River Width Adjustment. II: Modeling

By the ASCE Task Committee on Hydraulics, Bank Mechanics, and Modeling of River Width Adjustment[1]

Abstract: Many existing numerical models of river channel morphology are limited in applicability because they neglect time-dependent changes in channel width. In this paper, the status of field-based and numerical modeling approaches for assessing river channel width adjustments is reviewed. This review complements the review of fluvial hydraulics and bank mechanics presented in the companion paper. In addition to describing a field-based approach for assessing channel width adjustments, the quantitative time-dependent models of width adjustment that are currently available are described. Relatively few numerical models of width adjustment have been developed to date, and many processes of width adjustment have never been successfully quantified. Existing models are research tools and are not yet ready to be adopted in widespread engineering practice. An interdisciplinary approach to the analysis and modeling of river width adjustments is presented. The hierarchical approach to analysis of width adjustment is based on field, analytical, and numerical modeling techniques. The principal limitations of existing field-based and numerical modeling approaches are listed. A list of recommendations for further research is provided.

INTRODUCTION

This paper reviews the status of field-based and numerical modeling approaches for assessing river channel width adjustments, and complements the review of fluvial hydraulics and bank mechanics presented in the companion paper. In addition to describing a field-based approach for assessing channel width adjustment, the quantitative time-dependent models of width adjustment that are currently available (Table 1) are described. Numerical models create conceptions of physical reality that result in quantitative predictions. Given the complexity of natural systems, and gaps in our knowledge, the modeling approach is to restructure reality to a form that fits the resources and permits quantitative prediction. These models of restructured reality are what are used and published, forming the basis for subsequent efforts. Potential users must judge if in fact the model will solve a particular problem. As a general premise, users must accept a fair discrepancy between their problem and the model. In this paper, limitations of existing numerical models are discussed, to help clarify the nature of these discrepancies. An approach to reducing such discrepancies is presented and recommended. A list of conclusions summarizing the main points of the review is provided. That summary identifies the principal limitations of existing field-based and numerical modeling approaches. A list of recommendations for further research is provided.

As described in the companion paper, changes in stream channel width are caused by a wide variety of phenomena. Understanding and simulating these diverse processes is, therefore, challenging. The spatial and temporal domain of the quantitative model used to simulate these processes must, therefore, be defined. The spatial domain may include a few or possibly hundreds of channel widths, while the temporal scale of the model could span a few years, decades or even centuries (Tetzlaff and Harbaugh 1989). Defining the temporal scale also has implications for the availability of field data to test and validate the model at all stages of development.

Once a conceptual model of width adjustment has been developed, mathematical and numerical models must be selected to quantify it. In some cases no adequate quantitative model may be available, and empirical models based on appropriate field data may have to be used. Physically based models require coupling of a variety of processes that control width adjustment (see companion paper). For example, hydraulic models are required to quantify the fluid mechanics involved in width adjustment and sediment transport. Bank process models (deposition and erosion) are needed to define bank retreat or advance. Another submodel is required to simulate how the properties of newly deposited or eroded bank materials change with time. Sediment transport models are required to route materials to and through the bank region. A wide range of sediment sizes is often involved in width adjustment, and so models must be valid for individual size fractions ranging from silt and clay to gravel.

RAPID ASSESSMENT TECHNIQUES

Empirical Models of Channel Evolution

Although applicable only to incised channels, the conceptual channel evolution model of Harvey and Watson has been of value in developing an understanding of watershed and channel dynamics and in characterizing whether or not a reach is stable (Fig. 1). 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 (1988) and Simon and Hupp (1992).

Type I reaches are characterized by a 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 (providing 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 a sediment transport capacity that exceeds sediment supply, a 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 value than the Type I reach

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TABLE 1. List of Reviewed Models

Model (1)	Category (2)	Additional references (3)
Darby and Thorne (1996a)	Geofluvial, cohesive bank	Darby and Thorne (1994); Darby et al. (1996)
CCHEBank (Li and Wang 1993)	Geofluvial, noncohesive bank	Li and Wang (1994)
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 1988)	Extremal hypothesis	
Alonso and Combs (1986)	Geofluvial, cohesive bank	
WIDTH (Osman 1985)	Geofluvial, cohesive bank	Thorne and Osman (1988)

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 a sediment transport capacity that is highly variable with respect to the sediment supply, a 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 locally be greater 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 a sediment supply that exceeds sediment transport capacity resulting in aggradation of the channel bed, a bank height that approaches the critical bank height with a rate of bank failure lower than Type III reaches, a nearly trapezoidal cross-sectional shape, and a 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 initiation of natural levee deposits that form in aggraded reaches that were overwidened during earlier degradational phases.

Types V and VI reaches are located downstream of Type IV reaches and are characterized by a dynamic balance between sediment transport capacity and sediment supply for the effective discharge channel, a bank height that is less than the critical bank height for the existing bank angle, colonization by riparian vegetation, an accumulated bed sediment depth that generally exceeds 1.0 m, a width/depth ratio that exceeds the Type IV reach, and generally a compound channel formed within a newly formed floodplain. The channel is in dynamic equilibrium. Bank angles have been reduced by accumulation of failed bank materials at the toe of the slope and 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 a field reconnaissance that records the characteristic channel forms 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,b).

Channel Stability Diagram

The channel evolution sequence of Schumm et al. (1984) 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. 2 (Watson et al. 1988a,b). 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,b). 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,b). 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 slopes 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 for a degradational reach is >1 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. 2. As the channel evolves from a state of disequilibrium to a state of dynamic equilibrium through the five 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

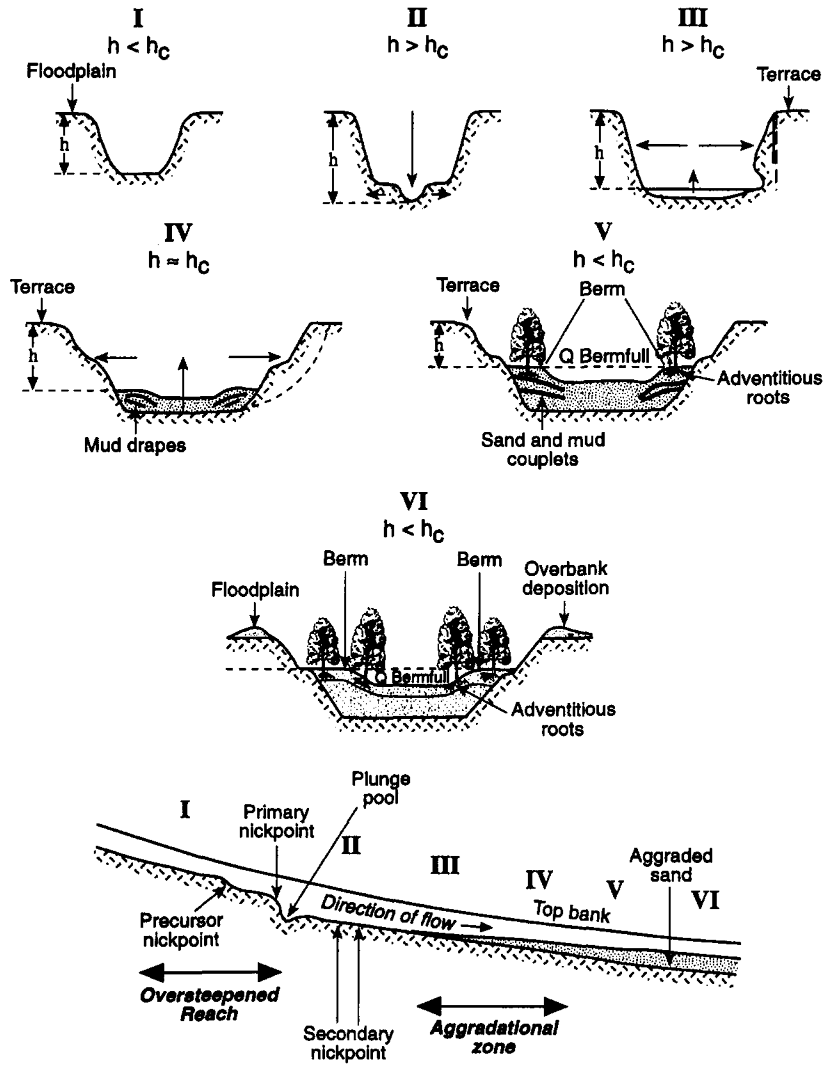

FIG. 1. Six-Stage Sequence of Incised Channel Evolution Originally Used to Describe Evolution of Oaklimiter Creek

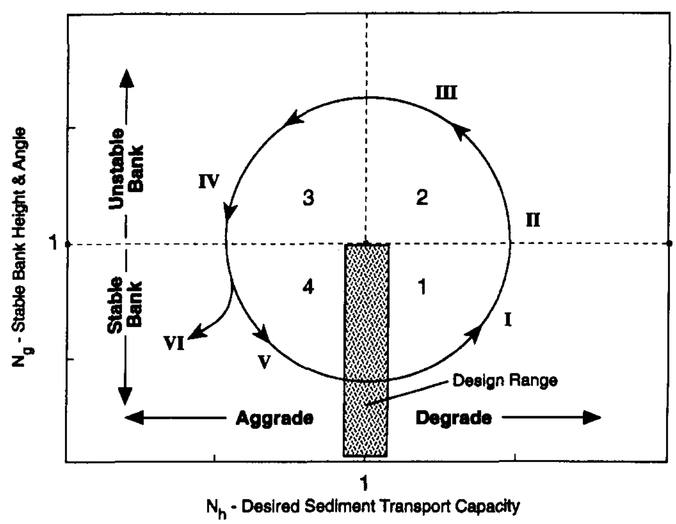

FIG. 2. Comparison of Oaklimiter Creek Channel Evolution Sequence and Channel Stability Parameters

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.

NUMERICAL WIDTH ADJUSTMENT MODELS

Fixed-width numerical morphological models are now commonly used in engineering practice to obtain predictions of channel adjustments in response to changes in the independent variables of flow and sediment discharge. Reviews of the status of fixed-width numerical morphological modeling have been documented by Fan (1988).

Fixed-width numerical models are limited in applicability to cases where width adjustments are not significant in the prototype channel. 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. Models are based on various approaches to representing the governing processes of flow, flow resistance, sediment transport and bank mechanics described in the companion paper. Twelve existing numerical width adjustment models (Table 1) are now reviewed.

Fluvial Hydraulics

Hydraulics and Hydrodynamics

A number of approaches have been used to estimate the flow field in the computational domain of the various numerical models (Table 2). Despite their undoubted significance (see companion paper), analysis of over-bank flows is 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 2). Each of the flow resistance equations in Table 2 are, strictly speaking, valid only for the physical conditions corresponding to the data used to originally derive them. None of the reviewed models account for the effects of vegetation on the flow.

The water routing submodel of the FLUVIAL-12 model (Chang 1988a,b) computes the water surface elevation and energy gradient at each cross section by solving one-dimensional (1D) versions of the flow momentum and continuity equations. For steady flow, the standard step method is employed, while 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-sectional averaged flow parameters and are unable to resolve near-bank boundary shear stresses sufficiently accurately for the purposes of estimating fluvial erosion of bank materials.

Various attempts to account for the lateral variation of the flow field 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 (-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 (1996a) 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 1D gradually varied 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 as 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 the 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 Ko-

TABLE 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[b] (7)
Darby-Thorne	Quasi2D[a]	Stepped hydrograph	No	Yes	Time and space variable	Strickler
CCHEBank	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[a]	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[a]	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]Quasi2D models refer to those models that simulate lateral variation of bed topography through use of multiple 1D stream tubes.
[b]None of these formulas account for the effects of bed forms.

vacs and Parker (1994) model the distribution of fluid-induced boundary shear stress on gently curved riverbanks. 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, 1994), the flow field is computed using CCHE3D (Wang and Hu 1990), an advanced three-dimensional (3D) hydrodynamic model, which can simulate unsteady, 3D free surface turbulent 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 number of 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 friction factor.

Sediment Transport and Continuity

Methods of sediment routing in each of the 12 models reviewed here are summarized in Table 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. Spatial differences in sediment flux so estimated determine the evolution of the bed topography via solution of the sediment continuity equation.

One of a variety of empirically calibrated sediment transport equations (Table 3) is used for estimating the sediment flux field. Some models offer users the choice of specifying a particular equation from a menu. 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 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 for 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 the bank regions that are often inclined at angles close to the angle of repose (typically 35°). The vectorial 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 a 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 bed material grain size adjustments in unstable rivers are as important as adjustments as 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, as 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 3.

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

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 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).

1D sediment routing procedures (Table 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

TABLE 3. Features of Sediment Routing Submodels of Reviewed Models

Model (1)	Routing method (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
CCHEBank	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
FLUVIAL-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

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 (1978). No method of accounting for non uniform distribution of erosion is described.

To address this issue, quasi-2D approaches have been proposed (Table 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 (1996a) 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 3) provide higher definition, though not necessarily more accurate, estimates of bed topography changes in the near-bank zones.

River Bank Mechanics

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

Retreat and Advance Processes

Processes of bank retreat and advance are reviewed in the companion paper and may occur together or separately at different locations and times along the same reach of 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 based on simulating bank stability 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 increases 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, as well as piping- and sapping-type failures [e.g., Hagerty (1991) and Ullrich et al. (1986)] associated with stratified banks and adverse ground-water conditions. Nonfluvial processes leading to bank retreat are excluded from all of the morphological models reviewed here.

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 as 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).

Inclusion of a method to predict the hydraulic shear erosion of cohesive bank materials is important in width adjustment modeling because not only does such erosion directly influence the rate of retreat of the banks, but it also steepens the bank profile and promotes retreat due to mass bank instability. Approaches that exclude analysis of fluvial erosion of bank materials (Table 4) are, therefore, somewhat limited. Widening models that attempt to account for fluvial erosion of cohesive bank materials (Table 4) are based on 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 a serious shortcoming. This is the case because predictions of widening

TABLE 4. Features of Bank Mechanics Submodels of Reviewed Models

	Bank Process				Bank Material			
Model (1)	Deposition (2)	Fluvial entrainment (3)	Types of bank failure (4)	Longitudinal extent of failure included (5)	Cohesive (6)	Noncohesive[b] (7)	Layered (8)	Heterogeneous (9)
Darby-Thorne	No	Yes	Planar Curved	Yes	Yes	No	No	No
CCHEBank	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	—[a]	—[a]	—[a]	—[a]	—[a]	—[a]	—[a]	—[a]
FLUVIAL-12	—[a]	—[a]	—[a]	—[a]	—[a]	—[a]	—[a]	—[a]
Alonso-Combs	No	No	Planar	No	Yes	No	No	No
WIDTH	No	Yes	Planar Curved	No	Yes	No	No	No

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

are sensitive to the estimated excess shear stress value, a parameter that is itself subject to uncertainty.

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 (see companion paper), most cohesive bank width adjustment modeling approaches (Table 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 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 4. Previous stability analyses were restricted to a simple bank geometry and excluded the effects of lateral fluvial erosion on the bank profile and, hence, back stability (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 either 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 for secondary, upper-bank failures. Such failures are fairly common (Thorne et al. 1981; 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 and 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 mechanism being discriminated on the basis of lower predicted factor of safety, represents the first attempt to account for the possibility of occurrence 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. Since 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 is still small compared to the number of potential failure mechanisms that may occur in nature (see companion paper).

For noncohesive riverbanks, models of widening have been proposed by Wiele and Paola (1989), Pizzuto (1990), Kovacs (1992), Wiele (1992), Li and Wang (1993, 1994), and Kovacs and Parker (1994). These approaches can be subdivided into two categories. First, Pizzuto (1990) and Li and Wang (1993, 1994) simulate the bank erosion mechanism using an heuristic procedure. When bank slope exceeds the angle of repose of the boundary materials, a heuristic slumping model was 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. Although the aforementioned authors were careful to state that their models were developed for idealized noncohesive sediments, it should be noted that, in natural environments, the influences of particle interlocking, pore-water pressure and vegetation may all give rise to an apparent "cohesion," so that the slumping model is inappropriate.

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 is that the bed-load formulations employed are 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 for slopes up to the angle of repose both in 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 oversteepened 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 the complex hydraulics found in natural rivers. Furthermore, the method should also be extended to account for mixtures of varying grain size before it can be widely applied to field conditions.

Homogeneous and Heterogeneous 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 the banks are characterized by a homogeneous vertical structure. Additionally, some models (Table 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 floodplain.

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 (1996a) 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 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 in solving this important problem.

Interaction of Fluvial Hydraulics and Bank Mechanics

Approaches Based on Extremal Hypotheses

Extremal hypotheses have been used in equilibrium approaches to predict channel morphology (see companion paper). These equilibrium methods offer predictions of the magnitude, rather than the rate, of width adjustment. This is the principal conceptual objection to width adjustment modeling approaches based on extremal hypotheses.

Two numerical models that claim the capability of simulating width and other channel adjustments and that are based on extremal hypotheses 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 towards uniformity (FLUVIAL-12), or towards 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 a 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.

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 are either undergoing width adjustments, or that 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, 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 are required of the mechanics by which the failed bank materials are transferred down the failure surface, 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 distribution 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 between mechanistic-widening models applicable for cohesive and noncohesive bank materials can be made. For noncohesive banks, at least the physical status (size, density, and cohesion) of disturbed noncohesive bank materials is known, since these values are identical to those of the undisturbed bank materials. In contrast, disturbed cohesive bank materials may have distinct physical properties compared to 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, 1994) employed schemes such that, when the bank slope exceeded the angle of repose, a heuristic slumping model is employed in which a failure surface inclined at the angle of repose is projected to the floodplain surface. Sediment above the failure plane is translated downslope, forming a deposit with a linear upper surface. The highest point of the deposit is the lowest point of the failure plane. The deposit extends downslope until its value equates 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, while 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 (1996a). 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-based 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 the noncohesive fraction of bank material so deposited to determine their fluvial transport rates.

Simon et al. (1991) proposed a conceptual model wherein 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 to be 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 those physical properties of the failed bank materials that are significant with respect to fluvial transport processes.

Darby and Thorne (1996a) 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 the 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, while 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.

TESTING AND APPLICATIONS

The capabilities, predictive abilities, scope, limitations, and usefulness of the various numerical models are now summarized. Tables 2–5 indicate that the reviewed models are limited in terms of the range of conditions in 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 development.

Tests with Laboratory Data

The reviewed models applicable to noncohesive bank materials (Pizzuto 1990; Wiele 1992; Li and Wang 1993, 1994; 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. 3. 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, 1994) and Pizzuto (1990) and 0.035 and 0.038 by Kovacs and Parker (1994) and 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. 3 can be used to provide some insight into the capabilities of each of the individual models. The Kovacs and Parker model [Fig. 3(a)] resulted in

TABLE 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	Goefluvial	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.

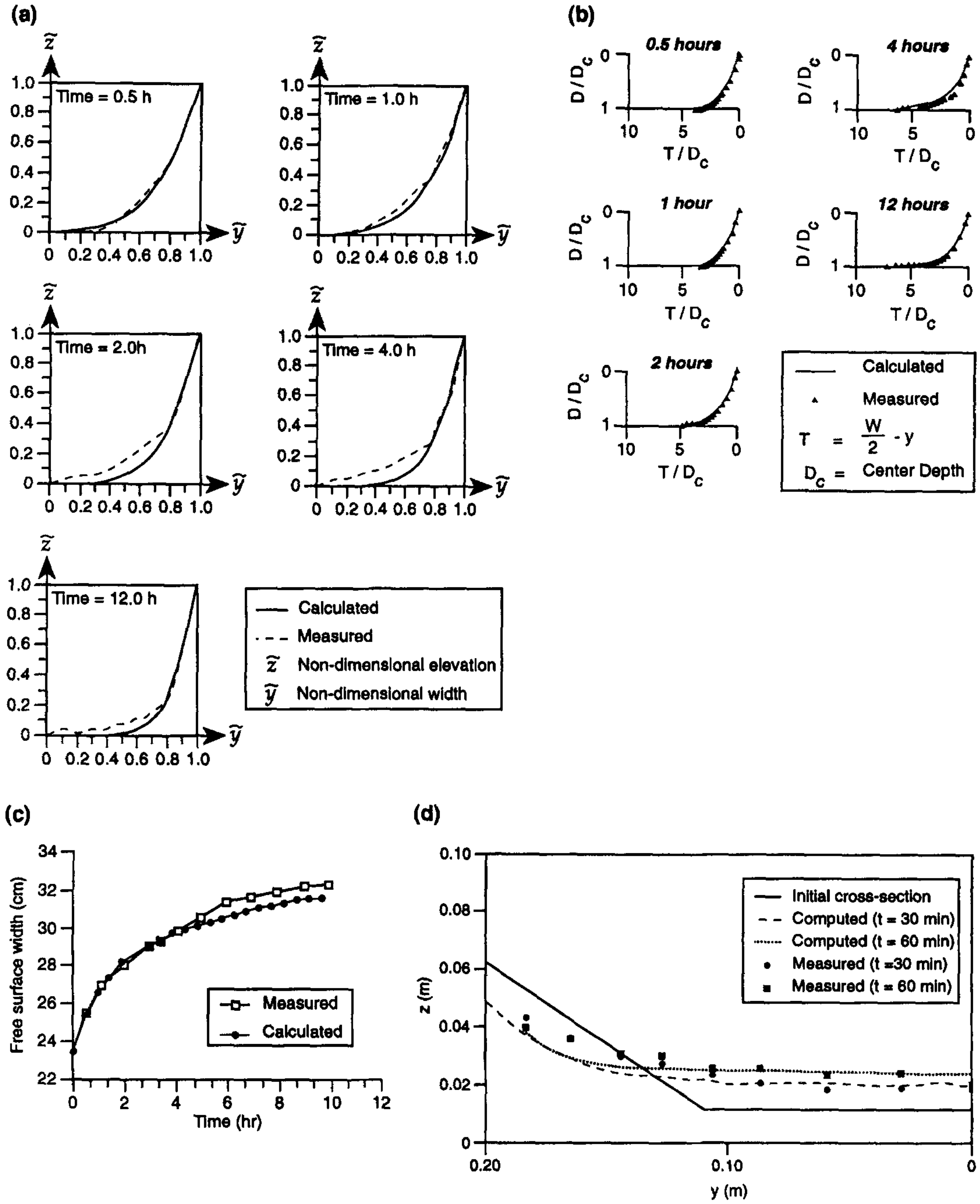

FIG. 3. Comparison of Simulated Output and Ikeda (1981) Flume Data for Models by: (a) Kovacs and Parker (1994); (b) Pizzuto (1990); (c) Wiele (1992); (d) Li and Wang (1993, 1994)

predicted cross sections with larger cross-sectional areas than those measured in reality. Pizzuto's (1990) model [Fig. 3(b)] provided close agreement between simulated and measured channel shapes throughout the extent of the simulation. Wiele's (1992) model [Fig. 3(c)] underpredicted measured widening rates, presumably reflecting the relatively high Shields stress and friction angle values selected by that author. Finally, Li and Wang (1993, 1994) obtained overpredictions of widening compared to the observed channel changes [Fig. 3(d)].

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 as 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, while 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 1996a) has been applied with unadjusted calibration parameters (Darby et al. 1996) (Fig. 4). Model calibration parameters were not adjusted from the values set during the course of model development. Although the model appeared to be able to replicate the observed sequence of channel adjustment, and overall agreement between the magnitudes of simulated and observed widths and depths were within ±10% of each other, simulated widening rates were underpredicted by a factor of 3 (Darby et al. 1996). Darby et

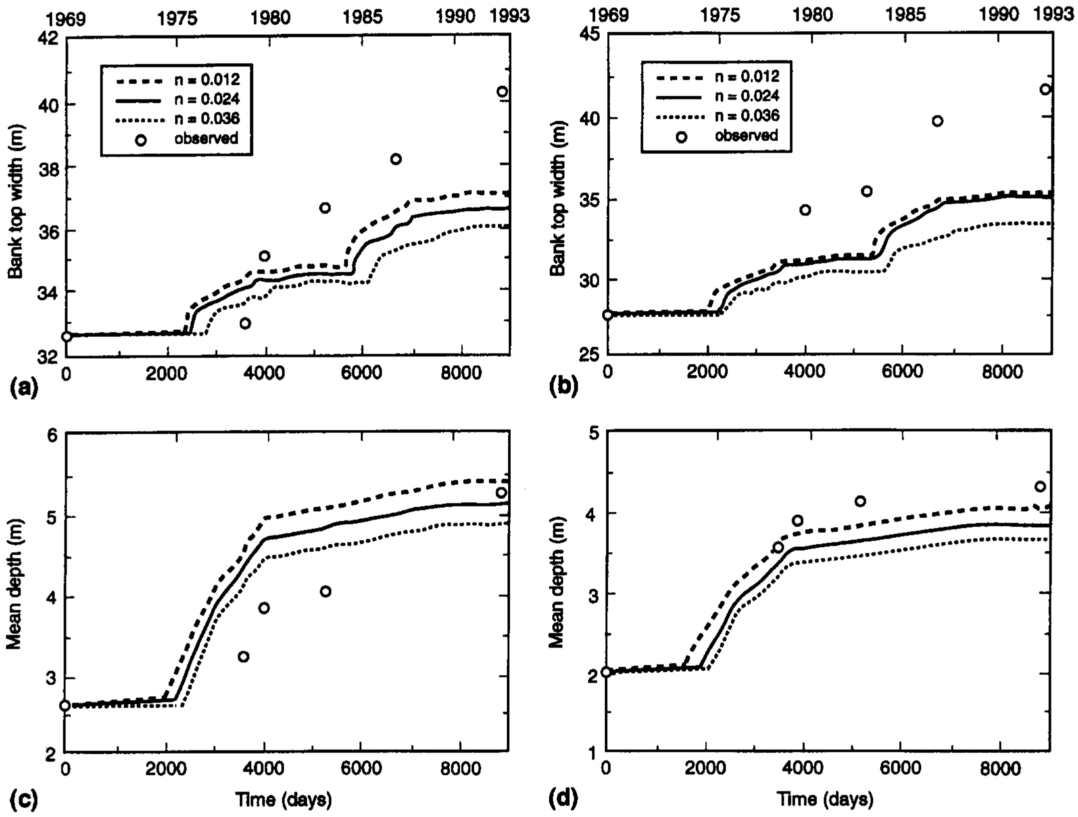

FIG. 4. Comparison of Simulated versus Observed Channel Morphology Parameters for Darby-Thorne Model at Two Study Sites in West Tennessee: (a) Bank-Top Widths at Chestnut Bluff; (b) Bank-Top Widths at Crossroads; (c) Mean Depths at Chestnut Bluff; (d) Mean Depths at Crossroads [from Darby et al. (1996)]

al. (1996) attributed this poor result to limitations with the Osman and Thorne (1988) mass-wasting algorithm (described earlier).

PROCEDURE FOR APPROACHING WIDTH ADJUSTMENT PROBLEMS

The wide range of geomorphic and engineering contexts for the treatment of problems 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. 5) is based on amassing and utilizing a wide range of information that is appropriate to the analysis of width adjustment problems. Despite each case being unique, the proposed procedure should have a number of elements that are relevant for the great majority of situations that arise.

Step 1. Problem Identification

Problem identification or formulation may be associated with a range of river engineering and societal activities. Questions to consider are

1. Does the problem arise from a natural channel 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 are affected, and what level of analysis and response is appropriate. The aim of successful problem identification is to avoid spending $200,000 on a $2,000 problem.

Step 2. Reconnaissance and Data Collection

In all cases a reconnaissance visit to the site and river reaches upstream and downstream is 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 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 the reaches that have been subject to change and reaches that are stable should be examined.

Depending on the amount of existing data available, it may be necessary to mount a specific data gathering campaign. In particular, if a numerical model is to be applied to the problem, then data collection will usually be essential. Guidance on the nature of the data that will be required for a numerical model study can be found in Appendix I.

Step 3. Desk Assessment of Equilibrium Conditions

Initially, it is recommended that simple desk methods should be used to calculate the equilibrium channel geometry. These desk methods should be based on the methods described in the section entitled Equilibrium Approaches of the companion paper. These predictions should be compared with existing conditions to provide an assessment of the current morphological status of the channel, for example, whether it is overwide,

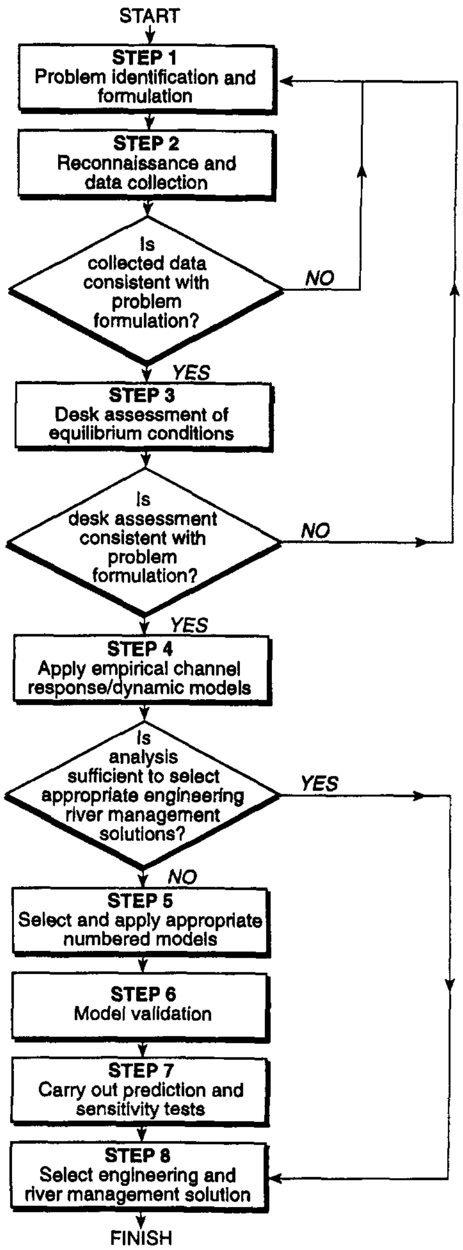

FIG. 5. Proposed Procedure for Identifying, Analyzing, and Modeling Width Adjustment Problems

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.

Step 4. Application of Empirical Channel Response or Dynamic Models

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 this paper, should be 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.

Step 5. Application of Numerical Models

If the complexity and severity of the width adjustment problem merits numerical modeling, a hierarchical modeling approach will usually be appropriate. Initially a ID 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.

Step 6. Model Validation

If possible, the numerical model results should be validated against any available field data.

Step 7. Model Prediction

The numerical models should be applied to existing conditions and also 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 those parameters that are either difficult to determine or that exhibit significant spatial or temporal variation.

Step 8. Selection of Engineering or River Management Solution

On the basis of the previous studies an appropriate plan of action should be formulated and implemented. One example of a management approach following some of the steps outlined earlier is provided by Simon and Downs (1995). They describe an interdisciplinary approach for evaluating stream channel instability conditions at the regional or state-wide scale. The region-wide 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 (7) 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.

CONCLUSIONS

From the preceding review, the conclusions are as follows:

1. Present knowledge of bank processes and flow modeling is sufficient to allow some tentative predictions of width adjustment to be made.
2. 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 in particular have the potential to become adopted as standard engineering tools.
3. Presently very few appropriate laboratory and field data sets are found to be 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.
4. At present no one universal width adjustment model ex-

ists that is applicable to all the circumstances under which width adjustments may occur.

RECOMMENDATIONS

The Task Committee (TC) recommends widespread adoption of the proposed procedure for approaching width adjustment problems (Fig. 5). In addition, the TC recommends that further research in the following topics is required to advance the capabilities of numerical width adjustment modeling. These topics are listed in order of priority as follows:

1. Comprehensive field and laboratory data is urgently required to allow detailed testing and verification of existing and future width adjustment models. A summary of the type of data required is provided in Table 6.
2. More accurate prediction of local boundary shear stress is required to better characterize sediment transport and fluvial entrainment processes in the near-bank zones.
3. Bank submodels must include more detailed representations of the process of disaggregation of cohesive bank material and the way in which such material is distributed laterally across the channel after failure of the banks.
4. Further research is required to better define the longitudinal extent of mass failures.
5. Research is required to improve our understanding of the effects of vegetation on flow and geotechnical processes, so that these effects may be included in width adjustment models.
6. Fundamental research on bank advance and channel narrowing is required. For natural river channels, such processes have been documented by geomorphologists, at least over relatively long time scales but generally only in qualitative or semiquantitative terms. Quantitative submodels of these processes do not yet exist.
7. Research is urgently required to improve the physical basis and predictive ability of existing methods of predicting the critical shear stress for entrainment of cohesive bank materials.
8. Research is required to elucidate the value of the dominant discharge or discharges that should be specified in modeling applications wherein the prototype hydrograph varies substantially through time.
9. Analyses of bank failure for a wider range of specific mechanisms of bank collapse than currently employed in width adjustment models should be incorporated in those models.
10. Research is required to investigate the significance of overbank flows and, if necessary, incorporate such effects into width adjustment models.

TABLE 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.

These research topics represent fundamental requirements for progress in the medium- to long-term. More immediately, there appears to be potential for improving current models through the continuous introduction of improved flow resistance, hydraulic, and bank mechanics submodels.

APPENDIX I. DATA SOURCES

Equilibrium Channels

Equilibrium channel geometry measurements have been reported for at least a century. A summary of published data sources has been 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 semitheoretical downstream hydraulic geometry relationships.

Brownlie (1981) published an extensive compilation of laboratory and field data. Khan (1971) reported 45 laboratory measurements of hydraulic geometry for straight, meandering, and braided sand-bed channels. The data sets from Brownlie and Khan include 73 measurements conducted in sand-bed channels under very diverse conditions ranging from small laboratory channels to very large rivers including 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 while 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.

Nonequilibrium Channels

For nonequilibrium channels, data sets that include all the parameters required to apply width adjustment models (Table 6) are comparatively rare. Laboratory experiments involving width adjustments in straight channels formed in sand were

TABLE 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 velocity	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

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 U.S. Army Corps of Engineers (1981). However, these reports do not contain all of the required data listed in Table 6. Data sets that do include many of the parameters listed in Table 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, Wash., 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 (1996a) numerical model. further information about these two data sets may be obtained through contact with the authors of these reports. Finally, data from Goodwin Creek, Miss., are available through contact with personnel at the USDA-ARS National Sedimentation Laboratory, Oxford, Miss.

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APPENDIX III. NOTATION

The following symbols are used in this paper:

h = bank height;
h_c = critical bank height for instability;
N_g = dimensionless bank stability parameter;
N_h = dimensionless fluvial stability parameter;
θ = angle of inclination of bed surface; and
ω = fraction of cohesive material present in bank material.