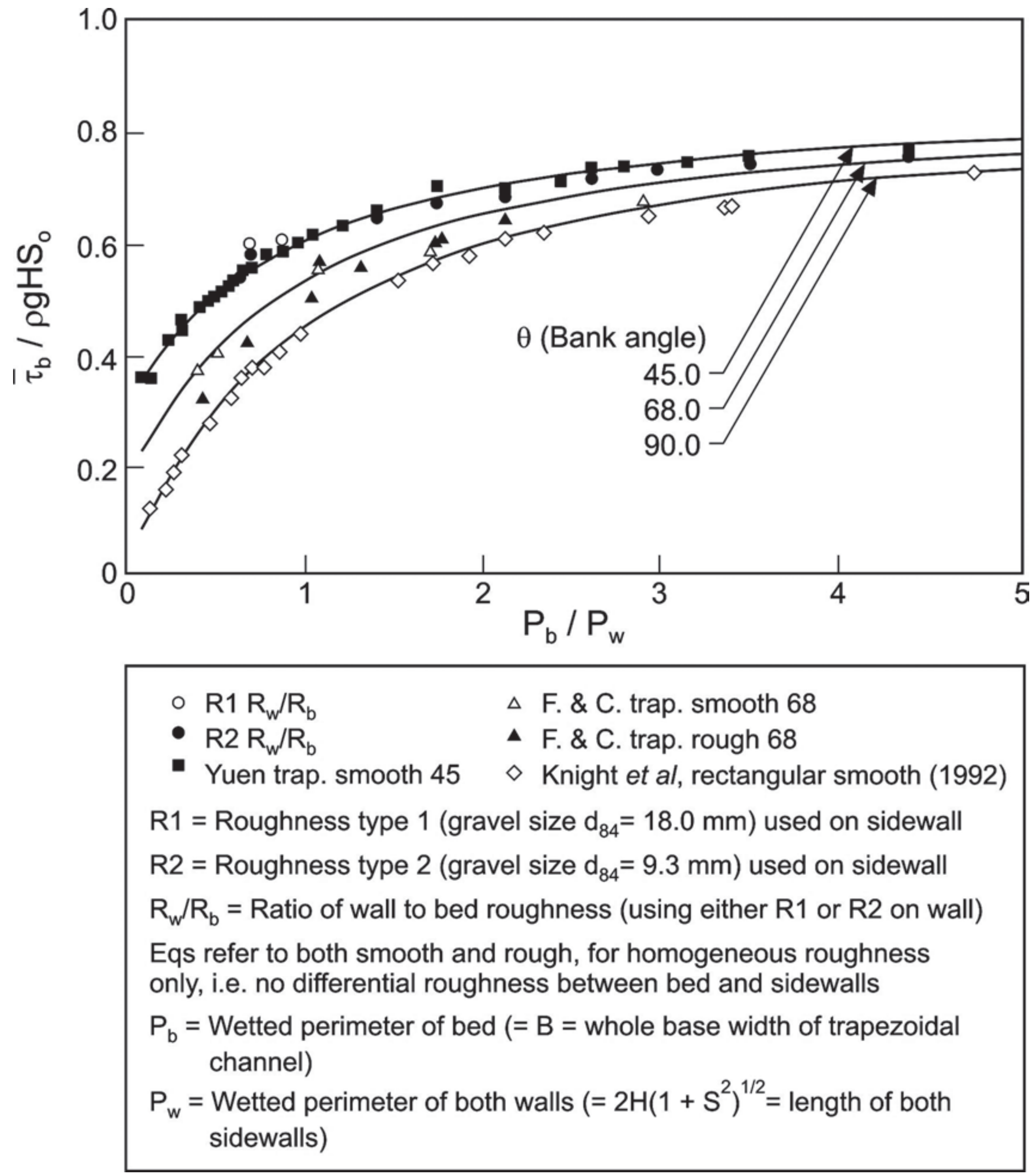

Fig. 7-7. Average wall shear stress $\tau_b/\rho g H S_o$ for smooth and rough trapezoidal channels with different side-slope angles (ASCE Task Committee on Hydraulics, Bank Mechanisms and Modeling of River Width Adjustment 1998a, with permission from ASCE).

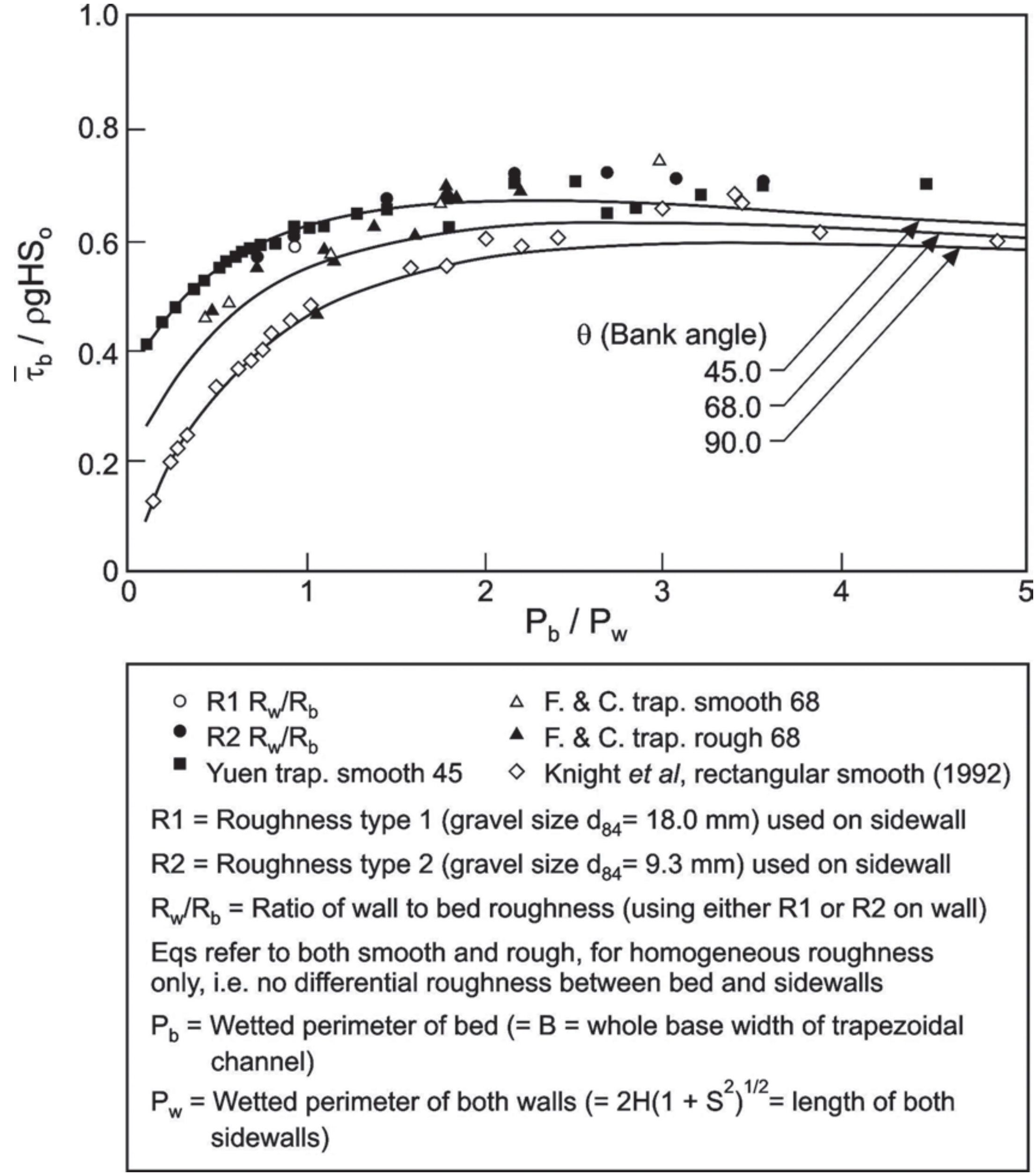

Fig. 7-8. Average wall shear stress $\tau_b/\rho g H S_o$ for smooth and rough trapezoidal channels with different side-slope angles (ASCE Task Committee on Hydraulics, Bank Mechanisms and Modeling of River Width Adjustment 1998a, with permission from ASCE).

all relate to in-bank flows. Similar plots and equations are available for the maximum stresses on the bed and banks, together with their locations, for both uniformly and non-uniformly roughened channels. High differential roughness may occur in engineered channels (with portions of the wetted perimeter especially rough due to riprap) or in natural channels (with banks that are significantly rougher than the bed due to dense riparian vegetation).

Because lenticular shapes more closely approximate the shape of natural alluvial channels they have often been the focus of river studies (Lundgren and Jonsson 1964; Ikeda 1981; Kovacs and Parker 1994). Five methods for determining the local boundary shear stress were reviewed by Lundgren and Jonsson (1964), with the area method being found to be most suitable for general use. Distributions of boundary shear stress based on a particular lenticular shape (Fig. 7-9) are shown for all five methods in Fig. 7-10. The way in which a noncohesive riverbank might be eroded into an equilibrium shape under the action of these distributions of applied shear stress is illustrated in Figs. 7-11 to 7-13 (Kovacs and

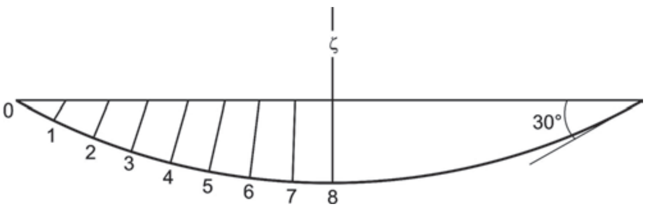

Fig. 7-9. Cross section used in numerical example (circular) (ASCE Task Committee on Hydraulics, Bank Mechanisms and Modeling of River Width Adjustment 1998a, with permission from ASCE).

Parker 1994). In these sketches the lateral distribution of streamwise boundary shear stress is of special significance because those regions of the cross section in which the stress is above the transport threshold will become active with regard to bed-material transport and will form the active width. The resulting cross-sectional shapes for straight alluvial channels under both threshold and mobile bed conditions have also been the subject of much investigation. However, it should be noted that, because the discharge and sediment supply in natural rivers are highly variable, the actual channel cross-sectional shape is constantly responding to changing stage bed forms and flow resistance. Hence, it is only possible to predict medium-term (5–10 years) time-averaged, cross-sectional shapes.

For over-bank flows in straight and meandering channels, considerably fewer experimental data are found with regard to both velocity and boundary shear stress (Knight and Demetriou 1983; Knight et al. 1989, 1990; Tominaga and Nezu 1991; Ackers 1992; Knight et al. 1992; Ackers 1993; Sellin et al. 1993; Tominaga and Nezu 1993). In general terms, for straight channels with floodplains the boundary shear stresses under over-bank flows vary in a more complex way than those for in-bank flows, with stresses in the main river channel decreasing because of the influence of the slower floodplain flows. Conversely, the floodplain boundary shear stresses rise above their expected 2D values because of the effect of the faster flowing, main river flow. Interaction between channel and floodplain flows results in some localized and complex effects in the vicinity of the

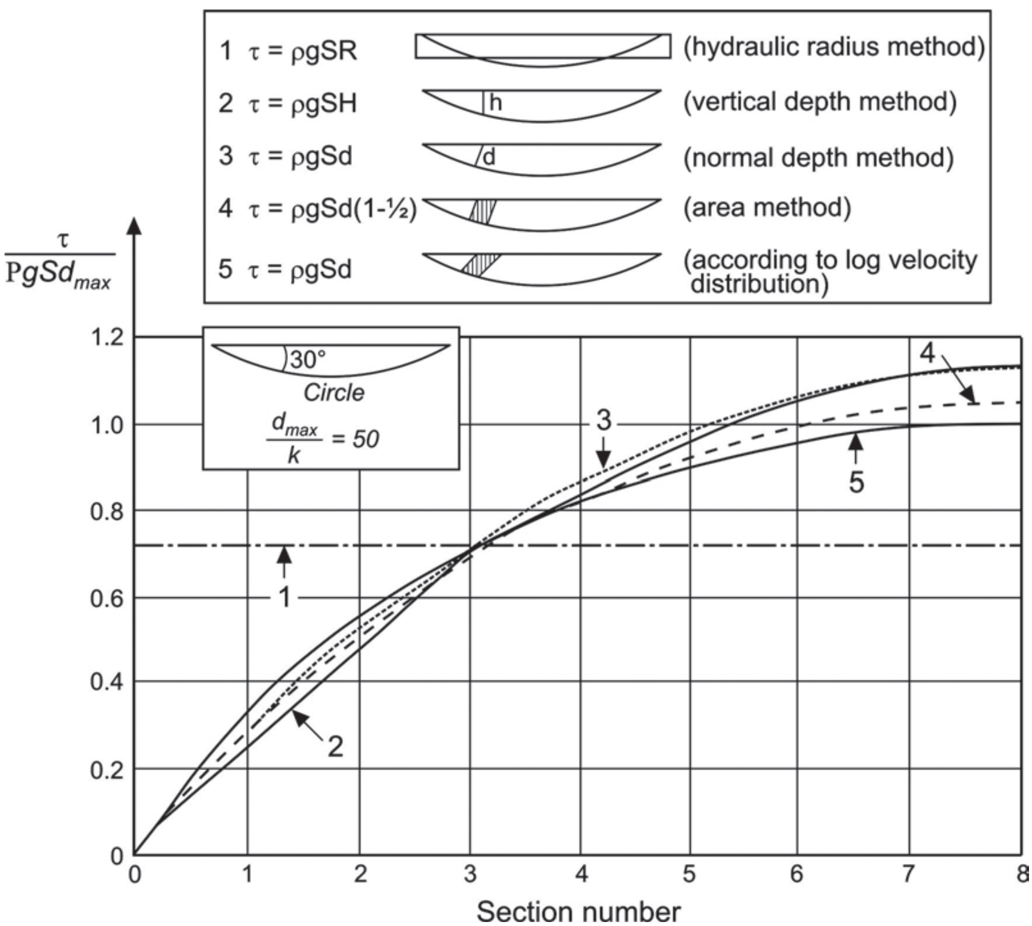

Fig. 7-10. Comparison between five different methods of determining shear stress distribution (ASCE Task Committee on Hydraulics, Bank Mechanisms and Modeling of River Width Adjustment 1998a, with permission from ASCE).

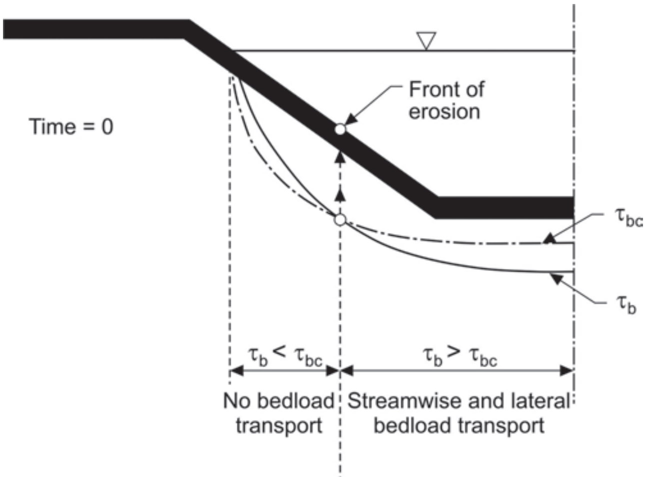

Fig. 7-11. Definition of front of erosion for initially straight trapezoidal cross section at time 0 (ASCE Task Committee on Hydraulics, Bank Mechanisms and Modeling of River Width Adjustment 1998a, with permission from ASCE).

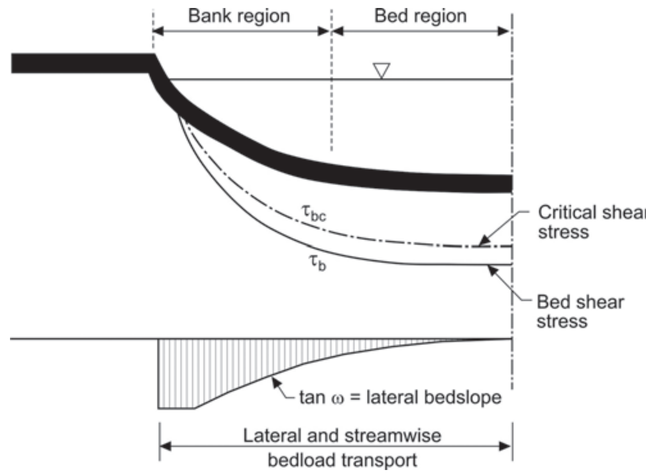

Fig. 7-12. Sketch of distribution of shear stress τ_b and lateral bed slope tan ω along perimeter of straight channel cross section during development of stable profile (ASCE Task Committee on Hydraulics, Bank Mechanisms and Modeling of River Width Adjustment 1998a, with permission from ASCE).

main channel riverbanks. These features are discussed by Knight and Cao (1994) in relation to large-scale experimental studies and by Knight and Shiono (1995) in relation to the relative importance of the three terms in (7-1).

The redistribution of boundary shear stress within the cross section during over-bank flow also has a profound effect upon sediment transport rate, as shown by Ackers (1992) and Abril (1995). It will consequently also have an effect on bank adjustments that occur when the river overflows its bank.

7.3.2.3 Longitudinal Changes Unlike artificially constructed channels, river channel cross sections are not generally uniform trapezoids. Consequently, width, depth, slope, and planform change significantly in the streamwise direction. Some schematization of the natural river is therefore necessary before a numerical model is constructed (Samuels 1990), and inevitably some streamwise averaging of cross-sectional area, hydraulic radius, energy gradient, mean boundary shear stress, etc. must be performed. This is potentially a source of error in any representation of the channel hydraulics and

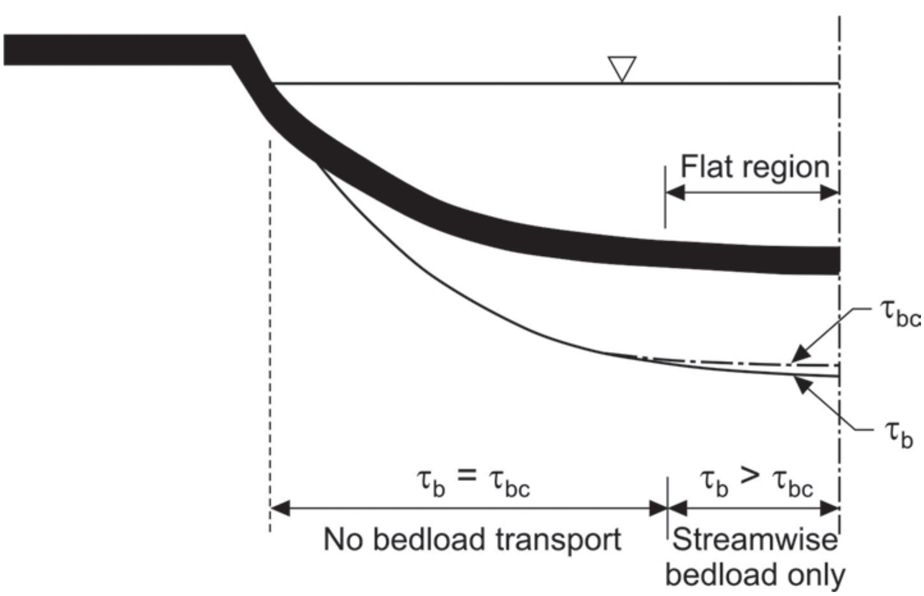

Fig. 7-13. Shear stress distribution in state of dynamic equilibrium (ASCE Task Committee on Hydraulics, Bank Mechanisms and Modeling of River Width Adjustment 1998a, with permission from ASCE).

must be treated with appropriate care (McBean and Perkins 1975; Cunge et al. 1980; Laurenson 1986). Samuels (1989; 1990) used perturbation analysis of the steady-flow equation to show that weighting the friction slope toward the upstream section gives considerably improved accuracy. For example, the longitudinal spacing between channel cross sections may be doubled if weighting coefficients are used rather than the arithmetic mean, with the same accuracy in water levels being achieved. However, the correct representation of the energy gradient or water-surface slope is difficult to achieve in natural channels where width, pool-riffle sequences (especially in gravel-bed rivers), and pool-crossing geometry (in sinuous rivers) introduce marked channel variability. In these cases, channel schematization, even using a weighting technique, may be inadequate. Further work is needed on the derivation of representative reach-averaged parameters and their significance in 1D models.

In meandering alluvial rivers, the boundary shear stress distribution, bed topography, bed load transport rate, and channel cross-sectional shape all vary considerably over the meander wavelength due to flow curvature effects. Field data (Dietrich and Smith, 1983, 1984, and many others) indicate that the near-bed flow and bed load transport along the outside bank of a meander bend are directed toward the inside bank (Fig. 7-14), leading to the development of a deep pool along the outside of the bend. On the inside bank, shoaling of the flow over the point bar causes the near-bank velocity to be directed toward the outside bank. Because the development of the pool can increase the effective height of the outside bank, scour on the outside of meander bends is an important hydraulic process that promotes bank erosion. Smith and McLean (1984) and Nelson and Smith (1989a; 1989b) have developed effective numerical models for computing flow and transport processes in meander bends, although these models are not accurate close to the bank itself because lateral momentum diffusion terms in the governing equations are neglected.

In overbank flows on meandering rivers, hydraulic processes are extremely complex and difficult to predict. Laboratory studies (Tominaga and Neza 1991; Sellin et al. 1993; Tominaga and Neza 1993; Wark et al. 1994; Knight and Shiono 1995) show that new flow structures are introduced during over-bank flow, and field studies (Fukuoka 1993; 1994; Lawler 1993a; 1993b) indicate that over-bank flows are strongly influenced by local morphological features.

7.3.2.4 Near-Bank Zone Knowledge of the velocity, boundary shear stress, secondary flows, and turbulence structure close to a riverbank is required before fluvial processes can be linked to channel-width adjustment. The preceding sections have highlighted current difficulties in predicting the details of near-bank hydraulics, even for relatively simple prismatic channels, whether flowing in-bank or over-bank. The 3D nature of near-bank flow leads to nonlinear distributions of Reynolds stresses normal to the boundary and velocity profiles that are not logarithmic (Knight and Shiono 1990; Shiono and Knight 1991; Meyer and Rehme 1994; Knight and Shiono 1995). The effect of these flow complexities on boundary shear stress distributions in the vicinity of riverbanks has been illustrated by Knight and Cao (1994).

At present, no simple formulas exist to characterize the lateral distribution of local time-averaged boundary shear stress around the wetted perimeter of a natural channel, although Figs. 7-7 and 7-8 give some guidance, and experimental data, such as those shown in Figs. 7-4 and 7-5, indicate how local values may vary about the cross-sectional mean. Consequently, boundary shear stresses in the near-bank zone either have been estimated from trends established experimentally, such as those shown in Figs. 7-4 to 7-10, or else have been obtained from turbulence models, of which the nonlinear k-ε (k is the turbulent kinetic energy and ε is the turbulent dissipation rate) and large-eddy simulation models are probably the most appropriate (Rodi 1980; Nezu and Nakagawa 1993; Thomas and Williams 1995; Younis 1996). Assuming that these local values are known, Figs. 7-11 to 7-13 show how the boundary shear stress and bed load region are often conceptualized in a typical width adjustment model (Ikeda and Izumi 1991; Kovacs and Parker 1994; Knight and Yu 1995). It should be remembered, however, that although the boundary shear stress is arguably one of the more important connecting links between the flow field and the distributions of erosion (or scour), deposition, and channel change (Breusers and Raudkivi 1991), it is not necessarily the dominant parameter responsible for bank erosion or width adjustment. Process dominance also depends on the geomorphic context within which width adjustment is taking place, the channel type (Fukuoka et al. 1993), and location within the watershed (Lawler 1992).

7.3.2.5 Adjustment of Channel Boundaries in Near-Bank Zone The adjustment of alluvial channel boundaries is usually related to spatially averaged hydraulic parameters, such as boundary shear stress, streamwise stream power, and energy gradient, together with data defining the net sediment supply to the system and the bank material properties (Molinas and Yang 1986; Hasegawa and Mochizuki 1987; Chang 1988a, 1988b; Hasegawa 1989; Wiele and Paola 1989; Pizzuto 1990; Lawler 1993a; Parker 1995). The variability of channel morphology and complexity of the turbulent flows described earlier might suggest that a probabilistic approach is more suitable than the deterministic treatments described previously. Whichever approach is selected, the same key hydraulic parameters are still likely to be included in process equations or functions and should be represented as faithfully as possible, despite some implicit longitudinal smoothing of localized flow structures or morphological features. The appropriate inclusion of 3D phenomena in either a 1D model or a depth-averaged model is still awaited and is likely to be derived from detailed 3D numerical simulations (ASCE Task Committee on Turbulence Models in Hydraulic Computation 1988; Li and Wang 1994a,b; Thomas and Williams 1995; Younis 1996). However, even allowing for future advances in representing the 3D flow,

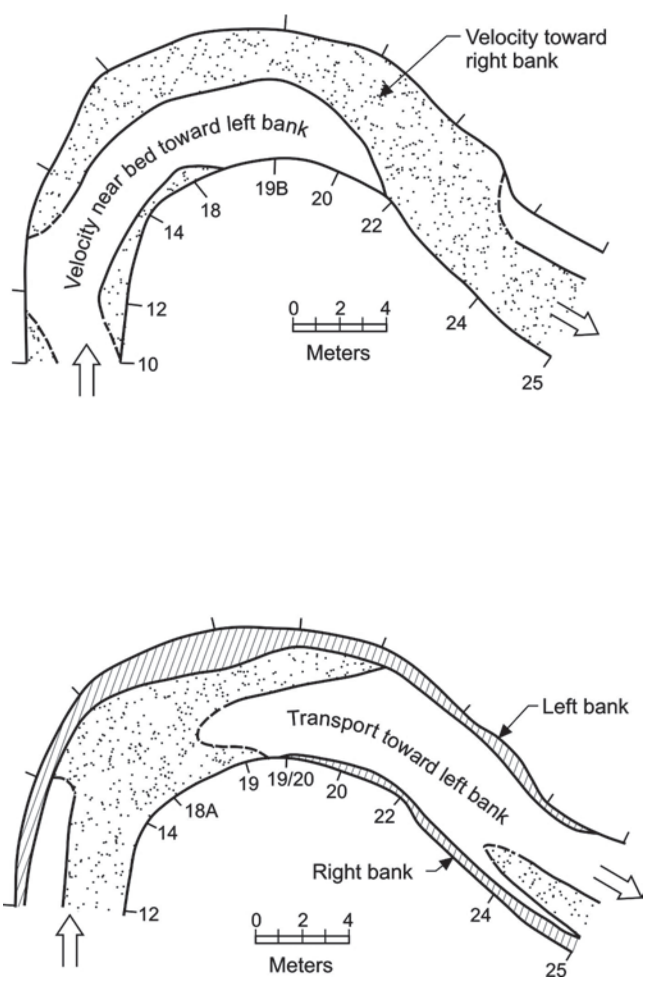

Fig. 7-14. Top: direction of flow near the bed in a meander bend determined using current meter data (Dietrich and Smith 1983). Bottom: direction of bed load transport in a meander bend. The shaded area represents bed load transport toward the right bank, whereas the clear area denotes bed load transport toward the left bank. The dashed lines indicate uncertainty as to correct position of the boundary between fields. Diagonal lines define the area occupied by submerged bank and immobile gravel (Dietrich and Smith 1984).

a comprehensive theoretical framework for determining the equilibrium form of stable alluvial channels still needs to be developed before attempting to simulate changes from the equilibrium profile.

Identification of the junction point between active (eroding) and inactive (noneroding) elements of the bank, together with characterization of the erosion front that moves this point, appears to be crucial to quantifying width adjustment. Hydraulic conditions at the active-inactive junction are especially difficult to determine with precision, even for in-bank flows. Near-bank hydraulic conditions for over-bank flows are strongly influenced by secondary flow structures close to the banks and must be represented carefully through the correct use of local friction factors, eddy viscosities, and depth-averaged secondary flow values. For this purpose, the depth-averaged approach described earlier is worthy of further study. Although this approach still has major drawbacks, a need still exists for calibration of specific channel shapes, and no details of vertical motion are available from the process equations.

It is also important to know the rates at which width, depth, slope, and local morphological adjustments are made, so that errors can be assessed when an incremental series of quasi-steady-state discharges are used to simulate a hydrograph. The dominant or effective discharge responsible for forming channel morphology, although easy to define in theory, is still poorly understood and, except for work by Ackers (1992), very little attention has been paid to the influence of over-bank flows on dominant discharge. The hypothesis that in an equilibrium channel the bank-full stage corresponds to the dominant or effective discharge has some theoretical basis, but it may be a special case within a variety of associations between important features of channel morphology and a range of effective flows (Hey 1975; Thorne et al. 1993; Biedenharn and Thorne 1994). Further experimental work on equilibrium and nonequilibrium alluvial channels is required before linkages between dominant discharge, the range of effective flows, and channel morphology can be substantiated.

7.3.3 Bank Mechanics

The fundamental processes responsible for channel-width adjustment are fluvial erosion, fluvial deposition, and mass bank failure. The following seven topics concerned with the mechanics of bankline movement are addressed in this section: (1) bank erosion; (2) weakening of resistance to erosion; (3) bank stability with respect to mass failure; (4) basal endpoint control; (5) effects of vegetation; (6) seepage effects; and (7) bank advance.

7.3.3.1 Bank Erosion Water flowing in an alluvial channel exerts forces of drag and lift on the boundaries that tend to detach and entrain surface particles. To remain in place, the boundary sediment must be able to supply an internally derived force capable of resisting the erosive forces applied by the flow. The origin of these resisting forces varies according to the grain size, the size distribution, and the nature of electrochemical bonding that may exist between particles. Alluvial bank materials are formed primarily by fluvial deposition and are often stratified, with a general fining-upward sequence. Therefore, the engineering characteristics and erodibility of the bank may vary with elevation. Also, floodplain deposits typically include alluvial sands and gravels, clay plugs, and strongly cohesive backswamp deposits, so that bank material properties vary spatially over relatively short distances. Although the distribution of sustained bank retreat along the course of a river depends primarily on the distribution of boundary shear stress in the near-bank zones, outcrops of particularly resistant material may act to slow the local bank retreat rate and to distort the fluvially driven pattern of channel planform evolution (Sun et al. 1996).

In the case of noncohesive sands and gravels, the forces resisting erosion are generated mainly by the immersed weight of the particles, although close packing of grains in imbricated patterns can also wedge particles in place, greatly increasing the critical boundary shear stress necessary for entrainment. Generally, the mobility of noncohesive bank materials can be predicted using a Shields-type entrainment function, but this must be modified to take into account the destabilizing effect of channel side slope. Also, the critical value of the dimensionless shear stress must be adjusted to allow for excessive tightness or looseness in packing of bank material particles (Thorne 1982).

Fine-grained bank materials, containing significant amounts of silt and clay, are to some degree cohesive and resist entrainment primarily through interparticle electrochemical bonding rather than through the immersed weight of the particles. When cohesive bank materials are entrained by the flow, it is aggregates of grains (such as soil crumbs or peds that have been produced by soil-forming processes) that are detached. Fluvial entrainment, therefore, requires that the local boundary shear stresses exceed the critical value to initiate motion of crumbs or peds rather than that related to the primary soil particles. Ped size and stability and interped bonding strength are not conservative soil properties, because they depend to some degree on the local history of soil development, in general, and recent antecedent conditions of wetting and drying, in particular. It follows that the conditions of incipient motion for cohesive bank materials are complex, time-dependent, and difficult to define.

A task committee (ASCE Task Committee on Erosion of Cohesive Sediments 1968) summarized early studies into the mechanics of cohesive bank erosion. The task committee recorded the results of noteworthy contributions by, among others, Smerdon and Beasley (1961) and Flaxman (1963) on channel stability in cohesive materials and Grissinger and Asmussen (1963) and Grissinger (1966) on the erodibility of cohesive soils. For example, Grissinger and Asmussen (1963) found that erosion resistance of clayey soils increased with the time that the materials were wetted. They postulated that when clay is initially wetted, the free water releases bonds between particles, but that, as free water is absorbed, the clay minerals hydrate and interparticle bonds are strengthened. This illustrates how the chemical bonding of clay particles may vary with time and the history of soil moisture changes.

A wealth of subsequent work has further addressed fundamental aspects of cohesive soil behavior, leading to important papers by Parthenaides and Passwell (1970); Kandiah and Arulanandan (1974); Arulanandan (1975); Arulanandan et al. (1975); Ariathurai and Krone (1976); Ariathurai and Arulanandan (1978); Abt (1980); Grissinger (1982)—who presents an excellent review of progress achieved up to the early 1980s; Kamphuis and Hall (1983); Parchure and Mehta (1985); Springer et al. (1985); Shaikh et al. (1988a; 1988b); and, more recently, Annadale and Parkhill (1995) and Kranenburg and Winterwerp (1997). Space limitations preclude an in-depth review of the findings of these papers here. However, in summary it can be concluded that although critical boundary shear stresses for cohesive bank soils are extremely difficult to predict accurately (Grissinger 1982), they tend to be higher than those for noncohesive bank materials. As a result, erosion rates for cohesive banks are generally lower than those for noncohesive materials (Vanoni 1975; Thorne and Tovey 1981).

Once entrained, crumbs and peds disintegrate rapidly due to corrasion at the channel boundaries and turbulent buffeting in the flow, so that most silt- and clay-sized sediment derived from bank erosion is transported in suspension and is conventionally classified as wash load.

7.3.3.2 Weakening of Resistance to Erosion The erodibility of bank soils can be increased markedly by processes of weakening and weathering. The processes responsible for loosening and detaching grains and aggregates are closely associated with soil moisture conditions at and beneath the bank surface. In poorly drained soils, positive pore-water pressures act to reduce bank stability, which can lead to bank failure, particularly during rapid drawdown of the channel stage following a high flow. Conversely, rapid immersion of a dry bank can lead to slaking, which is the detachment of aggregates by positive pore pressures due to compression of trapped air.

Changes in moisture content and freezing and thawing can significantly influence the erodibility of a riverbank. Swelling and shrinkage of soils during repeated cycles of wetting and drying can contribute to cracking that significantly increases erodibility and reduces soil shear strength. Shrinkage is especially damaging to the strength of the bank when intense drying of the soil leads to desiccation cracking. Heaving due to the 9% increase in water volume on freezing (Ritter 1978) and the growth of needle ice crystals at the bank surface, followed by collapse of ice wedges and needles during thawing of soil moisture, are highly effective in increasing the susceptibility of cohesive bank materials to flow erosion (Lawler 1993b).

Temporal variability in the erodibility of bank soils due to the operation of weakening processes means that the effectiveness of a given flow event in eroding a bank depends not only on the magnitude and duration of a particular event but also on antecedent conditions (Wolman 1959).

7.3.3.3 Bank Stability with Respect to Mass Failure Fluvial erosion drives bank retreat directly by removing material from the bank face, but it often also causes bank retreat by triggering mass instability. The stability of a bank with regard to mass failure depends on the balance between gravitational forces, which tend to move soil downslope, and forces of friction and cohesion, which resist movement. Failure of the bank occurs when scour of the bed next to the bank toe increases the bank height, or when undercutting increases the bank angle, to the point that motivating forces exceed restoring forces on the most critical potential failure surface, and the bank collapses in a gravity-induced mass failure.

The analysis of slope stability with respect to mass failure has been the topic of considerable research, primarily by geotechnical engineers but also by geomorphologists and geophysicists. Engineering research has concentrated on development of engineering designs for artificial slopes and embankments, but rather little of this work is applicable to the very steep slopes, undisturbed soils, complex sedimentary layering, and unspecified drainage conditions found in eroding natural riverbanks. Also, application of most geotechnical analyses requires detailed site investigation to provide the necessary data on profile geometry, soil properties, bank stratigraphy, and ground-water flow net. Although it is possible to collect such detailed information for a specific construction site or key location, the data obtained cannot easily be generalized to represent bank conditions along a reach of a river, due to inherent variability in the properties of natural alluvium and uncertainty concerning the local bank environment.

In fact, relatively little research specifically concerned with streambank stability has been undertaken, and the following brief review of slope stability literature concentrates on more recent work that is directly relevant to mass failure of riverbanks. However, it is acknowledged that treatment of riverbank stability shares a common origin with that of engineered slopes and embankments in the fundamental work of researchers, such as Bishop (1955); Peck and Deere (1958); Bishop and Morgenstern (1960); Morgenstern and Price (1965); Spencer (1967); Terzaghi and Peck (1967); Vaughan and Walbancke (1973); Fredlund and Krahn (1977); and Poulos et al. (1985).

There is a clear contrast in failure mechanics between noncohesive and cohesive materials because of significant differences in their soil properties. In a noncohesive bank, shear strength increases more rapidly with depth than does shear stress, so that critical conditions are more likely to occur at shallow depths. In a cohesive bank, shear stress increases more quickly than shear strength with increasing depth, so that critical surfaces tend to be located deep within the bank (Terzaghi and Peck 1967).

Noncohesive materials usually fail by dislodgement and avalanching of individual particles or by shear failure along shallow, very slightly curved slip surfaces. Deep-seated failures occur in cohesive materials with a block of disturbed, but more or less intact, bank material sliding into the channel along a curved failure surface. In high banks with shallow slope angles ($\theta < 60°$), the failure surface is curved and the block tends to rotate back toward the bank as it slides in a rotational slip (Fig. 7-15). Steep banks characteristically fail along almost planar surfaces, with the detached block of soil sliding downward and outward into the channel in either a planar slip or a toppling failure (Fig. 7-16) (Thorne 1982).

Rotational slips may be defined as base, toe, or slope failures depending on where the failure arc intercepts the ground surface (Fig. 7-15(a) and are analyzed using conventional geotechnical procedures (Bishop 1955; Fredlund 1987). The risk of failure is usually expressed by a factor of safety, defined as the ratio of restoring to disturbing moments about the center of the failure circle. In the method of slices (Fig. 7-15(b), the soil body within the failure arc is divided into vertical slices with forces acting as shown. To obtain a determinate solution for the factor of safety, it is necessary to make an assumption regarding interslice forces. For example, these forces are often assumed to act horizontally. The critical slip failure circle cannot be located simply, and usually a computer program is used to explore the large number of possible solutions to determine the position of the most critical arc.

Many eroding riverbanks are very steep, and near-vertical banks often occur at the outer margins of meander bends and along severely incised channels. Such steep slopes formed in friable soils are rarely encountered in hillslope and embankment studies, and consequently, stability analyses for planar slip have received relatively little attention in the geotechnical literature. Approaches that have been developed stem from the Culmann method, in which forces acting on the potential failure block are resolved normal to and along

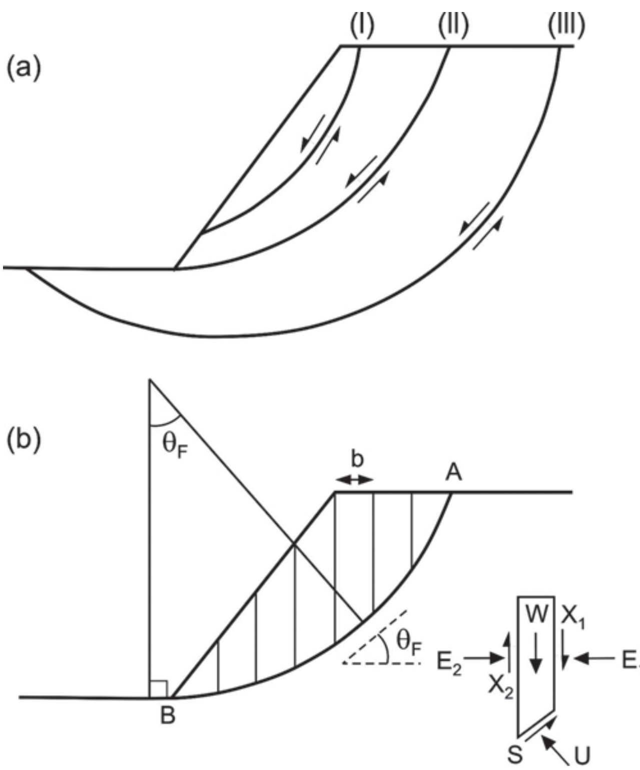

Fig. 7-15. (a) Rotational slip failures in cohesive bank: (i) slope failure, (ii) toe failure, (iii) base failure. (b) Stability analysis of slip circle by methods of slices. (After Thorne 1982. Copyright John Wiley & Sons Limited. Reproduced with permission.)

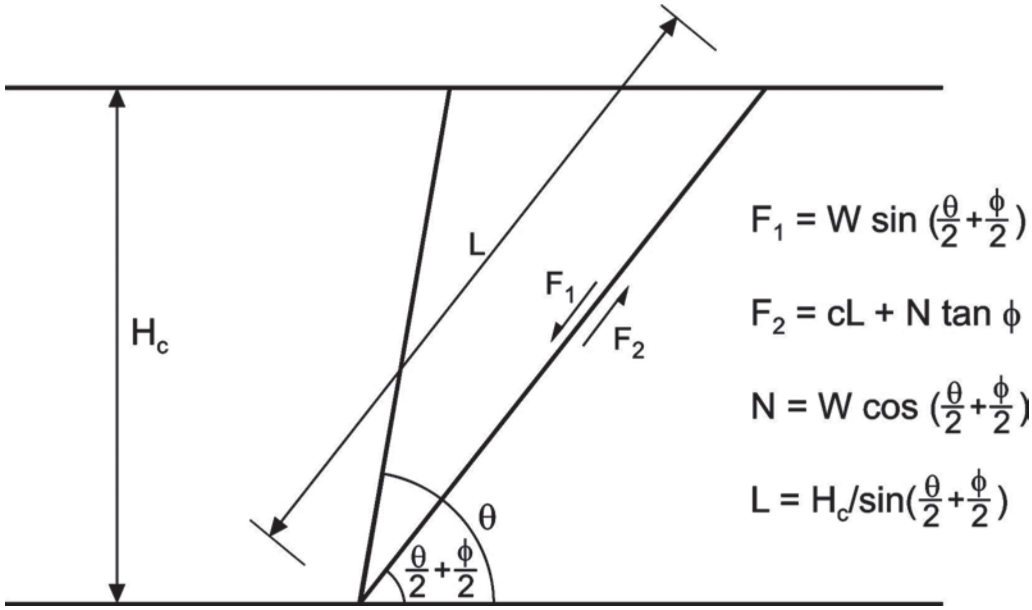

Fig. 7-16. Culmann analysis for plane slip failure. (After Thorne 1982. Copyright John Wiley & Sons Limited. Reproduced with permission.)

the failure plane, leading to the following equation to define the critical height for mass failure:

$$H_c = 4c(\sin\theta\cos\phi)/\gamma[1-\cos(\theta-\phi)] \qquad (7\text{-}5)$$

where

H_c = critical bank height;
c = cohesion;
θ = bank angle;
γ = bank material unit weight; and
ϕ = friction angle.

Tension cracks often develop downward from the ground surface, parallel to the bankline, behind steep banks because of horizontal tensile stress in the soil at this location (Terzaghi and Peck 1967). Tension cracks truncate the effective length of the potential failure surface, tending to destabilize the bank and reducing its stability relative to that predicted from a Culmann analysis. Cracks may occupy as much as half of the bank height, isolating a column or slab of soil, which then slides and topples forward into the channel in a toppling failure (Fig. 7-17).

Stability analyses applicable to the very steep (almost vertical), deeply cracked river cliffs associated with eroding, unstable streambanks have been undertaken by researchers in hydraulic engineering and fluvial geomorphology, but much more testing and validation is required before these models can be adopted for routine application as engineering design tools (Osman and Thorne 1988; Darby and Thorne 1996a; Millar and Quick 1997).

Cantilevered or overhanging banks are generated when erosion of an erodible layer in a stratified or composite bank leads to undermining of overlying, erosion-resistant layers. Thorne and Tovey (1981) pointed out that cantilevered banks may fail by shear, beam, or tensile collapse (Fig. 7-18). Shear failure (Fig. 7-18(a) occurs when the weight of the cantilever block exceeds the soil shear strength, causing the overhanging block to slip downward along a vertical plane. In a beam failure, a block rotates forward about a horizontal axis within the block (Fig. 7-18(b)) when disturbing moments about the neutral axis exceed restoring moments. Tensile failure (Fig. 7-18(c)) occurs when the tensile stress exceeds the soil tensile strength and the lower part of the overhanging block falls away. Frequently, the strength of cantilever blocks is significantly increased by root reinforcement due to riparian and floodplain vegetation. Flow erosion and tensile failure occurs below the root mat, leaving root-bound cantilevers that fail subsequently by either the beam or shear mechanism.

Whether bank failure occurs by rotational slip, toppling, or cantilever collapse, the primary force tending to move the failure block is the tangential component of the weight of the block. Fluvial erosion can increase the motivating force by increasing the bank height (through bed scour next to the bank toe) or by increasing the bank slope angle (through lateral erosion of the toe and lower bank). The weight of bank material also increases with the moisture content of the soil, and failure often follows the change from submerged to saturated conditions that occurs when drawdown occurs in the channel (Rinaldi and Casagli, 1999; Simon et al. 2000).

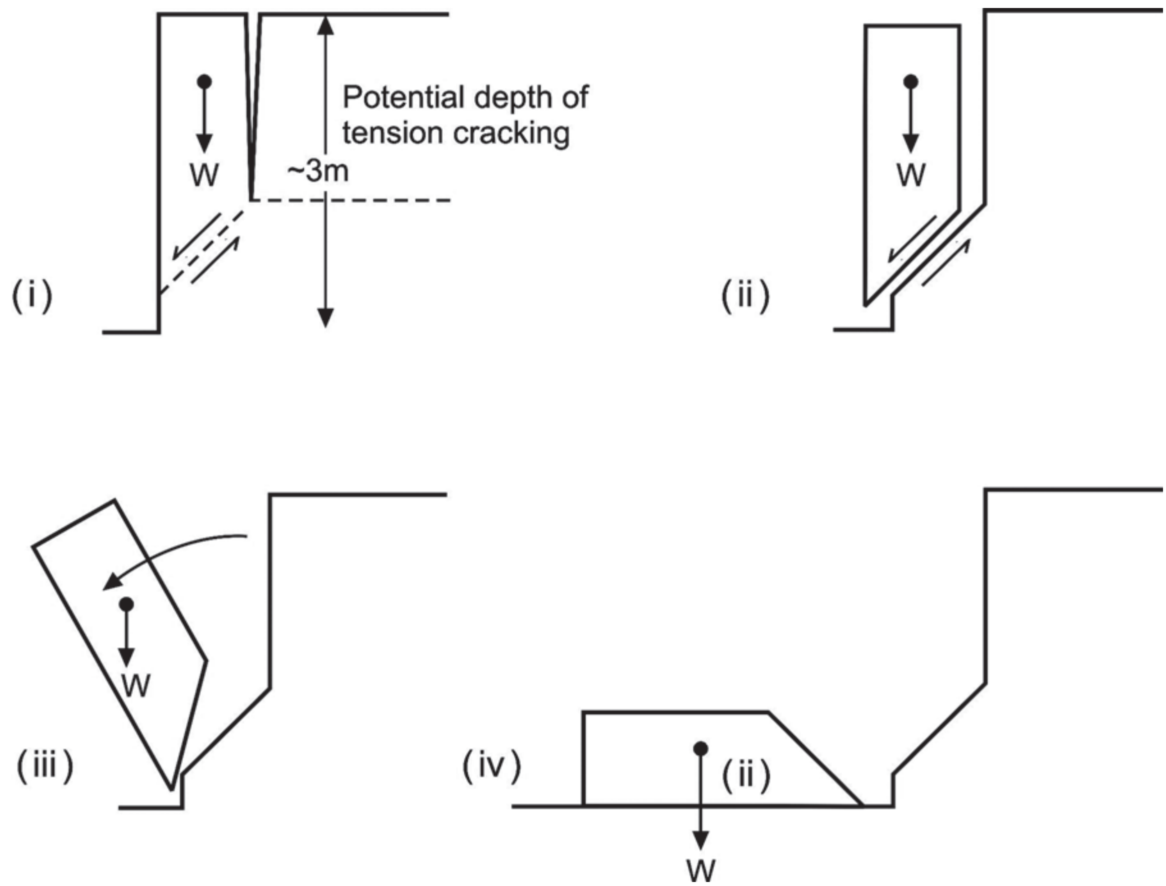

Fig. 7-17. Sequence (i–iv) of toppling failure on low steep stream bank. (After Thorne and Tovey 1981. Copyright John Wiley & Sons Limited. Reproduced with permission.)

Ample field evidence exists that bank failure may be triggered by any of these changes in motivating force. For example, Abam (1993) noted that bank failure in the Niger Delta, Nigeria, could often be attributed to increases in bank height and bank angle due to fluvial erosion and bed scour at the riverbanks. Abam (1993) also documented decreased bank stability due to rapidly falling water levels that led to (1) the loss of the confining pressure provided by the channel water level; (2) positive pore pressures due to poor drainage resulting from the low permeability of the soil; and (3) increases in the effective unit weight of the soil due to saturation.

7.3.3.4 Basal Endpoint Control Although fluvial erosion processes and geotechnical failures are controlled by different aspects of bank geomorphology, they are actually linked. The key to characterizing this link lies in recognizing that mass wasting delivers the failed material to the toe of the slope, or basal area, but does not entirely remove it from the bank profile. The removal of failed material from the basal area depends primarily on its entrainment by current and wave action, following by fluvial transport downstream. The concept of basal endpoint control explains how the medium- to long-term retreat rate of the bank is controlled by the rate of sediment entrainment and removal from the toe.

The concept of basal endpoint control was first developed by Carson and Kirkby (1972) to explain variations in hillslope profiles. Thorne (1982) applied the concept to riverbanks, proposing that bank retreat can only be sustained when the near-bank flow is able to remove failure debris and to continue to scour the basal area. In contrast, where the flow is unable to remove all the debris, basal accumulation occurs and a berm or bench of failed material develops. This tends to protect the bank from fluvial erosion and, by acting as a buttress against gravity failures, increases bank stability. On this basis, the balance of basal supply and removal of sediment can be defined by one of the following three states of basal endpoint control:

1. *Impeded removal*—Bank failures supply debris to the base at a higher rate than it is removed. Basal accumulation results, decreasing the bank angle and height and therefore increasing stability with respect to mass failure. The rate of debris supply decreases, favoring the second state.
2. *Unimpeded removal*—Processes delivering debris to the base and removing it are in balance. No changes in basal elevation or slope angle occur. The bankline

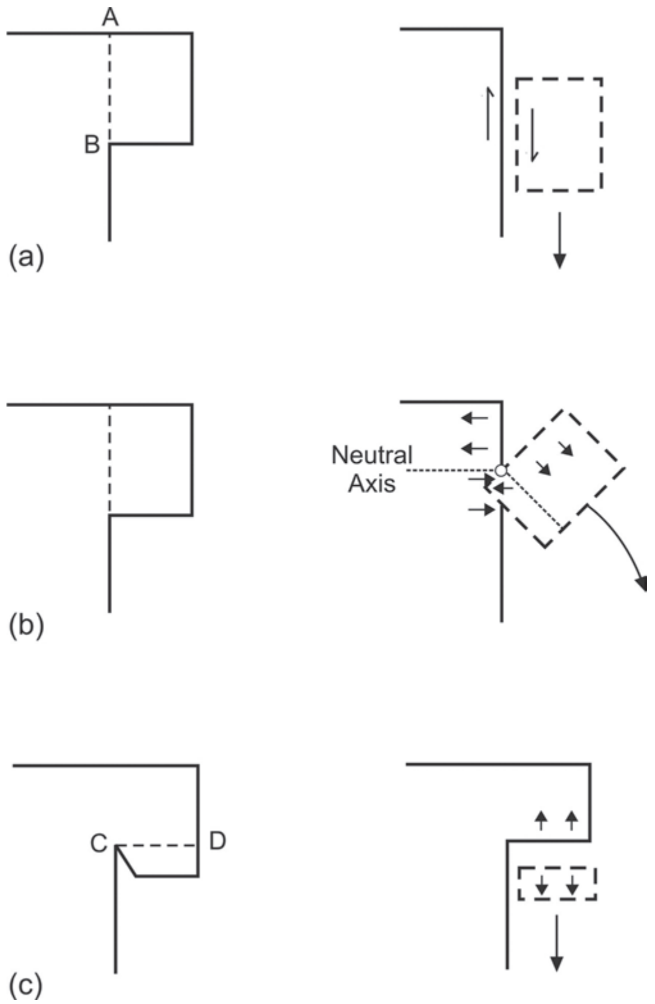

Fig. 7-18. Mechanisms of cantilever failure: (a) shear failure along AB; (b) beam failure about neutral axis; (c) tensile failure across (CD). (After Thorne and Tovey 1981. Copyright John Wiley & Sons Limited. Reproduced with permission.)

recedes by parallel retreat at a rate determined by the degree of fluvial activity at the base.

3. *Excess basal capacity*—Basal scour has excess capacity over the debris supply from bank failures. Basal lowering occurs, increasing bank angle and height, and therefore, decreasing stability with respect to mass failure. The rate of debris supply increases, favoring the second state.

The state of basal endpoint control is useful in explaining the medium- to long-term rates of riverbank retreat of advance. It also highlights the importance of considering the response of near-bank morphology to bank stabilization. The concept indicates that a reduction in debris supply that is due to bank stabilization may induce a state of excess basal capacity that generates very deep toe scour (Thorne et al. 1995). As pointed out by Maynord (1996), this additional scour must be properly accounted for in the design of the stabilization works if failure due to undermining is to be avoided.

Hagerty (1991) proposed that not all sustained bank retreat depends on the state of basal endpoint control. This proposal was based on the fact that piping is a widespread cause of sustained bank retreat along the Ohio River, which is apparently independent of the state of basal endpoint control. Even though the bank toe is stable, upper bank retreat has continued unabated for many years. However, closer inspection of the relevant bank profiles indicates that the reason that the toe has been stable is that, in this regulated river, the toe is well below pool level and is thus morphologically inactive. Piping in sand layers at about the elevation of the stranded low water plane has produced a bench that represents the toe of the morphologically active bank. At this elevation, bank retreat may still be considered to be covered by the concept of basal endpoint control, with the bank profile above pool elevation almost continually in a state of unimpeded removal, due to the ability of current and wave action to remove the fine debris supplied by piping. Creation of the bench and control of the profile thus depend on the piping process in supplying debris that can easily be removed by waves and currents that would not otherwise be able to erode intact bank material.

Hagerty et al.'s (1995) detailed treatment highlights the subtlety of interactions between fluvial and mechanical processes responsible for bank retreat, and it illustrates that great care must be taken in interpreting bank processes from bank form, especially in regulated rivers.

7.3.3.5 Vegetation Effects The role of vegetation in affecting bank erosion and width adjustment is complex and poorly understood. Although vegetation generally reduces soil erodibility, its impact on bank stability with respect to mass failure may be either positive or negative. Hence, depending on the geomorphic context and dominance of either fluvial processes or mass failure, vegetation may product either a net increase or a decrease in the rate of bankline shifting.

Vegetation can play an important role in limiting the effectiveness of bank erosion by detachment and entrainment of individual grains or aggregates of bank material. Compared to unvegetated banks, erosion of well-vegetated banks is reduced by one to two orders of magnitude (Carson and Kirkby 1972; Smith 1976; Kirkby and Morgan 1980). Gray and Leiser (1982) have reviewed the effects of herbaceous and, to a lesser extent, woody vegetation in reducing flow erosivity and bank erodibility and concluded that major effects include the following:

—Foliage and plant residues intercept and absorb rainfall energy and prevent soil compaction by raindrop impact.
—Root systems physically restrain soil particles.
—Near-bank velocities are retarded by increased roughness.
—Plant stems dampen turbulence to reduce instantaneous peak shear stresses.

—Roots and humus increase permeability and reduce excess pore water pressures.

—Depletion of soil moisture reduces water-logging.

Gray and Leiser (1982) also reviewed the ways that woody vegetation may affect the balance of forces promoting and resisting mass failure. Roots mechanically reinforce soil by transferring shear stresses in the soil to tensile stresses in the roots, which root strength is able to resist. However, this effect operates only to the rooting depth of the vegetation, and it does not reinforce potential failure planes that pass beneath the plant rootballs. Hence, root reinforcement is negated when bank height significantly exceeds rooting depth.

Soil moisture levels are decreased by interception on the canopy and evapotranspiration from the foliage, reducing the frequency of occurrence of the saturated conditions conducive to bank collapse. Anchored and embedded stems can act as buttress piles or arch abutments in a slope, counteracting downslope shear stresses and increasing bank stability. However, roots may also invade cracks and fissures in a soil or rock mass and thereby cause local instability by their wedging or prying action. The surcharge weight of vegetation may significantly increase motivating forces, causing destabilization of the bank, and wind loading of tall vegetation may exert an additional and potentially critical destabilizing moment on the bank.

These few examples illustrate the complexity of vegetation impacts on flow erosivity, soil erodibility, and mass stability. A recent scoping study on bank vegetation and bank protection reached the conclusion that vegetation may be either a positive or negative influence on bankline stability and retreat rate (Thorne et al. 1997). This may explain the apparently contradictory conclusions regarding the effect of bank vegetation on equilibrium channel width of, for example, Hey and Thorne (1986), who reported that stable channel width decreases as the density and stiffness of bank vegetation increase, and Murgertroyd and Ternan (1983), who found the opposite in a study of the effects of afforestation on channel form. Also, they may explain why the notable increases in the shear strength of root-permeated soils found in laboratory test soils by Waldron (1977) are not always replicated in strength measurements made in real riverbanks (Amarasinghe 1992).

As pointed out by Thorne and Osman (1988a), Darby and Thorne (1996a), and most recently, Thorne et al. (1997), a great deal of further research is necessary before vegetation effects can be properly understood and incorporated into the technical description of bank material characteristics under conditions representative of the range of environments encountered along natural streams and waterways.

7.3.3.6 Seepage Effects In addition to fluvial activity causing scour at the toe of the slope, grain-by-grain detachment, and mass wasting, Parola and Hagerty (1993) have identified a general class of failure mechanisms that is often very important to bank stability. This class of mechanisms is driven by seepage within the bank.

Pore-water movement within a bank is most vigorous during and following a high-flow event. As flood waters rise in a stream, the increased hydraulic head drives seepage into the bed and banks, resulting in groundwater recharge. As the flood stage recedes, hydraulic gradients reverse, driving seepage into the stream from the banks. The distribution of inflow, movement, and outflow through the bank is seldom uniform but is, in fact, strongly influenced by the layered stratigraphy that is characteristic of alluvial banks.

Alluvial banks consisting of sand, silt, and clay layers typically have hydraulic conductivity that is much greater in the horizontal direction than the vertical. Consequently, groundwater flow occurs principally by horizontal seepage into and out of sandy layers. During bank drainage, outflowing water may entrain and remove grains from a sand layer—a process termed piping by Hagerty (1991). Piping erosion leads to undermining of overlying, less pervious layers causing those layers to deflect and distort. The most common result of this undermining is the formation of cracks in the undermined layer, where the soil is unable to support the tensile stresses created by deflection (Parola and Hagerty 1993). Mass wasting then occurs as cracking reduces the operational strength of the bank.

Another type of bank failure associated with strong seepage is gully development. Although gully development is usually regarded as resulting from surface erosion, subsurface erosion by piping may lead to subsequent collapse of the pipes to form gullies along streambanks (Harvey et al. 1985). This mode of gully formation is particularly likely in loess deposits.

Bank weakening and erosion by seepage are often overlooked by river engineers. Failure to identify subsurface piping erosion can lead to misclassification of the erosion problem and subsequent problems with bank stabilization works that are adequate to armor the bank against fluvial attack but that are likely to fail due to internal erosion driven by piping.

7.3.3.7 Bank Advance Bank advance occurs through sediment deposition, a process that tends to narrow the channel. Bank advance occurs in a variety of different geomorphic settings (Figs. 7-1 and 7-2), and sediment may be deposited from bed load, suspended bed-material load, wash load, or a combination of all three transport processes. Despite this diversity, however, processes of bank advance have a common result: new floodplain deposits are created as the bank advances. This suggests that bank advance should be more broadly viewed as one of several processes that create new floodplains.

In a meandering stream, bank advance may occur on point bars on the insides of the channel bends (Figs. 7-1b and 7-2b) (Sundborg 1956; Leopold and Wolman 1957; Leopold et al. 1964; Jackson 1976a, 1976b) or on concave bank benches on the outsides of channel bends. Point bars are initiated by the creation of a point bar platform deposited primarily from bed load transport (Nanson 1980). The platform is the base on which develops a scroll bar of fine traction and suspended load. As the scroll bar grows, vegetation may be established, further enhancing deposition. When the scroll bar is fully developed, approximately half of the new floodplain sediment is deposited from suspension, and the other half from bed load (Nanson 1980).

Bank advance and channel narrowing can also occur by deposition of berms or benches at the margins of the channel (Fig. 7-2a). Pizzuto (1994) and Moody et al. (1999) describe the formation of new floodplain "benches" along the Powder River in southeastern Montana (Fig. 7-19). These new floodplains were formed primarily by deposition from suspension in a widened channel created by a 25 to 50 year flood in 1978. Schumm et al. (1984) and Simon (1989) described the formation of "berms" as part of the evolution of incised channels. These authors note that as incised channels recover, the bed aggrades and berms develop on the channel margins. Harvey and Watson (1988) propose that the berms are formed from dunes left as remnant bed forms following high flows. The remnant dunes are then draped with fine-grained silts and clays from the suspended load as the flow diminishes, stabilizing the deposits. Repetition of this process eventually produces a stable berm, permanently advancing the bankline.

Bank advance may also occur as sediment is draped onto riverbanks from suspension. Taylor and Woodyer (1978) describe sand-mud couplets that increase in thickness and grain size with depth that are formed by deposition of suspended sediment on riverbanks. Taylor and Woodyer (1978) note that the bank advance process may be accelerated through sediment trapping in pioneer vegetation.

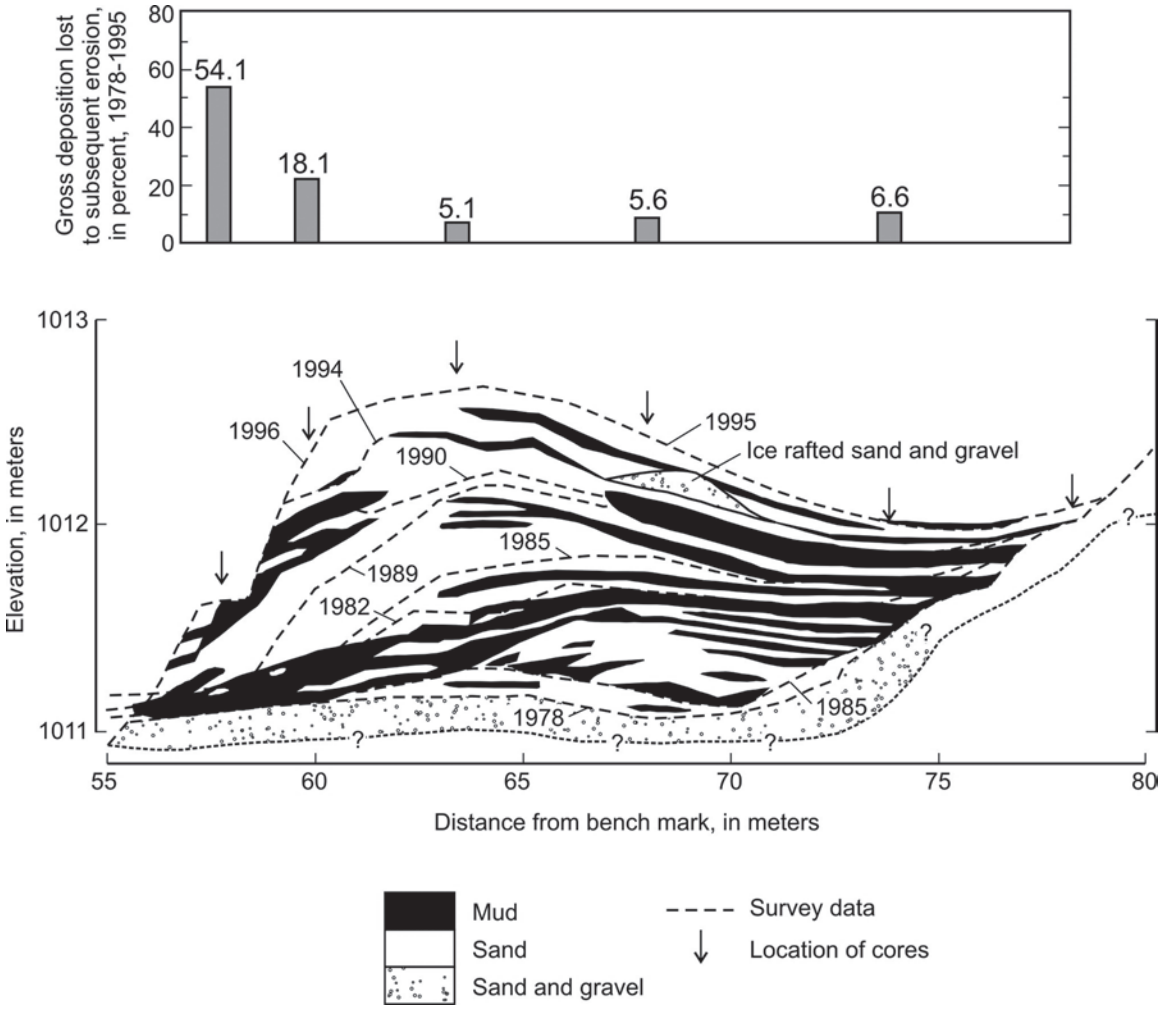

Fig. 7-19. Stratigraphic cross section of the flood plain at Section PR120 across Powder River near Moorhead, Montana, showing the history of bank advance from 1978 to 1996. The mud is shown as solid black areas and the sand as white areas. Some of the annual and biennial surveyed surfaces are labeled and the arrows indicate the locations of cores. Complete topographic cross sections of this site in 1978 and 1996 are presented in Fig. 7-3 (uppermost sections).

7.4 METHODS FOR EVALUATING BANK EROSION AND WIDTH ADJUSTMENT

7.4.1 Introduction

In this section, methods for evaluating bank erosion and width adjustment are described. The first method involves the development of qualitative conceptual models. The remaining methods are quantitative, involving either empirical equations or equations developed theoretically by specifying quantitative models of selected physical processes.

It is important to recognize that none of these methods apply to all rivers, and that all of the methods greatly simplify field conditions. Furthermore, the variety of different approaches available in the literature suggests that scientists who study rivers rarely agree on the best method for predicting the extent of bank erosion or deposition. As a result, the scientific knowledge required to solve practical engineering problems of bank erosion and width adjustment may not always be available. Each problem, then, will require careful study before an engineering solution can be proposed.

7.4.2 Conceptual Models of Channel Evolution and Processes

For a problem to be solved, it must be clearly defined. For problems of bank erosion and width adjustment, this implies understanding the processes that are acting at a particular site and their temporal and spatial context. This understanding may be developed as a "conceptual model," which could be summarized as a series of diagrams, or as a verbal description of how bank erosion and width adjustment occur at a particular site. In a very few selected cases, the conceptual model could actually be quantified and summarized by one or more equations.

The first part of this chapter summarized processes that control bank erosion and width adjustment, and some conceptual models of channel evolution were presented in Figs. 7-1 and 7-2. Here, a detailed example of a conceptual model of incised channel evolution is presented.

7.4.2.1 A Conceptual Model for Incised Channel Evolution Although applicable only to incised channels, the six-stage conceptual channel evolution model of Harvey and Watson (1986) has been of value in developing an understanding of watershed and channel dynamics and in characterizing whether or not a reach is stable (Fig. 7-20). The model was originally based on observations of the channel evolution of Oaklimiter Creek, a tributary of Tippah River in northern Mississippi (Schumm et al. 1984). The Oaklimiter sequence describes the systematic response of a channel to base level lowering and encompasses conditions that range from disequilibrium (Type I) to a new state of dynamic equilibrium (Type VI). It should be recognized that these categories are only conceptual and variation may be encountered in the field. Similar conceptual models have been proposed by Thorne and Osman (1988a; 1988b) and Simon and Hupp (1992).

Type I reaches are characterized by sediment transport capacity that exceeds sediment supply, bank height that is less than the critical bank height, a U-shaped cross section, and small precursor knickpoints in the bed of the channel (provided that the bed material is sufficiently cohesive and little or no bed material is deposited). Width/depth ratios at bank-full stage are highly variable.

Type II reaches are located immediately downstream of the primary knickpoint and are characterized by sediment transport capacity that exceeds sediment supply, bank height that is less than the critical bank height ($h < h_c$), little or no bed sediment deposits, a lower bed slope than the Type I reach, and a lower width/depth ratio than the Type I reach because the depth has increased, but the banks are not yet unstable.

Type III reaches are located downstream of Type II reaches and are characterized by sediment transport capacity that is highly variable with respect to the sediment supply, bank height that is greater than the critical bank height ($h > h_c$), bank erosion that is due primarily to slab failure (Bradford and Piest 1980), bank loss rates that are at a maximum, bed sediment accumulation that is generally <0.6 m but can be greater locally due to local erosion sources, and channel depth that is somewhat less than in Type II reaches.

Type IV reaches are downstream of Type III reaches and are characterized by sediment supply that exceeds sediment transport capacity, resulting in aggradation of the channel bed, bank height that approaches the critical bank height with a rate of bank failure lower than for Type III reaches, nearly trapezoidal cross-sectional shape, and width/depth ratio higher than the Type II reaches. The Type IV reach is aggradational and has a reduced bank height. Bank failure has increased channel width, and in some reaches, the beginnings of berms along the margins of an effective discharge channel can be observed. These berms are the beginning of natural levee deposits that form in aggraded reaches that were overwidened during earlier degradational phases.

Type V and VI reaches are located downstream of Type IV reaches and are characterized by dynamic balance between sediment transport capacity and sediment supply for the effective discharge channel, abank height that is less than the critical bank height for the existing bank angle, colonization by riparian vegetation, accumulated bed sediment depth that generally exceeds 1.0 m, width/depth ratio that exceeds the Type IV reach, and generally a compound channel formed within a new floodplain. The channel is in dynamic equilibrium. Bank angles have been reduced by accumulation of berm materials. Types V and VI reaches are distinguished primarily by the possible occurrence of overbank deposition in Type VI reaches.

The primary value of the sequence is that it enables the evolutionary state of the channel to be determined from field observations that record the characteristic channel forms

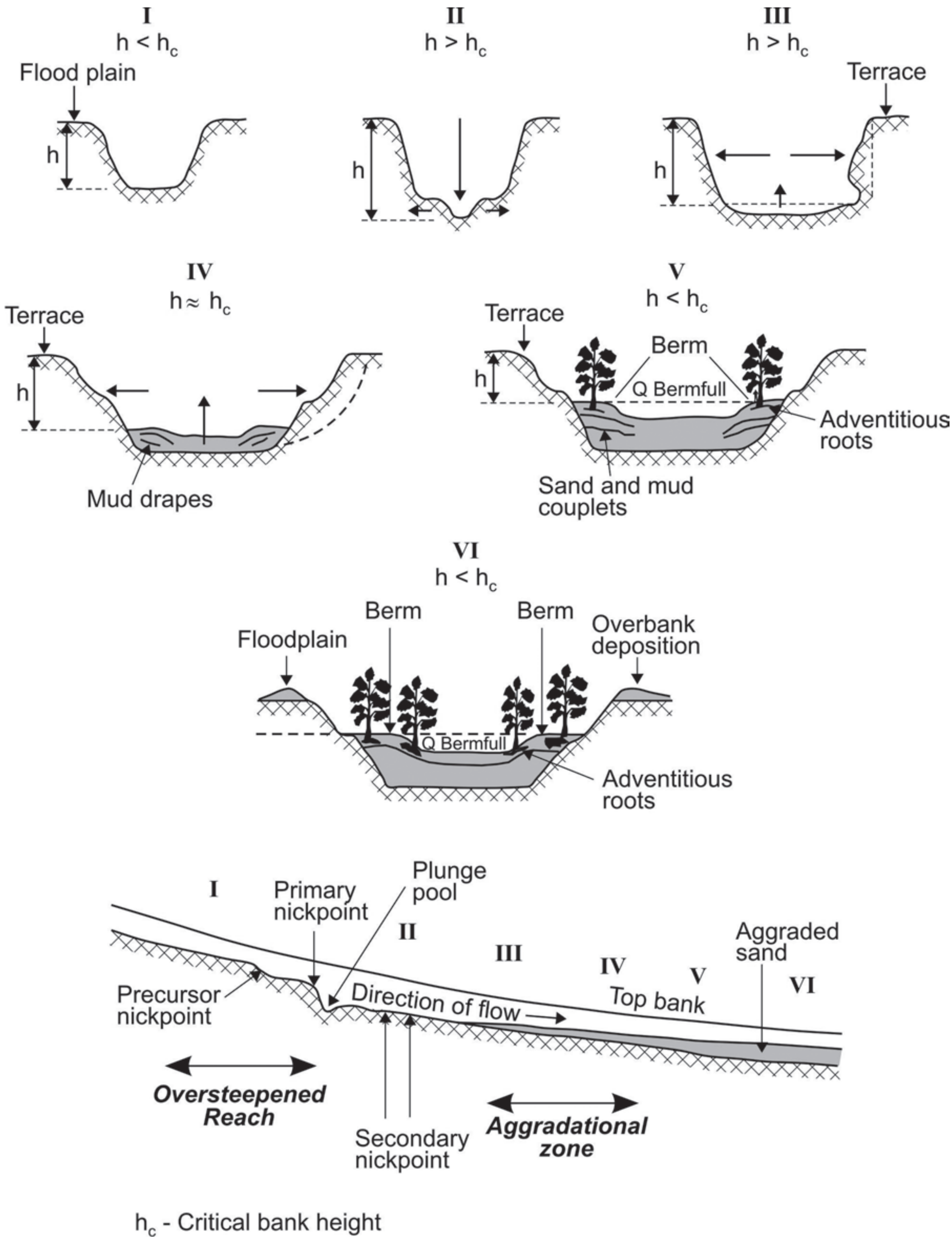

Fig. 7-20. Six-stage sequence of incised-channel evolution originally used to describe the evolution of Oaklimiter Creek (ASCE Task Committee on Hydraulics, Bank Mechanisms and Modeling of River Width Adjustment 1998b, with permission from ASCE).

associated with each stage of evolution. The morphometric characteristics of the channel reach types can also be correlated with hydraulic, geotechnical, and sediment transport parameters (Harvey and Watson 1986; Watson et al. 1988a; 1988b).

7.4.2.2 Channel Stability Diagram The channel evolution sequence of Schumm et al. (1984) and Harvey and Watson (1986) can be viewed in terms of two dimensionless stability numbers: (1) N_g is a measure of bank stability, and (2) N_h is a measure of fluvial stability. For a channel to be stable, fluvial stability and bank stability are both essential conditions. The desirable range for long-term channel stability is for N_g to be <1, and for N_h to be ~1, as shown in Fig. 7-21 (Watson et al. 1988a; 1988b). Quantifying the channel evolution sequence through the use of the dimensionless parameters N_g and N_h allows stability conditions along channel reaches to be ranked during rapid assessment and reconnaissance studies.

N_g is defined as any reasonable measure of bank stability expressed in terms of a factor of safety. The factor of safety represents the ratio between resisting and driving forces, such that banks are unstable for $N_g < 1$ and stable for $N_g > 1$. To allow flexibility, the operational definition of N_g is tailored according to the data available during a specific study (Watson et al. 1988a, 1988b). For example, in an initial reconnaissance of a site, the field investigator may note that banks over 3 m in height are generally unstable. In that circumstance, N_g could be the ratio of the bank height at a site divided by 3 m, which would yield $N_g \leq 1$ for stable bank heights. With better data and analyses, N_g could be the geotechnical bank safety factor computed with full knowledge of geotechnical properties, bank angle, and materials.

Similar flexibility is built into the operational definition of N_h, which was first defined as the ratio between the desired sediment supply and the actual sediment transport capacity (Watson et al. 1988a; 1988b). However, N_h could be any reasonable ratio of parameters that could be used as surrogates for sediment transport, such as the ratio of computed (or measured) sediment transport rates for the upstream supply reach and the stream reach of interest. In an initial reconnaissance, the thalweg slope of a stable channel may be surveyed and compared with the thalweg slope of the reach of interest. N_h would equal the ratio of the slope of the reach of interest divided by the stable slope. N_h is >1 for degradational reaches and is <1 for aggradational reaches.

The dimensionless stability numbers, N_g and N_h, can be related to the Oaklimiter sequence, as shown in Fig. 7-21. As the channel evolves from a state of disequilibrium to a state

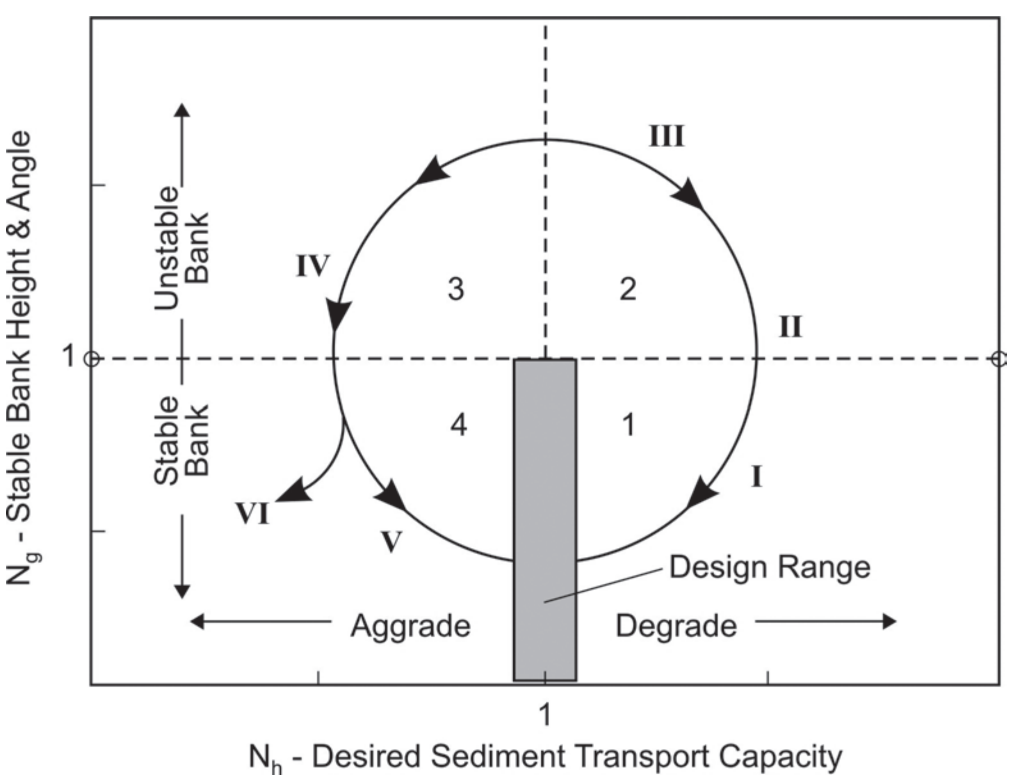

Fig. 7-21. Comparison of Oaklimiter Creek channel evolution sequence and channel stability parameters (ASCE Task Committee on Hydraulics, Bank Mechanisms and Modeling of River Width Adjustment 1998b, with permission from ASCE).

of dynamic equilibrium through the six reach types of the Oaklimiter sequence, the channel progresses through the four stability diagram quadrants in a counterclockwise direction. Rehabilitation of the channel should attempt to avoid as many of the quadrants as possible to reduce the amount of channel deepening and widening.

Each quadrant of the stability diagram is characterized by geotechnical and hydraulic stability number pairs, and stream reaches that plot in each quadrant have common characteristics with respect to stability, flood control, and measures that may be implemented to achieve a project goal.

In Quadrant 1 ($N_g < 1$, $N_h > 1$), the channel bed may be degrading or may be incipiently degradational; however, the channel banks are not geotechnically unstable. Bank erosion is occurring only locally, and bank stabilization measures, such as riprap or bioengineered stabilization, could be applied. However, local bank stabilization would not be successful if bed degradation continued and destabilized the stabilization measures; therefore, bed stabilization measures should be considered for long-term effectiveness of bank stabilization measures. If flood control is a project goal, almost any channelization or levee construction would increase N_h and shift the value to the right. Flow control using a reservoir can address flood control capacity, which may cause other changes in channel dynamics. The designer must be aware of the channel response to imposed conditions relating to the stability factors.

Quadrant 2 ($N_g > 1$, $N_h > 1$) streams are unstable. The channel bed is degrading and channel banks are geotechnically unstable. Grade control must be used to reduce bed slope, transport capacity and N_h. Bank stabilization measures will fail in this quadrant because the bed is continuing to degrade, which will destabilize the foundation of the bank stabilization. Both flood control and bank stability must be considered when determining the height to which grade control should be constructed. A series of grade control structures can reduce bank height sufficiently to stabilize the banks, but a combination of lower grade control and bank stabilization may meet flood control, ecological, and stability objectives. Emplaced habitat features are subject to failure caused by degradation of the bed, bank failure, and lowering of water-surface elevations.

Quadrant 3 ($N_g > 1$, $N_h > 1$) is characterized by gravity-driven bank failure but without continued bed degradation. Bank stabilization could be effective without grade control emplacement, but both measures should be considered. Flow control in these two quadrants could be beneficial. Emplaced habitat features may be inundated by channel aggradation or affected by adjacent bank failure.

Quadrant 4 ($N_g < 1$, $N_h < 1$) is characterized by general stability. Local bank stabilization measures will be effective. As N_h decreases in this quadrant, the potential for channel aggradation-related flood control problems or inundation of habitat features will increase.

7.4.3 Equilibrium Approaches

7.4.3.1 The Engineering Significance of Geomorphic Equilibrium In section 7.2, streams that are "graded" or "in regime" are defined as those whose morphology does not change "significantly" with time due to quasi-steady supplies of water and sediment. In section 7.4.3, quantitative models of graded streams are discussed. Before these models are presented, however, it is important to understand how graded streams are defined and how they may be recognized. Streams that are "graded" are not static, and they may change their morphologies during floods or other short-term perturbations. These changes may be large enough to have important engineering consequences.

A useful conceptual model of the temporal evolution of graded streams has been proposed by Schumm and Lichty (1965), who explained how streams can change progressively with time and yet still be considered "graded." To resolve this paradox, Schumm and Lichty (1965) defined three timescales for viewing river channels. Figure 7-22 presents these timescales using channel gradient to illustrate morphologic change, though any morphologic feature (e.g., width, depth) could be used. It is also important to recognize that the time axis of Fig. 7-22 is imprecise, and may be shorter or longer depending on the morphologic variable plotted on the y axis and the particular river system that is being investigated.

Over short periods, referred to by Schumm and Lichty (1965) as "steady time," the channel morphology is constant, because flows large enough to change the slope are not likely to occur. Over longer periods, defined as "graded time" by Schumm and Lichty (1965), the channel morphology oscillates about a temporally steady average value. This is the classic behavior associated with graded streams. It is important to note that the morphology is not constant, but changes tend to average to zero when viewed over sufficient time. Over the longest time scale, defined by Schumm and Lichty (1965) as "cyclic time," a drift in the morphology may be observed.

Schumm and Lichty's (1965) conceptual model suggests that field observations over graded timescales could be used to determine if a stream is graded or not. However, because decades of measurements are typically required, such data are rarely available. As an alternative, geomorphologists and engineers often use regression equations to determine if a river's morphology can be explained by variables such as discharge or bank sediment type that do not involve time (the power-law approach presented in section 7.4.3.2 is a typical example) (Fig. 7-23). If these regression equations explain a significant amount of the variance in the observations, streams are often considered to be graded or in a quasi-equilibrium state (Leopold and Maddock 1953; Wolman 1955; Leopold et al. 1964).

Figure 7-22 implies, however, that significant morphologic variation may still occur even if stream morphology

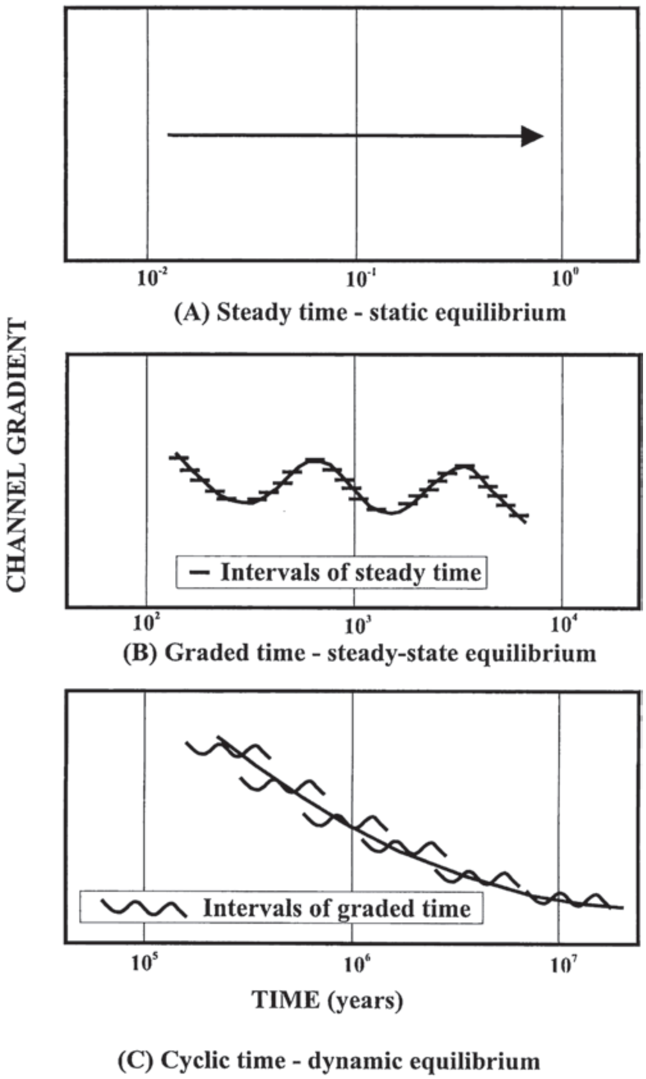

Fig. 7-22. Different time intervals and associated equilibrium in geomorphic analyses. (A) Steady time (static equilibrium). No change in channel gradient over short periods. (B) Graded time (steady-state equilibrium). Constant average channel gradient with periodic fluctuations above and below the average condition. Measurements made during intervals of steady time within the graded time period may show no change in channel gradient. (C) Cyclic time (dynamic equilibrium). Gradual lowering of the average channel gradient over long time intervals. Intervals of graded time and steady-state equilibrium exist within the cyclic time scale (Ritter 1978).

can be explained by discharge or any other variable that does not include time. Data from the Powder River watershed (Figs. 7-3, 7-19, and 7-22) present a useful example. Figure 7-23 demonstrates that the width is highly correlated with mean annual discharge in the Powder River basin. However, observations of the channel morphology of the Powder River between Moorhead and Broadus, Montana, from 1975 to 1998 demonstrate that a 25 to 50 year flood increased the channel area by an average of 62% (Pizzuto 1994). Subsequent deposition caused a substantial decrease in channel area (Moody et al. 1999). These oscillations are not large enough to invalidate Fig. 7-23, and thus the channel is in some sense graded or in regime. However, the observed temporal changes in channel form could greatly influence engineering structures, suggesting that equilibrium approaches should be used with considerable caution as design tools.

7.4.3.2 Regime Theory: A Power-law Approach The initial approach to predicting equilibrium channel form was based on empirical methods developed from field observations and regression equations and applied to the design of stable canals. The first regime relation was proposed by Kennedy (1885) over a century ago. Several regime relations followed, and these have been repeatedly refined and enhanced. The regime equations attributed to Lindley (1919), Lacey (1920), Simons and Albertson (1963), and Blench (1969) are probably the most widely known.

Although regime equations are extensively used by engineers, with successful outcomes, they suffer several shortcomings, including the facts that they are not dimensionally homogeneous and that their validity is limited to the basins and data from which they were derived. More sophisticated regime relations have been proposed by employing computers to obtain regression equations based on much larger data sets (Brownlie 1981a; 1981b). Most work, including that previously cited, pertains to sand-bed streams, but equivalent regime relations have also been proposed for gravel-bed streams; reviews are presented by Bray (1982) and Hey and Thorne (1986). More recently, semianalytical work by Julien and Wargadalam (1995) has attempted to refine the regime approach within a framework based on the governing principles of open channel flow.

Geomorphologists have used data from natural steams and laboratory flumes to develop power-law hydraulic geometry relations between channel top width, average depth, average velocity, and bank-full discharge (Leopold and Maddock 1953); Fig. 7-23 is an example. The exponents in these relations exhibit surprising universality, particularly the one for channel width, which has been found to be ~0.5 for rivers with widely varying flow regimes and sediment characteristics located in different physiographic regions of the world. However, the regression coefficients are found to vary significantly from one locality to another, which renders power-law hydraulic geometry relations inappropriate as tools for general design purposes.

The relevant empirical formulas developed in these approaches, as well as others that are not mentioned here due to lack of space, are described in detail in standard river mechanics books (e.g., Garde and Ranga-Raju (1977) and Simons and Senturk (1992)) and earlier review papers (e.g., Ferguson (1986)).

7.4.3.3 Extremal Hypothesis Approach The last two decades have seen the proliferation of approaches that employ an extremal hypothesis as part of their formulation for predicting channel morphology. Equations for sediment

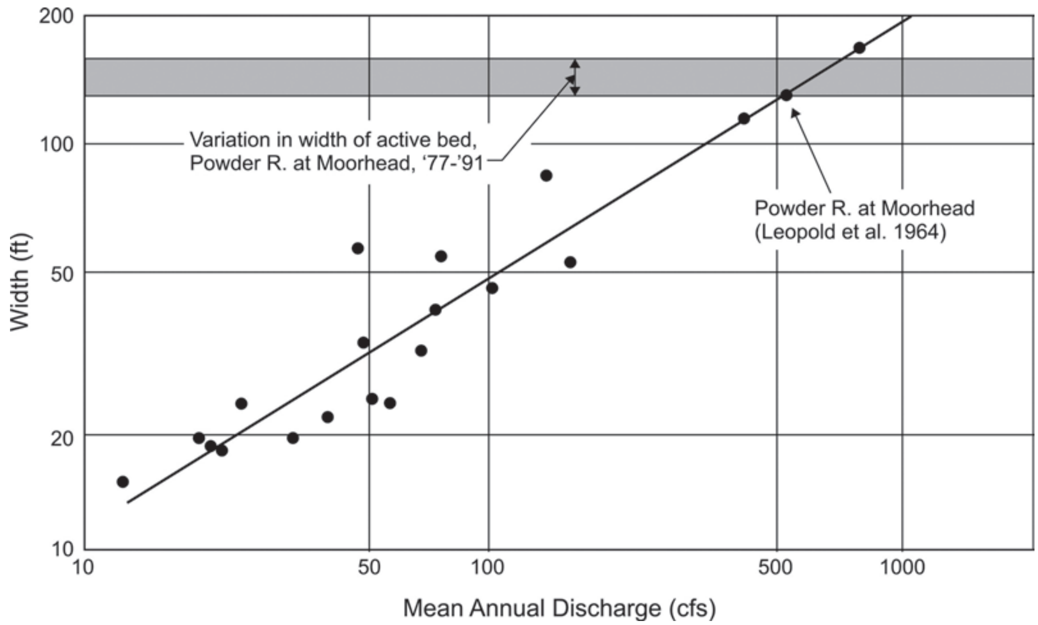

Fig. 7-23. Relationship between mean annual discharge and width for the Powder River watershed in Wyoming and Montana (Leopold et al. 1964). The shaded region represents the variation in width documented by cross sections from 1977 to 1991 (Pizzuto 1994).

transport and alluvial friction are combined with a third relationship to determine channel width and to predict regime or equilibrium conditions. This third relationship has frequently been expressed in terms of the maximization or minimization of a parameter, such as stream power, energy dissipation rate, or sediment concentration. Extremal hypotheses have been introduced by Chang (1980); Yang et al. (1981); Yang and Song (1986); Bettess and White (1987); Chang (1988a); Yang (1992); Chiu and Abidin (1995); and Millar and Quick (1997), among others. An extremal hypothesis, based on stream power, also forms the basis of the analytical approach of White et al. (1981) to the river regime and the Wallingford tables for the design of stable channels (White et al. 1981).

The theoretical justification for such hypotheses and the relationships between them are still not entirely clear. Also, when extremal hypotheses are applied, a clear understanding is required of the physical constraints presented by geological or other boundary conditions on the evolution of a channel toward a form that minimizes its rate of energy expenditure. The predictions based on such methods, however, provide global, if not exacting, agreement with a wide range of observations.

7.4.3.4 Tractive Force Methods Tractive force methods employ the basic laws of mechanics to obtain expressions that specify the geometry of stable channel cross sections. This approach was initiated in the late 1940s by the U.S. Bureau of Reclamation, and it resulted in the threshold channel theory (Glover and Florey 1951; Lane 1955). The theory is based on a fluid momentum balance that is used to obtain the local boundary shear stress and a stability criterion for the sediment particles that make up the channel perimeter. It assumes that the channel is straight, that secondary flow is negligible, and that sediment is noncohesive and does not vary within the channel. Most importantly, the tractive force approach assumes that the channel morphology is adjusted so that sediment across the perimeter of the cross section is at the threshold of motion. Under these conditions, sediment is neither eroded nor deposited at any point on the cross section. When these assumptions are satisfied, a cosine profile is predicted for the stable cross section (Fig. 7-24).

A threshold channel does not allow for bed-load transport. Diplas (1990) and Parker (1979) showed that the Glover and Florey method cannot be extended to generate channels capable of transporting sediment while they maintain threshold banks. This result is contrary to numerous observations from natural streams and flume experiments, which attest to the possible coexistence of a mobile bed and

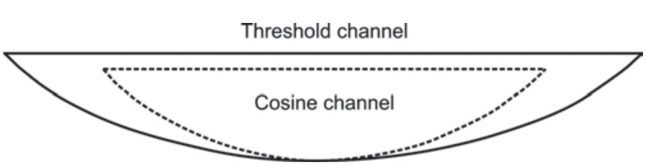

Fig. 7-24. Comparison between threshold channel profile obtained from momentum-diffusion model and cosine profile (adapted from Vigilar and Diplas 1997, with permission from ASCE).

stable banks. Parker (1978b) overcame this inconsistency by employing the momentum balance of Lundgren and Jonsson (1964), which accounts for lateral turbulent diffusion of downstream momentum. Due to the complexity of the corresponding differential equation, his solution was limited to the flatbed region, whereas the bank geometry was solved as a first-order solution, yielding a cosine profile. Thus, Parker (1978b) was able to reconcile the existence of sediment movement within a stable channel. Ikeda et al. (1988) extended the results of Parker (1978b) to include sediment heterogeneity, and Ikeda and Izumi (1990) considered the effect of bank vegetation, whereas Parker (1978a) and Ikeda and Izumi (1991) examined the influence of suspended load on channel dimensions.

The tractive force model, in the form proposed by Parker, was recently refined by Diplas and Vigilar (1992). The main differences from the previous work were that the governing equations were solved numerically and the bank geometry was not assumed, but became part of the solution. As a result, the threshold channel shape turned out to be different from a cosine curve, having a greater top width and center depth (Diplas and Vigilar 1992; Vigilar and Diplas 1994; 1997; 1998). For the example shown in Fig. 7-24, the longitudinal slope is 0.00081, the value of the critical Shields parameter is 0.056, and the sediment is semiangular, with $D_{50} = 45$ mm and $D_{90} = 75$ mm. The cross-sectional area of the threshold channel and the water discharge that it conveys are more than twice those for a cosine channel under the same conditions. This is attributed to the role of momentum diffusion, which results in decreased stresses in the central region of the channel (thus allowing a deeper flow) and increased stresses in the upper bank regions (forcing banks to assume gentler slopes to prevent erosion). Knowledge of the local topography, the sediment size and shape, and the value of the critical Shields parameter uniquely determine the dimensions of a threshold channel and its discharge.

In the case of a channel with stable banks and a mobile bed, the bank profiles change with the width of the flatbed section (Fig. 7-25) (Vigilar and Diplas 1997; 1998). However, beyond an aspect ratio of 12, which is typical of natural streams, the bank profile remains constant, and the channel is termed "wide." The stable channel dimensions and bed load transport capacity can be determined for known local bed slope, sediment size and shape, value of the critical Shields parameter, and water discharge. If the bed load discharge is specified, the channel bed slope becomes part of the solution.

It is important to recognize, however, that tractive force methods do not accurately represent channels where sediment is deposited on the banks or eroded from the banks. For these conditions (which represent the majority of natural channels), only a few preliminary mechanistic models of equilibrium channels are available (Parker 1978b; Pizzuto 1984).

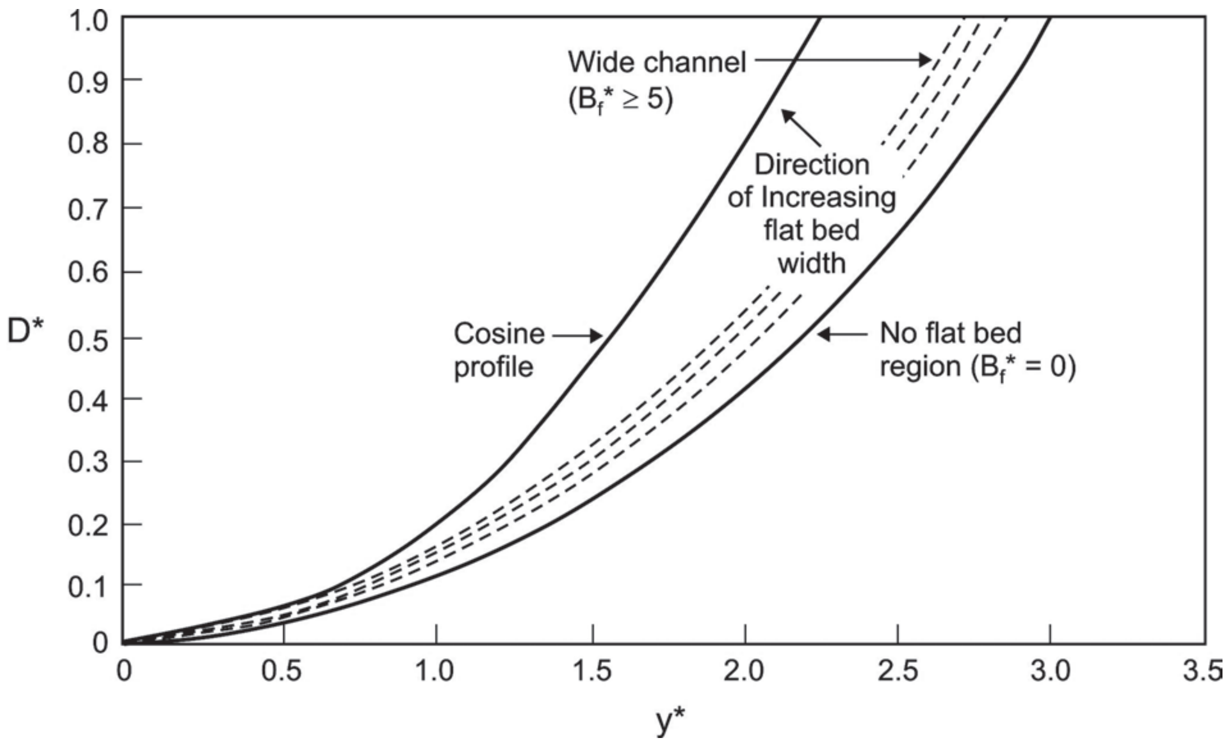

Fig. 7-25. Bank profiles generated by momentum-diffusion model for different values of flatbed width of channel (ASCE Task Committee on Hydraulics, Bank Mechanisms and Modeling of River Width Adjustment 1998a, with permission from ASCE).

7.4.4 Empirical Methods Based on Field Observations

In many cases, an existing theory or model may not be available to predict rates of bank erosion, deposition, or width adjustment at a particular site. However, field observations may be used to develop empirical equations that may be used for prediction. Although such empirical equations typically have little or no generality, they may provide useful short-term predictions if future conditions are similar to those used to develop the empirical equations.

Rates of bank retreat are predicted by two different methods. One involves using maps, aerial photographs, or historical surveys to determine past rates of bank erosion. For prediction, these rates may simply be extrapolated into the future to provide an estimate of the future position of the eroding bank. Another method involves developing an empirical equation that includes one or more physical parameters that control rates of bank erosion. Figure 7-26 illustrates a correlation between rates of bank retreat and velocity near the bank (Pizzuto and Meckelnburg 1989) based on field observations of bank retreat from the Brandywine Creek in southeastern Pennsylvania (the values of the near-bank velocity were obtained using the flow model of Ikeda et al. 1981). The regression equation represented by the best-fit line, although only valid at the study site, could provide accurate predictions of the future positions of the retreating bank.

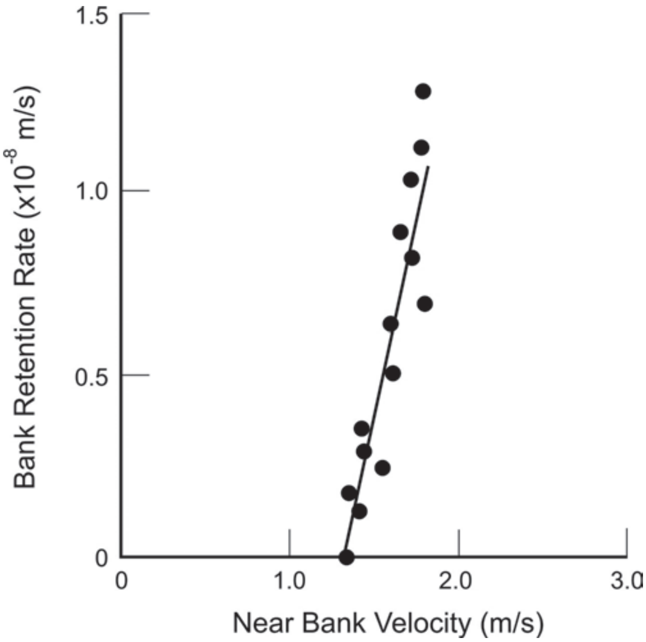

Fig. 7-26. Relationship between rate of bank retreat and near-bank velocity for a meander bend of the Brandywine Creek in southeastern Pennsylvania (after Pizzuto and Meckelnburg 1989).

7.4.5 Numerical Width Adjustment Models

Fixed-width numerical morphological models are now commonly used in engineering practice to obtain predictions of the extent of scour and fill of a bed in response to changes in the independent variables of flow and sediment discharge. The status of fixed-width numerical morphological modeling has been reviewed by Fan (1988).

Fixed-width numerical models are limited in applicability to cases where width adjustments in the prototype channel are not significant. To address this deficiency, a number of attempts to account for time-dependent width adjustments in numerical morphological models have been made. It should be recognized at the outset that each of these models is in some way limited. Twelve numerical width adjustment models based on various approaches to representing the governing processes of flow, flow resistance, sediment transport, and bank mechanics are reviewed in Table 7-1. A promising numerical width adjustment model described by Nagata et al. (2000) was published after the following analysis was completed.

7.4.5.1 Fluvial Hydraulics and Hydrodynamics A number of approaches have been used to estimate the flow field in the computational domains of the various numerical models (Table 7-2). Despite their undoubted significance, over-bank flows are excluded from all of these approaches. The approaches are based on simplifications of the governing flow momentum and continuity equations and are therefore limited in validity to the particular conditions defined in making the simplifying approximations. Additionally, each approach requires an estimate of the friction factor, which is usually either specified by the user or calculated using an empirically calibrated roughness equation. The friction factor estimate may or may not be allowed to vary through space and time (Table 7-2). Each of the flow resistance equations in Table 7-2 is, strictly speaking, valid only for the physical conditions corresponding to the data originally used to derive it. None of the reviewed models account for the effects of vegetation on flow.

The water-routing submodel of the FLUVIAL-12 model (Chang 1988a; 1988b) computes the water-surface elevation and energy gradient at each cross section by solving 1D versions of the flow momentum and continuity equations. For steady flow, the standard step method is employed, whereas solution procedures suggested by Fread (1971; 1974) and Chow (1973) are followed for unsteady-flow routing. A correction for flow resistance due to secondary flow effects in curved channels is made (Chang 1988a). Osman (1985), Alonso and Combs (1986), and Borah and Bordoloi (1989) developed similar approaches to flow routing in their morphological models. Unlike FLUVIAL-12, these methods also neglect secondary flows and are applicable to steady flows only, though unsteady flows are approximated through the use of a stepped hydrograph with discharge constant in any one time step. The 1D flow-routing methods provide estimates of cross-sectionally averaged flow parameters

Table 7-1 List of Reviewed Models

Model (1)	Category (2)	Additional references (3)
Darby and Thorne (1996a)	Geofluvial, cohesive bank	Darby and Thorne (1996b); Darby et al. (1996)
CCHEBank (Li and Wang 1993)	Geofluvial, noncohesive bank	Li and Wang (1994a, b)
Kovacs and Parker (1994)	Geofluvial, noncohesive bank	Kovacs (1992)
Wiele (1992)	Geofluvial, noncohesive bank	Wiele and Paola (1989)
RIPA (Mosselman 1992)	Geofluvial, cohesive bank	Struiksma et al. (1985); Olesen (1987); Mosselman (1991); Talmon (1992)
Simon et al. (1991)	Geofluvial, cohesive bank	
Pizzuto (1990)	Geofluvial, noncohesive bank	
STREAM2 (Borah and Bordoloi 1989)	Geofluvial, cohesive bank	Borah and Dashputre (1994)
GSTARS (Yang et al. 1988)	Extremal hypothesis	
FLUVIAL-12 (Chang 1988b)	Extremal hypothesis	
Alonso and Combs (1986)	Geofluvial, cohesive bank	
WIDTH (Osman 1985)	Geofluvial, cohesive bank	

Table 7-2 Features of Flow Routing Submodels of Reviewed Models

Model (1)	Dimension (2)	Discharge variation over time (3)	Secondary flow (4)	Lateral shear (5)	Friction factor (6)	Flow resistance formulas[a] (7)
Darby-Thorne	Quasi2D[b]	Stepped hydrograph	No	Yes	Time and space variable	Strickler
CCCHEBank	3D	Unsteady flow	Yes	Yes	Constant	Keulegan
Kovacs-Parker	2D	Steady flow	No	Yes	Constant	Keulegan
Wiele	2D	Steady flow	No	Yes	Constant	Keulegan
RIPA	2D	Stepped hydrograph	Yes	No	Constant	Specified
Simon et al.	Quasi2D[b]	Stepped hydrograph	No	No	Time and space variable	Strickler, Darcy, and Chezy
Pizzuto	2D	Steady flow	No	Yes	Constant	Einstein
STREAM2	1D	Stepped hydrograph	No	No	Constant	Specified
GSTARS	Quasi2D[b]	Stepped hydrograph	No	No	Time and space variable	Strickler, Darcy, and Chezy
Fluvial-12	1D	Unsteady flow	Yes	No	Time and space variable	Strickler and Brownlie
Alonso-Combs	1D	Stepped hydrograph	No	No	Constant	Specified
WIDTH	1D	Stepped hydrograph	No	No	Time and space variable	Strickler

Note: Strickler = Strickler (1923); Keulegan = Keulegan (1938); Einstein = Einstein (1950); Brownlie = Brownlie (1983).
[a]None of these formulas account for the effects of bed forms.
[b]Quasi2D models refer to those models that simulate lateral variation of bed topography through use of multiple 1D stream tubes.

and are unable to resolve near-bank boundary shear stresses sufficiently accurately to estimate fluvial erosion of bank materials.

Various attempts to account for the lateral variation of flow fields in natural channels have been made. Both the GSTARS (Molinas and Yang 1986; Yang et al. 1988) and modified BRI-STARS (Simon et al. 1991) models employ quasi-two-dimensional (quasi-2D) flow-routing procedures based on the stream tube approach. Stream-tube-based approaches are limited because they normally exclude lateral momentum exchange processes due to secondary flows and lateral shear induced by bank friction, and they are limited to steady flows. These approaches are also expected, therefore, to have low predictive ability for near-bank-zone applications.

Darby and Thorne (1996c) adopted a quasi-2D method in which lateral distributions of flow velocity and boundary shear stress were estimated at each cross section via numerical solution of a version of the flow momentum and continuity equations in which lateral shear stress terms were retained (Wark et al. 1990). The method is valid for steady, uniform flow but was applied in conjunction with a gradually varied 1D flow-routing model solved using the standard step method (Chow 1973) to estimate longitudinal variations in water-surface elevations and energy gradients at each of the modeled sections. The flow submodel employed by Darby and Thorne provides an improved representation of the flow field compared to 1D and stream-tube flow-routing methods. However, the validity of this method is limited because secondary flows are neglected (the approach was intended for straight channels only).

The 2D depth-averaged flow submodel of RIPA (Mosselman 1992) is based on differential equations expressing the conservation of mass and momentum of water. This model includes a correction for the deformation of the flow field due to secondary flow, but the influence of lateral shear on near-bank flows is neglected. Wiele (1992) included both terms in his flow submodel.

The flow submodels employed by Pizzuto (1990) and Kovacs and Parker (1994) model the distribution of fluid-induced boundary shear stress on gently curved riverbanks in straight channels. The methods are valid for steady, uniform flows; they include lateral shear stress terms but ignore momentum transfer by secondary currents. Both methods are only valid where bed and bank curvature is small.

In the CCHEBank model (Li and Wang 1993; 1994a,b), the flow field is computed using CCHE3D (Wang and Hu 1990), an advanced 3D hydrodynamic model, which can simulate unsteady free surface turbulent 3D flow fields in open channels. Secondary flows and lateral shear stress terms are also included in the model. This 3D flow model has the fewest simplifying approximations of the models reviewed here and, therefore, has the greatest potential for successfully modeling near-bank flows. However, simplifying assumptions are still required in the eddy viscosity closure model, and the flow model is also subject to the limitations of the method used to specify the friction factor.

7.4.5.2 Sediment Transport and Continuity Methods of sediment routing in each of the 12 models reviewed here are summarized in Table 7-3. Sediment routing is accomplished by relating sediment transport at each computational node to the flow field and physical properties of the bed material there. An empirically calibrated sediment transport equation is used to estimate the sediment flux field. Some models offer users the choice of specifying a particular equation from a menu. Spatial differences in sediment flux so estimated determine the evolution of the bed topography through solution of the sediment continuity equation.

The models are uniformly limited in validity to conditions corresponding to those originally used to calibrate the available sediment transport equation. Even within these constraints, and optimistically assuming that the flow field has been predicted accurately, sediment flux predictions are prone to order-of-magnitude errors (Gomez and Church 1989; Yang and Wan 1991).

A particular limitation of width adjustment modeling applications is that most sediment transport equations are valid only for bed surfaces inclined at low angles ($\sin \theta < 0.1$), though in noncohesive channels such equations are applied in bank regions that are often inclined at angles close to the angle of repose (typically 35°). The vertical bed load transport equation developed by Kovacs and Parker (1994), and included in their bank erosion model, is the only model reviewed here that accounts for the effects of large bed slopes ($\sin \theta > 0.1$).

In some models, sediment sorting is handled through the use of mixed (active) layer theory. Accurate prediction of the bed-material grain-size distribution throughout the model simulation is important if the flow resistance and sediment transport submodels are to have any chance of continuing to predict the flow and sediment-transport fields with acceptable accuracy throughout the simulation. Research has indicated that bed-material grain-size adjustments in unstable rivers are as important as adjustments in gradient, depth, or width (Hoey and Ferguson 1994). Ability to account for the transport of heterogeneous sediment mixtures is particularly important in the context of width-adjustment models, because the grain-size distribution of eroded bank materials is often quite different from that of the original bed material. Summary information regarding the mixed layer scheme employed in each of the models is provided in Table 7-3.

The wide ranges of potential grain sizes frequently involved in the width-adjustment process also dictate that both bed load and suspended-sediment fluxes must be accounted for in width-adjustment modeling. Table 7-3 summarizes the capabilities of the various sediment-routing submodels with respect to this issue.

Table 7-3 Features of Sediment-Routing Submodels of Reviewed Models

Model (1)	Routing methods (2)	Streamwise flux difference (3)	Transverse flux difference (4)	Bed load (5)	Suspended load (6)	Transport equations (7)	Sorting (8)	Bed material (9)
Darby-Thorne	Quasi2D	Yes	Yes	Yes	Yes	Engelund and Hansen (1967)	Yes	Sand
CCCHEBank	2D	Yes	Yes	Yes	No	Meyer-Peter and Muller (1948)	No	Gravel
Kovacs-Parker	2D	No	Yes	Yes	No	Kovacs and Parker (1994)	No	Gravel
Wiele	2D	No	Yes	Yes	No	Parker (1979) and Meyer-Peter and Muller (1948)	No	Sand and gravel
RIPA	2D	Yes	Yes	Yes	No	Engelund and Hansen (1967) and Meyer-Peter and Muller (1948)	No	Sand and gravel
Simon et al.	Quasi2D	Yes	No	Yes	Yes	Yang (1973; 1984); Ackers and White (1973); and Engelund and Hansen (1967)	Yes	Sand and gravel
Pizzuto	2D	No	Yes	Yes	No	Parker (1983)	No	Sand
STREAM2	1D	Yes	No	Yes	Yes	Yang (1973); Graf (1971); and Meyer-Peter and Muller (1948)	Yes	Sand and gravel
GSTARS	Quasi2D	Yes	No	Yes	Yes	Yang (1973; 1984); Ackers and White (1973); and Engelund and Hansen (1967)	Yes	Sand and gravel
FLUVIA L-12	1D	Yes	No	Yes	Yes	Yang (1973); Parker et al. (1982); Ackers and White (1973); Engelund and Hansen (1967); and Graf (1971)	Yes	Sand and gravel
Alonso-Combs	1D	Yes	No	Yes	Yes	Alonso et al. (1981)	Yes	Sand and gravel
WIDTH	1D	Yes	No	Yes	Yes	Engelund and Hansen (1967)	No	Sand

In each model, changes in bed elevation resulting from spatial differences in the predicted sediment flux field are computed through numerical solution of the sediment continuity equation. The sediment continuity equation is usually simplified by neglecting either the longitudinal or transverse sediment-flux difference terms (Table 7-3). These simplifications limit the validity of these models; it can be shown that both streamwise and transverse sediment-flux differences are, in fact, equally significant in controlling near-bank bed topography changes (Darby and Thorne 1992).

One-dimensional sediment-routing procedures (Table 7-3) neglect transverse sediment fluxes and require various assumptions concerning the distribution of predicted changes in bed elevation across the channel cross section. In this context, the most important areas are the near-bank zones, because predicted changes in bed elevation directly influence the stability of the banks and, hence, the predicted widening or narrowing rates. For example, Osman (1985) assumed that the bed level change is distributed evenly over the entire cross section. In contrast, Alonso and Combs (1986) and Borah and Bordoloi (1989) utilized various assumptions to distribute the scour and fill more realistically across the section. Alonso and Combs (1986) accounted for nonuniform sediment deposition across the channel cross section using relations describing the lateral flux of suspended sediments proposed by Parker (1978a). No method of accounting for nonuniform distribution of erosion is described.

To address this issue, quasi-2D approaches have been proposed (Table 7-3). Simon et al. (1991) proposed a quasi-2D sediment-routing model based on the stream-tubes concepts employed in the GSTARS model. Darby and Thorne (1996c) divided each modeled cross section into three (one central and two near-bank) segments. This was done to provide more refined estimates of bed topography evolution in the near-bank zones. Each near-bank segment extended a distance of two bank heights from the base of the bank. In contrast to the quasi-2D approaches, fully 2D solutions of the sediment continuity equation (Table 7-3) provide higher definition, though not necessarily more accurate, estimates of bed topography changes in the near-bank zones.

7.4.5.3 Riverbank Mechanics A summary of methods of modeling bank mechanics in each of the reviewed models is provided in Table 7-4. None of these methods accounts for the impacts of riparian vegetation.

7.4.5.3.1 Retreat and Advance Processes Processes of bank retreat and advance may occur together or separately at different locations and times along the same reach of a river. Modeled rates of bank advance and retreat on both banks at a single section determine the rate of width adjustment. Bank advance processes, that is, processes of bank deposition and channel narrowing, are excluded by most of the modeling approaches reviewed here.

Fluvially controlled processes of bank retreat are essentially twofold. Fluvial shear erosion of bank materials results in progressive incremental bank retreat. Additionally, increases in bank height due to near-bank bed degradation or increase in bank steepness due to fluvial erosion of the lower bank may act alone or together to decrease the stability of the bank with respect to mass failure. Bank collapse may lead to rapid, episodic retreat of the bankline. Depending on the constraints of the bank material properties and the geometry of the bank profile, banks may fail by any one of several possible mechanisms Thorne 1982), including planar (e.g., Lohnes and Handy (1968)-, rotational (e.g., Bishop (1955)-, and cantilever (e.g., Thorne and Tovey (1981)-type failures. A separate analysis is required for analysis of bank stability with respect to each type of failure.

Nonfluvially controlled mechanisms of bank retreat include the effects of wave wash, trampling and grazing by livestock, and piping- and sapping-type failures (e.g., Hagerty 1991; Ullrich et al. 1986) associated with stratified banks and adverse groundwater conditions. Nonfluvial processes leading to bank retreat are excluded from all of the models reviewed here.

7.4.5.3.2 Fluvial Entrainment of Bank Materials For models of noncohesive bank erosion, hydraulic shear erosion of the banks is implicitly simulated through application of the sediment-transport submodel in the near-bank zone. Comparatively little is known about the mechanics of cohesive-bank fluvial entrainment. Excess-shear-stress formulations are difficult to apply because the value of shear stress required to entrain the bank particles varies widely and is influenced by diverse processes (Grissinger 1982). For example, processes such as frost heave or desiccation, which result in weakening of the intact material, may exert a more dominant control on observed rates of fluvial erosion than the intensity of the near-bank flow (Lawler 1986).

It is important to include a method of predicting the hydraulic shear erosion of cohesive bank materials in width-adjustment modeling because erosion directly influences the rate of retreat of the banks, and it also steepens the bank profiles and promotes retreat due to mass bank instability. Approaches that exclude analysis of fluvial erosion of bank materials (Table 7-4) are therefore somewhat limited. Widening models that attempt to account for fluvial erosion of cohesive bank materials (Table 7-4) utilize empirically based methods, such as that of Arulanandan et al. (1980), which was reviewed extensively by Osman and Thorne (1988). Borah and Dashputre (1994) and Darby and Thorne (1996b) have, however, suggested that these methods are subject to serious shortcomings.

7.4.5.3.3 Cohesive- and Noncohesive-Bank Stability Analyses Despite the fact that natural riverbanks are liable to failure by a number of specific mechanisms of bank collapse, most cohesive bank-width-adjustment modeling approaches (Table 7-4) have been based solely on analysis of planar failures.

The mass-wasting algorithms developed by Osman (1985) and reported in Osman and Thorne (1988) account for the bank profile geometry associated with natural, eroding

Table 7-4 Features of Bank Mechanics Submodels of Reviewed Models

Model (1)	Bank Process				Bank Material			
	Deposition (2)	Fluvial Entrainment (3)	Types of bank failure (4)	Longitudinal extent of failure included (5)	Cohesive (6)	Noncohesive[a] (7)	Layered (8)	Heterogenous (9)
Darby-Thorne	No	Yes	Planar curved	Yes	Yes	No	No	No
CCCHEBank	Yes	Yes	None	No	No	Yes	No	No
Kovacs-Parker	No	Yes	None	No	No	Yes	No	No
Wiele	No	Yes	None	No	No	Yes	No	No
RIPA	No	Yes	Planar	No	Yes	No	No	No
Simon et al.	No	No	Planar	No	Yes	No	No	No
Pizzuto	No	Yes	None	No	No	Yes	No	No
STREAM2	No	Yes	Planar	No	Yes	No	No	No
GSTARS	—[b]	—[b]	—[b]	—[b]	—[b]	—[b]	—[b]	—[b]
FLUVIA L-12	—[b]	—[b]	—[b]	—[b]	—[b]	—[b]	—[b]	—[b]
Alonso-Combs	No	No	Planar	No	Yes	No	No	No
WIDTH	No	Yes	Planar Curved	No	Yes	No	No	No

[a]Noncohesive bank sediments are assumed uniform in size.
[b]Bank mechanics submodels are not included in these models, which are instead based on extremal hypotheses.

riverbanks that are destabilized through a combination of lateral erosion and bed degradation. These algorithms are employed in most of the cohesive bank approaches listed in Table 7-4. Previous stability analyses were restricted to a simple bank geometry and excluded the effects of lateral fluvial erosion on the bank profile (Lohnes and Handy 1968; Little et al. 1982).

The Osman-Thorne stability analysis is, however, subject to two main limitations (Simon et al. 1991). First, it does not include the effects of pore-water pressure and hydrostatic confining pressures. Second, the analysis constrains the failure plane to pass through the toe of the bank, excluding the possibility of secondary, upper-bank failures. Such failures are fairly common (Thorne et al. 1981a,b; Simon and Hupp 1992).

Simon et al. (1991) employed a bank stability analysis designed to account explicitly for hydrostatic and pore-water pressure effects on bank stability, while relaxing the assumption that the failure plane must pass through the toe of the bank. This enables bankline adjustments in response to secondary, upper-bank failures to be simulated. Conversely, Simon et al. (1991) excluded the effects of fluvial erosion on the bank profile that were accounted for in the Osman-Thorne (1988) stability analysis.

Darby and Thorne (1996a) accounted for two specific mechanisms of bank erosion and retreat, using the stability analyses proposed by Osman (1985) for rotational failure mechanisms and Osman and Thorne (1988) for planar failure mechanisms. Consideration of both rotational and planar failures, the failure mechanisms being discriminated on the basis of lower predicted factor of safety, represents the first attempt to account for the possibility of multiple failure mechanisms. This is important, because the shape of the failure surface is largely determined by the failure mechanism, and the failure surface forms the new bank profile following mass failure. Because stability of the bank is sensitive to the shape of the bank profile, predicting the correct failure surface is important in ensuring that predictions of bank stability and retreat continue to be accurate throughout a model simulation that includes several consecutive bank failures. However, the range of specific mechanisms of bank collapse included by Darby and Thorne (1996c) is still small compared to the number of potential failure mechanisms that may occur in nature.

For noncohesive riverbanks, models of widening have been proposed by Wiele and Paola (1989); Pizzuto (1990); Kovacs (1992); Wiele (1992); Li and Wang (1993; 1994a,b); and Kovacs and Parker (1994). These approaches can be subdivided into two categories. First, Pizzuto (1990) and Li and Wang (1993; 1994a,b) simulate the bank erosion mechanism using a heuristic procedure (a similar approach is also adopted by Nagata et al. [2000]). When bank slope exceeds the angle of repose of the boundary materials, a slumping model is employed such that a failure surface inclined at the angle of repose is projected to the flood-plain surface. Sediment above the failure plane is moved downslope, forming a deposit with a linear upper surface.

The second approach is characterized by the work of Kovacs and Parker (1994). Their vectorial bed load equation and bank erosion models represented considerable advances in modeling noncohesive sediment transport. Kovacs and Parker (1994) realized that the fundamental problem of previous analyses was that the bed-load formulations employed were valid only at angles much less than the angle of repose, but it is the entrainment and transport of noncohesive sediment particles on steep slopes that is precisely the problem of interest. To avoid this problem, Kovacs and Parker (1994) formulated a vectorial bed load transport equation (Parker and Kovacs 1993) for coarse-sediment transport that was applicable to slopes up to the angle of repose in both the streamwise and transverse directions. Kovacs and Parker (1994) applied the vectorial bed load transport equation to simulate the widening observed by Ikeda (1981) in his laboratory experiments. According to their approach, widening is initiated when bank erosion along the lower part of the bank causes the local slope of the upper bank to exceed the angle of repose of the sediments. A discontinuity in slope is created between the over-steepened upper bank and the lower part of the bank; this discontinuity migrates up the bank with a characteristic velocity, widening the channel as it propagates. Using their bed load transport equation and an integral form of the sediment continuity equation, Kovacs and Parker (1994) derived a rigorous expression for the propagation velocity of the discontinuity in slope, allowing them to reproduce the widening rates observed by Ikeda (1981).

Further development of their methods is needed before they can become a practical design and simulation tool. In particular, the bank erosion and transport models need to be coupled with a sophisticated 2D or 3D flow model to account for complex hydraulics found in natural rivers. Furthermore, the method should also be extended to account for mixtures of varying grain sizes before it can be widely applied to field conditions.

7.4.5.3.4 Homogenous and Heterogenous Bank Structures The physical properties of natural riverbanks are frequently characterized by great spatial variability in their vertical structure and distribution. Many banks are composed of multiple sediment horizons, often featuring a fine-grained cohesive layer above a noncohesive granular layer. Despite this, all of the bank stability analyses employed in the models reviewed here assume that banks are characterized by a homogeneous vertical structure. Additionally, some models (Table 7-4) do not represent spatial variation in the physical properties of bank materials, either along the banks in the streamwise direction, or extending into the flood plain.

7.4.5.3.5 Longitudinal Extent of Mass Failure Most of the reviewed analyses assume that the volume of bank sediments delivered to the channel per unit reach length, required as a source term in the sediment continuity equation, is equal to the product of the unit failure volume of bank material and the reach length. Application of bank stability analyses without consideration of the actual longitudinal extent of the failure can result in serious overestimation of this source term in the sediment continuity equation, propagating errors in estimated bed and bank adjustments throughout the entire simulation. Darby and Thorne (1996b) attempted to account for the longitudinal extent of mass failures within modeled reaches. Darby and Thorne suggested that the volume of sediment supplied within a modeled reach should be equal to the unit volume (per unit channel length) supplied by mass-wasting processes multiplied by the product of the length of the modeled reach and the probability of failure occurring at the computational node. Darby and Thorne suggested that the measurable statistical variations in bank material properties along the reach (Simon 1989) could be substituted into the deterministic Osman-Thorne bank stability equations to obtain the probability of failure using the procedure of Huang (1983). Darby and Thorne's approach is a tentative first step toward solving this important problem.

7.4.5.4 Interaction of Fluvial Hydraulics and Bank Mechanics

7.4.5.4.1 Approaches Based on Extremal Hypotheses Two numerical models that use extremal hypotheses to simulate width and other channel adjustments are the FLUVIAL-12 (Chang 1988a,b) and GSTARS codes (Molinas and Yang 1986; Yang et al. 1988). FLUVIAL-12 and GSTARS assume that changes in cross-sectional area determined from the sediment-routing module represent an overall change in area that may be applied to both the bed and the banks. The total area is distributed over the cross section by first calculating the magnitude of width adjustment, and then distributing the computed area over the bed and banks. Width corrections at each cross section are computed assuming that the stream power for the reach moves toward uniformity (FLUVIAL-12) or toward a minimization of energy dissipation rate (GSTARS), in accordance with the extremal hypothesis that forms the basis for each of these approaches. However, banks composed of cohesive sediments are not accounted for in any of the (noncohesive) sediment-transport equations used in the sediment-routing module. This procedure is not obviously applicable, therefore, to channels with banks composed of cohesive sediments.

FLUVIAL-12 and GSTARS also add entrained bank materials into the bed-material transport scheme simplistically: The bank-material size distribution is transferred instantaneously to the bed-material active layer. Although this is reasonable for noncohesive sediments, the processes of cohesive bank-material breakdown are not yet known. The authors of the two models provided no information on how both cohesive and noncohesive bank sediments were distributed across the channel section following mass failure.

Independent of their capability to predict changes in channel width, FLUVIAL-12 and GSTARS are both characterized by another limitation. Only an overall estimate of the total change in channel width in any time step is made by the extremal hypothesis, and therefore the extent of advance and/or retreat of the left and right banks individually is unknown. Distributions of changes in total width between left and right banks are specified by the user.

7.4.5.4.2 Geofluvial Approaches In contrast to approaches based on extremal hypotheses, other methods have been developed that are based on coupling flow- and sediment-routing models with bank-erosion and mass-wasting algorithms. Such approaches are here termed "geofluvial" and focus on treating bankline adjustments mechanistically. Critical issues concern the need to

1. Predict accurately, in channels with the complex topography characteristic of natural rivers, the boundary shear stress distribution in each of the near-bank zones;
2. Determine the corresponding sediment flux field over the entire channel width;
3. Use the boundary shear stress distribution to determine the rate of fluvial particle-by-particle erosion on both banks, whether composed of cohesive or noncohesive materials;
4. Estimate the stability of the updated bank geometries and determine the volume (if any) of bank sediments delivered to the channel;
5. Characterize the exchanges of sediment between the banks and the bed material to satisfy conservation of sediment mass in channels that either are undergoing width adjustments, or are laterally migrating with stable width.

Topic 5 is the main focus of concern in this section. In geofluvial approaches, interactions between fluvial hydraulic and bank processes are modeled based on a solution of the sediment continuity equation. A given bed topography describes the geometry of the bank profile. Estimates of the sediment flux field and stability of the banks with respect to mass failure are then obtained. If a bank is unstable, then the width of the simulated failure block(s) determines the magnitude of bankline retreat during a time step. The volume of material involved in the failure, determined by the geometry of the failure surface, controls the bank-material input term in the sediment continuity equation, which is solved to determine the bed topography in the subsequent time step.

To couple the flow- and sediment-routing and bank-mechanics submodels in this way, an overall estimate of the failure-block volume is, in itself, insufficient. Precise details of the mechanics by which the failed bank materials are transferred down the failure surface are needed, because the lateral distribution of failure products determines the magnitude of the bank-material inflow term at each computational node. In addition, information regarding the physical properties (size, density, and cohesion) of the disturbed bank material at each node is required so that the fluvial transport of these materials can be calculated in subsequent time steps.

No empirical information regarding the processes of, and controls on, the lateral distribution and physical status of bank material following fluvial entrainment or mass failure is currently available, either for laboratory or natural channels. Empirical information is not available regarding the fluvial transport of heterogeneous mixtures of disturbed bank and bed material. Conceptually, the lateral distributions and physical status of failed bank materials are determined by the geometry of the failure surface and channel-bed topography, the physical characteristics of the undisturbed bank materials, and the hydraulics of the flow.

In light of these difficulties, a distinction can be made between mechanistic widening models applicable to cohesive and noncohesive bank materials. For noncohesive banks, at least the physical status (size, density, and cohesion) of disturbed noncohesive bank materials is known, because these values are identical to those of the undisturbed bank materials. In contrast, disturbed cohesive bank materials may have physical properties distinct from those of intact bank materials, particularly if the failure products become immersed in the flow.

For noncohesive banks, two main approaches to estimating the lateral distribution of bank failure products can be identified. Pizzuto (1990) and Li and Wang (1993; 1994a,b) employed schemes such that, when the bank slope exceeded the angle of repose, a heuristic slumping model was employed in which a failure surface inclined at the angle of repose was projected to the floodplain surface. Sediment above the failure plane was translated downslope, forming a deposit with a linear upper surface. The highest point of the deposit was the lowest point of the failure plane. The deposit extended downslope until its value equaled the volume eroded. Wiele (1992) and Kovacs and Parker (1994) employed an approach in which the sediment continuity equation was manipulated to treat the bank erosion products as a transverse sediment flux. This approach is more consistent with a grain-by-grain noncohesive bank erosion mechanism, whereas the former approach is more consistent with slumping or toppling mechanisms of bank failure (Wiele 1992).

For cohesive banks, geofluvial approaches assume that failed bank materials are instantaneously deposited close to the toe of the bank. Failure products are distributed uniformly across the near-bank flow segments defined by Simon et al. (1991) and Darby and Thorne (1996c). Mosselman (1992) stated that failure products were distributed evenly across the near-bank computational cells. Borah and Bordoloi (1989) used a linear distribution function based on local sediment transport capacity. Osman (1985) and Alonso and Combs (1986) did not specify exactly how bank failure products were distributed in their models, other than stating that they were deposited close to the toe.

Some mechanistic approaches (Osman 1985; Alonso and Combs 1986; Borah and Bordoloi 1989; Mosselman 1992) assume that the banks are composed of a fraction of cohesive material (ω) that becomes wash load after being eroded and a fraction of noncohesive materials ($1-\omega$) with the same properties as the bed material. The sediment-transport submodels employed in these approaches are then directly applied to compute transport rates for noncohesive sediment.

Simon et al. (1991) proposed a conceptual model where failed bank materials are considered to represent bank material, bed material, bed-material load, or wash load, according to the physical properties of the failed materials and the hydraulic properties of the flow. The approach they present is perhaps best regarded as a conceptual framework from which to proceed. Application of the existing approach is currently hindered by two limitations. First, Simon et al. (1991) did not allow the possibility of bed-material load being deposited on the banks, thus excluding the possibility of fluvially controlled bank-accretion and channel-narrowing mechanisms. Second, no information is yet available on how to predict the physical properties of the failed bank materials that are significant with respect to fluvial transport processes.

Darby and Thorne (1996c) assumed that undisturbed cohesive bank material failure blocks tended to disaggregate into disturbed aggregates of some measurable size range during mass failure. Darby and Thorne noted that these disturbed aggregates, though composed of cohesive particles, were themselves large enough to behave as noncohesive sediment particles. Darby and Thorne went on to suggest a criterion to discriminate whether or not the failure block would disaggregate, based on energy dissipated during mass failure and internal resistance of the failure block. Darby and Thorne used the criterion to hypothesize that steep planar failures would tend to result in disaggregated blocks delivered to the basal region of the bank as noncohesive sediment clasts, whereas shallower rotational failures would tend to remain as intact blocks of bank materials. Knowledge of the size and density of deposited sediment assumed to behave as noncohesive sediment particles allowed standard sediment-transport analyses for heterogeneous sediment (Rahuel et al. 1989) to be applied to the failed bank material aggregates deposited as bed material in the near-bank sediment-routing segments. No means of predicting the size of the disturbed bank material aggregates was suggested by Darby and Thorne.

7.4.5.5 Testing and Application of Numerical Models The capabilities, predictive abilities, scope, limitations, and usefulness of the various numerical models are now summarized. Tables 7-2 to 7-5 indicate that the reviewed models are limited in terms of the range of conditions to which they may be applied, as determined by the limitations of the assumptions in the hydraulic, flow-resistance, sediment-transport, and bank-erosion modules used in each model.

7.4.5.5.1 Tests with Laboratory Data The reviewed models applicable to noncohesive bank materials (Pizzuto

Table 7-5 Summary of Approaches, Testing Status, and User Documentation of Reviewed Models

Model (1)	Approach (2)	Planform (3)	Test case run (4)	Laboratory data test (5)	Field data test (6)	User's Manual (7)
Darby-Thorne	Geofluvial	Straight	Yes	No	Yes	No
CCHEBank	Geofluvial	Straight	Yes	Yes	No	No
Kovacs-Parker	Geofluvial	Straight	Yes	Yes	No	Yes
Wiele	Geofluvial	Straight	Yes	Yes	Yes[a]	No
RIPA	Geofluvial	Arbitrary single-thread	Yes	No	Yes[a]	No
Simon et al.	Geofluvial	Straight	No	No	No	No
Pizzuto	Geofluvial	Straight	No	Yes	No	No
STREAM2	Geofluvial	Straight	Yes	No	Yes[a]	No
GSTARS	Extremal	Arbitrary	Yes	No	Yes[a]	Yes
FLUVIAL-12	Extremal	Arbitrary	Yes	No	Yes[a]	Yes
Alonso-Combs	Geofluvial	Straight	Yes	No	No	No
WIDTH	Geofluvial	Straight	Yes	No	No	No

[a]Denotes calibrated field test.

1990; Wiele 1992; Li and Wang 1993; 1994a,b; Kovacs and Parker 1994) have been tested with a common data set obtained from a laboratory study (Ikeda 1981). Results from these studies are shown in Fig. 7-27. However, assessment of the relative performance of these models is not attempted here because some small, but significant, differences are found in the numerical values of coefficients used by each of the aforementioned authors. Specifically, the critical dimensionless Shields stress is assumed to be 0.03 by Li and Wang (1993; 1994a,b) and Pizzuto (1990), 0.035 by Kovacs and Parker (1994), and 0.038 by Wiele (1992), respectively. The value of the internal angle of friction of the boundary material (which also influences the dynamic Coulomb friction coefficient) was assumed to be 33° by Pizzuto (1990) and 40° by the other authors.

Although a direct comparison of the relative performance of each model is not appropriate, Fig. 7-27 can be used to provide some insight into the capabilities of each of the individual models. The Kovacs and Parker model (Fig. 7-27(a)) resulted in predicted cross sections with cross-sectional areas larger than those measured in reality. Pizzuto's (1990) (Fig. 7-27(b)) model provided close agreement between simulated and measured channel shapes throughout the extent of the simulation. Wiele's (1992) model (Fig. 7-27(b)) underpredicted measured widening rates, presumably reflecting the relatively high Shields stress and friction-angle values selected by that author. Finally, Li and Wang (1993; 1994a,b) obtained overpredictions of widening compared to the observed channel changes (Fig. 7-27(d)).

7.4.5.5.2 Field Testing Those authors who have attempted to test their models with field data have tended to calibrate the adjustable model parameters to improve agreement between predicted and observed data. Authors also tend to characterize their results using qualitative terminology such as "reasonable agreement" and "acceptable results." In these circumstances, it is futile to attempt to summarize and compare the accuracy of those models, particularly because the same source data set has not been used to test each analysis. For calibrated testing analyses, the reader is referred to the source material. Borah et al. (1982) and Borah and Dashputre (1994) tested components of the Borah and Bordoloi (1989) model, whereas Chang (1988a,b), Yang et al. (1988), Mosselman (1992), and Wiele (1992) fully reported both the development and testing of their codes.

One model (Darby and Thorne 1996c) has been applied with unadjusted calibration parameters (Darby et al. 1996) (Fig. 7-28.). Model calibration parameters were not adjusted from the values set during the course of the model development. Although the model appeared to be able to replicate the observed sequence of channel adjustment, and the magnitudes of simulated and observed widths and depths agreed within ±10% of each other overall, simulated widening rates were underpredicted by a factor of 3 (Darby et al. 1996). Darby et al. (1996) attributed this poor result to limitations of the Osman and Thorne (1988) mass-wasting algorithm.

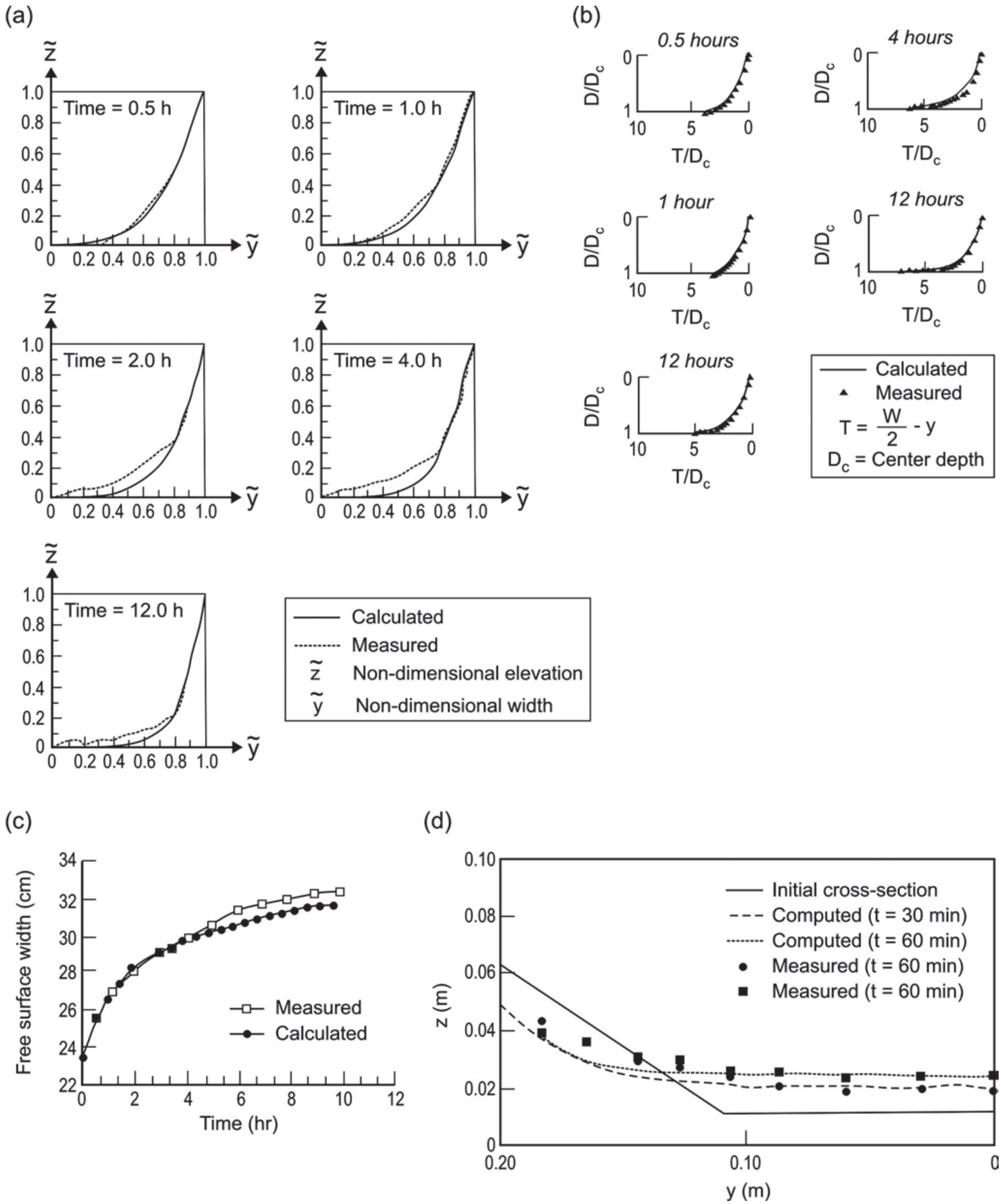

Fig. 7-27. Comparison of simulated output and Ikeda (1981) flume data for models by (a) Kovacs and Parker (1994); (b) Pizzuto (1990); (c) Wiele (1992); (d) Li and Wang (1993; 1994a,b) (ASCE Task Committee on Hydraulics, Bank Mechanisms and Modeling of River Width Adjustment 1998b, with permission from ASCE).

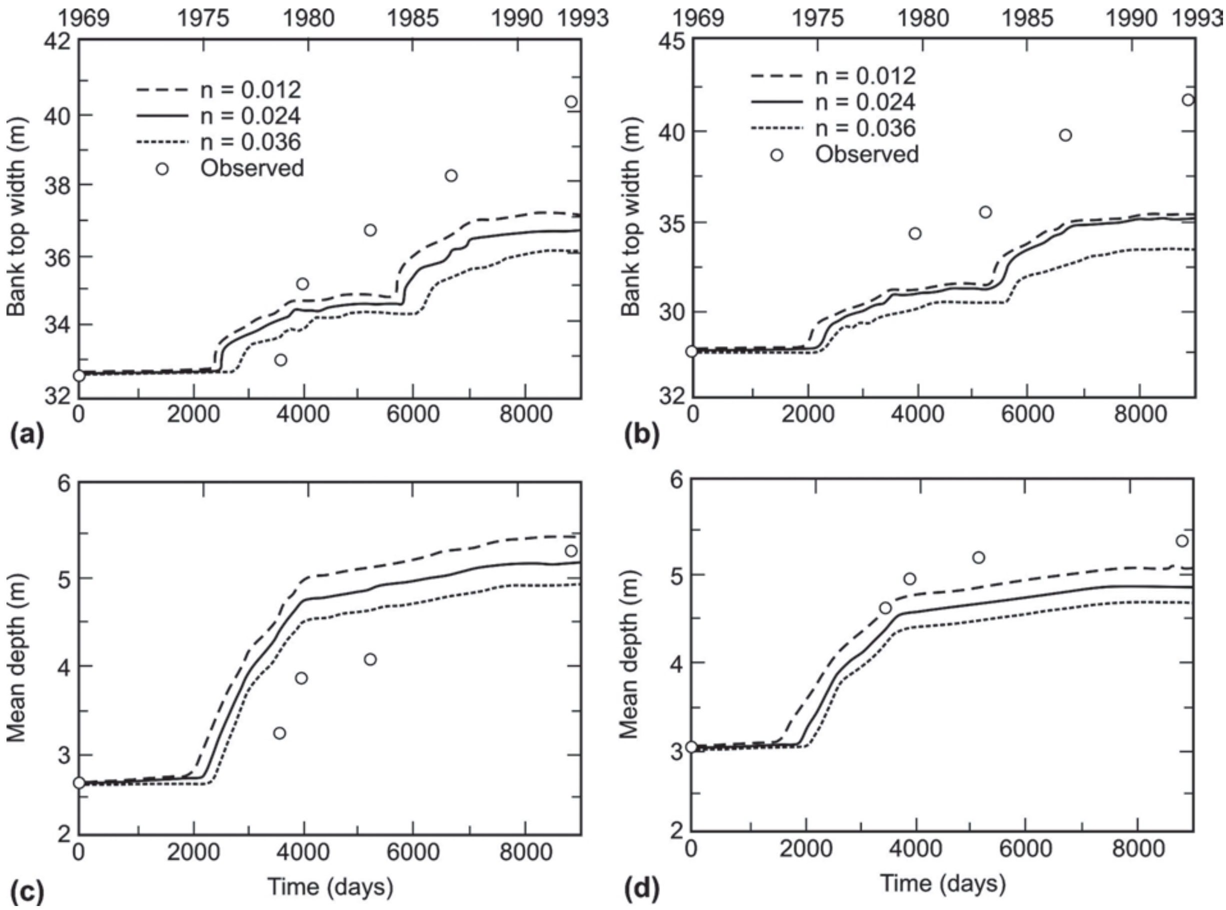

Fig. 7-28. Comparison of simulated versus observed channel morphology parameters for Darby-Thorne model at two study sites in West Tennessee: (a) bank-top widths at Chestnut Bluff; (b) bank-top widths at Crossroads; (c) mean depths at Chestnut Bluff; (d) mean depths at Crossroads (from Darby et al. 1996, with permission from ASCE).

7.5 PROCEDURE FOR APPROACHING WIDTH-ADJUSTMENT PROBLEMS

The wide range of geomorphic and engineering contexts associated with width adjustment makes it essential that practicing engineers adopt a broad and rational approach to such problems. Such an approach can be used to analyze the majority of problems that arise with the assurance that important factors are not overlooked, appropriate analytic techniques are applied, and effective engineering solutions are selected. The procedure proposed here (Fig. 7-29) is based on amassing and utilizing a wide range of information. Although each case is unique, the proposed procedure should have a number of elements that are relevant for the majority of situations.

7.5.1 Step 1: Problem Identification

Width-adjustment problems may be associated with a range of river engineering and societal activities. Questions to consider are the following:

1. Does the problem arise from a natural response?
2. Does it involve channel response to existing engineering works?
3. Does it require the prediction of channel response to proposed engineering works?

In all cases, it is necessary to formulate the problem in terms of whether it is existing or predicted, who or what is affected, and what level of analysis and response is appropriate. The aim of successful problem identification is to select a cost-effective engineering approach that will solve the problem.

7.5.2 Step 2: Field Data Collection

In all cases, visits to the site and river reaches upstream and downstream are essential. Particular attention should be paid to identifying channel characteristics, bank conditions, bank materials, the extent of existing or expected bank problems, the nature of the flow, the nature of the bed materials, the presence and nature of any vegetation, and the presence and

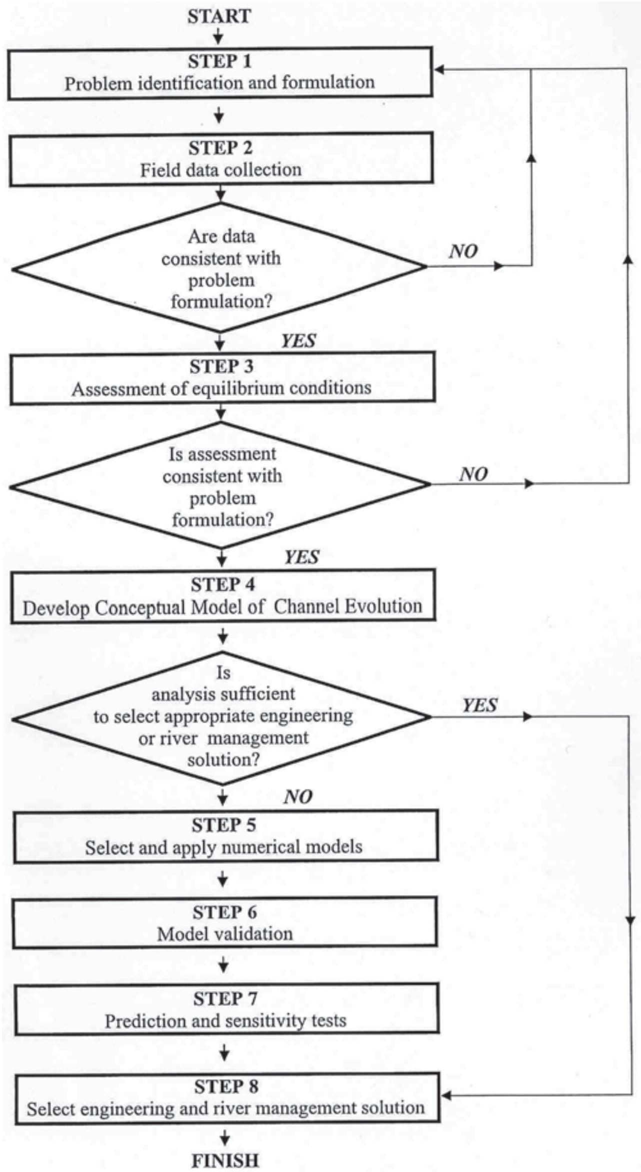

Fig. 7-29. Proposed procedure for identifying, analyzing, and modeling width-adjustment problems.

condition of any engineering structures. Stream reconnaissance techniques are described by, among others, Kellerhals et al. (1976); Thorne (1992); and Downs and Brookes (1994).

In all cases, it is necessary to identify the nature and extent of the width-adjustment problem that may arise. Where there have been width changes in the past, both reaches that have been subject to change and reaches that are stable should be examined.

Depending on the number of existing data available, it may be necessary to mount a specific data-gathering campaign. Data are needed to assess the equilibrium morphology of the channel and, in some cases, to understand the nature of the problem. If the use of numerical models is warranted, field measurements will always be needed. Data requirements for numerical modeling studies are discussed further in Appendix I.

7.5.3 Step 3: Assessment of Equilibrium Morphology

As a first step, the equilibrium morphology of the channel should be estimated using methods described in section 7.4.3. Of the methods discussed in section 7.4.3, regime theory is probably the most reliable, but field data near the particular field site will be needed to determine the necessary empirical coefficients and exponents.

Once predictions of the equilibrium morphology are available, the predicted morphology should be compared with the existing morphology to provide an assessment of the current morphological status of the channel; for example, whether it is overwide, of equilibrium width, or underwide. Where the impact of proposed engineering works is being considered, the equilibrium conditions should also be compared to the proposed channel conditions.

7.5.4 Step 4: Developing Conceptual Models of Channel Evolution

If the channel is actively evolving under natural conditions, or is responding to engineering intervention or regulation, then simple empirical channel response or dynamic models, such as those described in section 7.4.2, should be developed and applied in an attempt to explain both existing and, if appropriate, proposed conditions. Application of such models should aid in identifying the dominant processes and trends of channel change and can form a framework for subsequent, more detailed modeling.

7.5.5 Step 5: Application of Numerical Models

If the complexity and severity of the width adjustment problem merit numerical modeling, a hierarchical modeling approach will usually be appropriate. Initially a 1D model should be applied to the study reach to provide the overall setting of any additional detailed modeling. If appropriate, to provide a more detailed assessment of width adjustment, it may be necessary to apply 2D or 3D models to the whole or part of the study reach. Selection of numerical models appropriate for this purpose may be guided by the comments provided in this paper. At present, models of width adjustment are still undergoing active development, so selecting a useful model is not a simple task.

7.5.6 Step 6: Model Validation

The numerical model results should be validated. This will nearly always require an extensive program of field observations.

7.5.7 Step 7: Model Prediction

The numerical models should be applied to existing conditions and also used to assess the impacts of any proposed works. Model predictions should include a sensitivity analysis of the results to the various parameters specified in the model. Particular attention should be paid to parameters that either are difficult to determine or exhibit significant spatial or temporal variation.

7.5.8 Step 8: Selection of Engineering or River Management Solution

On the basis of the previous steps, an appropriate plan of action should be formulated and implemented. One example of a management approach is provided by Simon and Downs (1995). They describe an interdisciplinary approach to evaluating stream-channel instability conditions on the regional or statewide scale. The regionwide studies were motivated primarily by the desire of some state transportation departments in the United States to inventory the potential for channel instability to damage bridge crossings and other transportation infrastructure. A modular procedure was developed based on (1) initial site evaluations; (2) geographic-information-system-based data input and management; (3) ranking of relative channel stability conditions; (4) identification of spatial trends; (5) ranking of socioeconomic impacts and identification of problem sites; and (6) collection of additional field data for enhanced desktop and modeling analyses of future conditions at the problem sites (Simon and Downs 1995). Based on this approach, the state transportation departments were provided with a product that enabled them to optimize repair and maintenance schedules for damaged infrastructure or infrastructure at risk from channel adjustment.

7.6 CONCLUSIONS

1. Width adjustments take place within a wide range of geomorphic contexts. Adjustments may occur as part of the natural evolution of the channel morphology, or they may be caused by river engineering structures, river management policies, or changes in land use in the watershed or riparian zone.
2. To understand, predict, and manage changing channel width, it is essential that civil engineers understand the geomorphic context within which width adjustment is occurring.
3. The time- and space-averaged boundary shear stress is an important parameter in predicting both equilibrium width and width adjustment. However, the lateral distribution of local values of boundary shear stress is poorly understood, especially for channels with nonuniform cross sections.
4. Improved understanding of the effects of over-bank flows on river-width adjustment processes is needed.
5. A variety of mass-failure mechanisms may be involved in bankline retreat. Care must be taken to match the slope stability analysis used to check bank stability to the critical failure mechanisms observed in the field. It is essential that engineers identify actual and potential instability mechanisms prior to selecting an engineering or management strategy for dealing with bank retreat and width adjustment.
6. The long-term rate of bank retreat or advance of the bank toe can be explained using the concept of basal endpoint control. However, seepage-driven procedures operating within a bank can lead to serious bank instability due to piping even when wave and current action at the toe is not excessive.
7. Bank advance takes place through sediment accumulation as a berm or bench in the channel and by the development of floodplains on migrating point bars. Bank advance is often accelerated by invasion of pioneer riparian vegetation.
8. Current knowledge of bank processes and flow modeling is sufficient to allow some tentative *predictions* of width adjustment to be made.
9. Analysis of equilibrium width in stable channels can be approached using (1) empirical regime methods; (2) extremal hypotheses; and (3) rational tractive force methods. These approaches are strictly limited to prediction of time-invariant width in graded or regime channels. They can be used with care to predict asymptotic values of width following disturbance of the graded or regime condition, but they cannot predict either the rate of change or intermediate width attained during dynamic adjustment of channel morphology. Tractive force methods are limited to straight channels with noncohesive banks. Despite these limitations, these methods have many useful engineering applications.
10. To date, models of river width adjustment can be divided into two broad approaches: (1) those based on extremal hypotheses, and (2) those based on the geofluvial approach. The former have been used in engineering practice more frequently than the latter, which are at present used essentially as research tools. However, geofluvial approaches have the potential to become adopted as standard engineering tools.
11. Currently, very few appropriate laboratory and field data sets are suitable for testing width-adjustment models. This has resulted in a lack of comprehensive testing and verification analyses of existing models on benchmark field and laboratory data sets.
12. At present, no single model or method exists that is applicable to all the circumstances under which width adjustments may occur.

APPENDIX. DATA SOURCES

1 Equilibrium Channels

Equilibrium channel geometry measurements have been reported for at least a century. A summary of published data sources was presented by Julien and Wargadalam (1995), based on a compilation of available data by Wargadalam (1993). The data encompass measurements from 835 field channels and 45 laboratory channels that were used to test semi-theoretical downstream hydraulic geometry relationships.

Brownlie (1981a,b; 1983) published an extensive compilation of laboratory and field data. Khan (1971) reported 45 laboratory measurements of hydraulic geometry for straight, meandering, and braided reaches. Griffiths (1981) reported 136 gravel-bed river geometry measurements collected from 46 rivers in New Zealand. Of these, 84 were conducted under rigid bed conditions, whereas 52 are for mobile bed conditions. Church and Rood (1983) published a compendium of river regime data that lists 496 hydraulic geometry measurements reported in the technical literature. This data set includes measurements from rivers in Canada and the United States, which were carefully selected from 25 references published between 1955 and 1983. Hey and Thorne (1986) reported data from 62 river measurement sites from stable gravel-bed rivers in the United Kingdom. Higginson and Johnston (1988) published data from 68 sites under bank-full flow conditions from rivers in Northern Ireland. Colosimo et al. (1988) published 42 gravel-bed river measurements from streams in Calabria, Southern Italy. The range of flow parameters covered by all these data is summarized in Table 7-7.

Table 7-7 Range of Flow Parameters Covered in Equilibrium Channel Data Set of Julien and Wargadalam (1995)

Parameter (1)	Range (2)
Discharge	0.00018–26,600 m^3/s
Channel width	0.16–1,100 m
Average flow depth	0.003–15.7 m
Mean flow depth	0.09–4.7 m/s
Channel slope	0.00004–0.08
Median grain size	0.12–400 mm
Width/depth ratio	4.2–507
Relative submergence	1.4–70,400
Froude number	0.017–4
Shields number	0.001–8.5
Grain shear Reynolds number	1.6–156,000

2 Nonequilibrium Channels

For nonequilibrium channels, data sets that include all the parameters required to apply width adjustment models (Table 7-6) are comparatively rare. Laboratory experiments involving width adjustments in straight channels formed in sand were conducted by Ikeda (1981). Ikeda et al. (1988) performed similar experiments in a gravel channel. Data on width adjustment in rivers can be found in Brice (1982); Nanson and Hickin (1983); Richardson et al. (1990); and the USACE (1981). However, these reports do not contain all of the required data listed in Table 7-6. Data sets that include many of the parameters listed in Table 7-6 are generally not available in the literature. However, three data sets have been identified that are suitable for use with numerical models of width adjustment. Data for the Toutle River, Washington, are described by Simon (1992). Similarly, data from the South Fork Forked Deer River, West Tennessee, were used by Darby et al. (1996) to test the Darby and Thorne (1996c) numerical

Table 7-6 Minimum Data Required to Apply Geofluvial-Based Numerical Width Adjustment Models

Data Item (1)	Notes (2)
(*a*) Time-independent data (initial conditions)	
Cross-sectional surveys	Required to define initial channel morphology. Surveys are required at several sites along the prototype reach.
Bed material size distribution	Required to define the initial bed material characteristics. Data is required at each cross section.
Bank material characteristics	Measurements of cohesion, friction angle, unit weight, and particle size distribution at left and right banks of each cross section are required to define the bank-material characteristics.
(*b*) Time-dependent data (boundary conditions)	
Discharge	Value of discharge to be used in each discrete time step of the simulation
Sediment supply	Value of sediment load at the upstream boundary of the prototype reach during each discrete time step of the simulation.

model. Further information about these two data sets may be obtained through contact with the authors of these reports. Finally, data from Goodwin Creek, Mississippi, are available through contact with personnel at the USDA-ARS National Sedimentation Laboratory, Oxford, Mississippi.

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