CHANNEL REHABILITATION DESIGN GUIDANCE MANUAL

Final Technical Report

By

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Objective

The objective of this project was to suggest revisions and to update the initial version of the DEC River Rehabilitation Manual, using the most recent findings and data from published literature.

Administrative Record

The project was initiated on 15 September, 1999. Dr N P Wallerstein was recruited by the University of Nottingham in October 1999 as the designated Research Assistant employed on the project. Detailed progress and reporting meetings of the Project team from the University of Nottingham and Colorado State University took place with Dr Biedenharn at Vicksburg in November 1999 and at Nottingham in July 2000. The Interim Report was submitted in May 2000.

Study Approach

Colin Thorne and Chester Watson acted as Principal Investigators with Nick Wallerstein responsible for day to day running of the project. After reviewing the current version of the DEC River Rehabilitation Manual and identifying areas for further enhancement, the team performed literature and archive studies and then produced a range of additional data, further examples and new text.

Initially, the document was assessed line by line and chapter by chapter. Recommendations made for revisions to the current text and for the writing of new material, additional to the original content of the document to bring the Manual up to date with current research findings and to broaden its scope beyond the DEC streams of North Mississippi.

Acting on the revisions and additions agreed in Vicksburg, in November the RA worked under the supervision of the PI at Nottingham to generate draft text, diagrams and artwork to be included in the updated manual. The draft material was passed to the co-PI at Colorado State University for editing at the end of 1999.

Draft materials and the updated manual were reviewed at a second progress meeting in Nottingham during July 2000. Modifications to draft entries were agreed and a final round of updates was initiated in light of new information available at that time.

Following the July meeting, the work necessary to complete revision of the Manual was performed by Dr Wallerstein under the guidance of the PIs. Additional text and artwork was delivered to CSU where the new material was amalgamated into the final version of the Manual. The next section details the materials added to the Manual.

Text and Materials Added under this Project

2.1.2 RIVER AND WATERSHED PROBLEMS

The key to a success rehabilitation project is to identify the causal problems. Within the watershed problems can be split into two categories, **catchment** problems and **channel** problems. These problems result in a set of impacts that act upon the channel and watershed and it is these impacts which become manifest in the field and which must be addresses by the engineer.

Catchment problems and their impacts include:

Problem	Impact
Deforestation	Upland erosion and gullying leading to increased sediment and large woody debris input to channels that may cause sedimentation and flooding. But also a more flashy runoff hydrograph which may cause catchment wide channel incision. Increase in fines load causing habitat degradation.
Intensive Agriculture	Rapid overland runoff leading to gullying and increased sediment load to the channel resulting in sedimentation
Urbanization	Rapid overland flow runoff leading to a flashier hydrograph that may result in channel incision.
Climate change	Impact depends upon the direction of climatic change. Change to a drier/colder climate may result in reduction of flood flows and reduced sediment transport. Conversely, change to a wetter/warmer climate may result in increased flooding, greater sediment transport and channel incision
Base level change	Natural base level change (Sea level or tectonic activity) may cause channel incision or aggradation and channel avulsion/adjustment of channel planform, depending upon the direction of base level change.

Channel problems and their impacts include:

Problem	Impact
Channelization / dredging / meander cutoff (both natural and man-made)	Increased flow velocity through the straightened section that may lead to channel bed incision followed by bank erosion and the loss of agricultural land, bridges, in-channel and near channel structures. Sedimentation problems downstream may affect navigation and causing flooding Destruction of in stream habitat
Dams	Direct supply reservoirs: average flows reduced, low flows more frequent flood levels reduced. Regulating/hydropower reservoirs: decreases in frequency of both low and flood flows. Both types of dam trap sediment load that can result in clear-water scour downstream of the reservoir, leading to channel degradation.
Inter-basin water transfer	Increase or decrease flow downstream of works depending on whether water is being imported or abstracted causing either greater sediment transport or reduction of flow competence and consequently sediment deposition.
Navigation	Boat wash can accelerate bank erosion and propellers increase suspension of sediment destroying habitat. Channel improvement to facilitate navigation (dredging, weir, lock and dam construction) is also detrimental to fish and invertebrate habitat.
Levees	Flood defence levees increase in-channel flows which may result in increased sediment transport causing bed and bank erosion. Downstream affect may be channel aggradation that may cause levees to be overtopped and result in channel avulsion.
Clearing and snagging	Removal of vegetation both dead and alive will reduce channel roughness, increasing flow velocities. This may reduce the flood stage but may also increase sediment transport and thus accelerate bed/bank erosion.
Gravel mining	Increases downstream suspended and bedload sediment, causing aggradation and destruction of habitat. May cause upstream degradation.
Stabilization works	Grade control and bank protection structures may themselves cause downstream sediment starvation leading to scour and channel incision.

Disturbance to a river reach as a result of changes in environmental controls usually involves continual dynamic adjustment processes that are reflected in a series of transient states. Figure 1 shows that, in addition to progressive changes, **stepped** and **impulse** influences and associated responses may occur. The timescales of these adjustments reflect the '**relaxation time**' of the river reach in question.

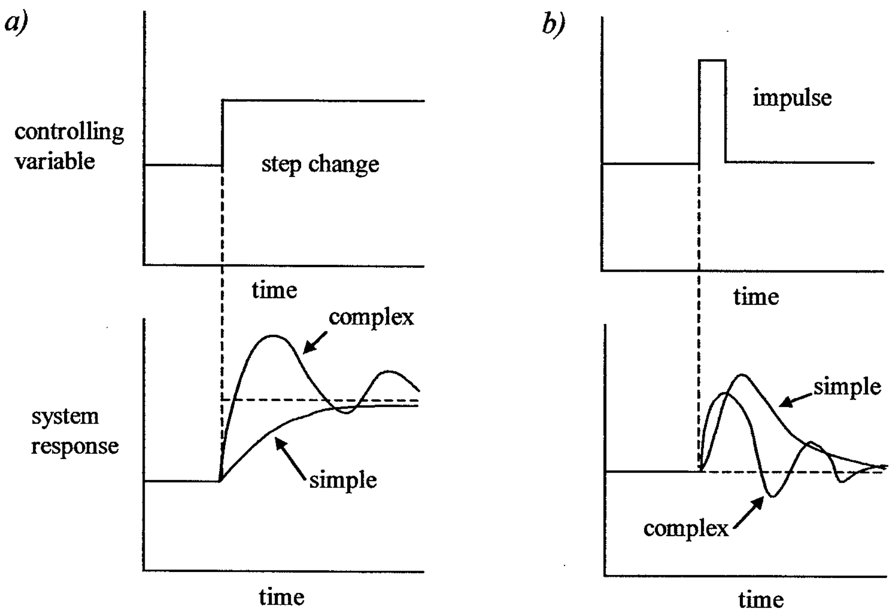

Figure 1. Types of externally imposed change, and responses: (a) shows possible responses to a step-function change, and (b) to a single impulse. Modified from Thorne et al. (1997)

The dynamics of channel change arise because of the feedback that occurs amongst the channel shape, the flow pattern, and the processes of sediment transport. Hey (1979) describes this qualitatively for the one-dimensional (long-profile) case, arguing that an external stimulus such as a sudden lowering of the base level would be transmitted upstream, and that each location along the affected reach would experience alternate phases of degradation and aggradation of decreasing amplitude. This model was quantified by Pickup (1975, 1977) see Thorne et al., (1997, chapter 10). The limitation of a model such as Pickup's is that it assumes that the morphological adjustments occur only in terms of long-profile change, and that channel banks remain stable. However, it is likely that vertical instability will cause bank erosion and therefore generate channel widening. When an additional bed material load supply is produced by bank erosion during bed degradation the problem becomes two-dimensional, and after knickpoint recession the equilibrium gradient may be steeper than the initial gradient because of adjustment of channel width.
The basic model employed by Pickup (1975, 1977) can also be applied to the plan form adjustment of a river reach. Figure 2 suggests a qualitative feedback model applicable to coarse-grained river beds, in which the topographic changes of the bed are primarily controlled by bedload movement, and suspended bed material load is of relatively minor significance for the topography.

However successful analytical or numerical modelling of both the transient and equilibrium channel responses to disturbance awaits physically realistic models of bank erosion and width adjustment processes.

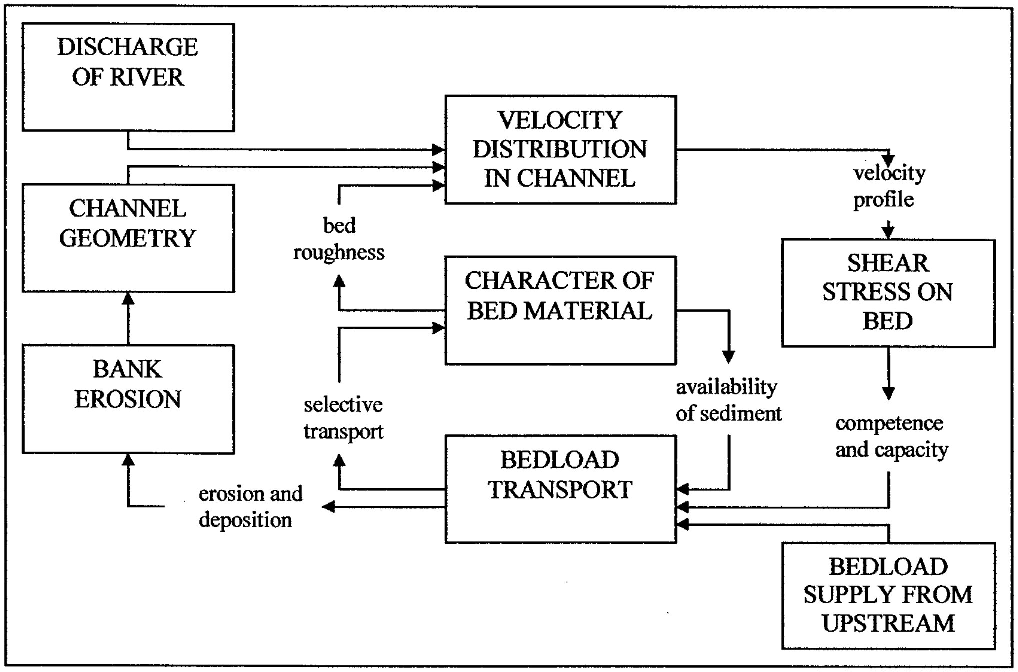

Figure 2. A feedback model of channel change: initial form, flow pattern, transport pattern and sediment mass conservation determine patterns of channel bed change. Modified from Thorne et al. (1997).

Fluvial processes appear to be inherently **non-linear**, and to involve discontinuous responses to change in control variables. Schumm (1973) argued that drainage basins exhibited both '**extrinsic**' thresholds and '**intrinsic**' thresholds. An example of an extrinsic threshold is the threshold of motion for bed material movement initiated when a critical flow shear stress is exceeded. Schumm (1973) discovered an intrinsic threshold while performing drainage basin flume experiments. This example occurred when sediment deposition downstream of a knickpoint continued for a sufficient period of time that the gradient of the aggrading surface became steep enough to trigger a new phase of incision. The threshold slope-discharge curve between meandering and braided rivers is also an example of an intrinsic threshold.
Because of the presence of intrinsic thresholds in is possible that adjustment of the fluvial system may be catastrophic in nature. The theory of 'cusp catastrophe' has been developed by Graf (1988). If is simple system is defined by two control variable (a and b) which represent a force and a resistance variable and a response variable c then these are related by an energy function E(a, b, c). For each a-b pair a value of c minimises E. A map of the system in equilibrium (where change in E is minimised) defines a three dimensional surface which may have a fold in it so that for certain combinations of a-b, c may have more than one value which satisfies the minimum energy criterion. The relationship between stream power (variable a), bank erosion (variable b) and sinuosity (variable c) may be represented in this way (Figure 3). Cusp

catastrophe demonstrates that quite divergent responses may be made by two systems undergoing changes (paths A to B_1 and B_2).
Thus multiple modes of channel adjustment may occur as a result of a single disturbance in the system.

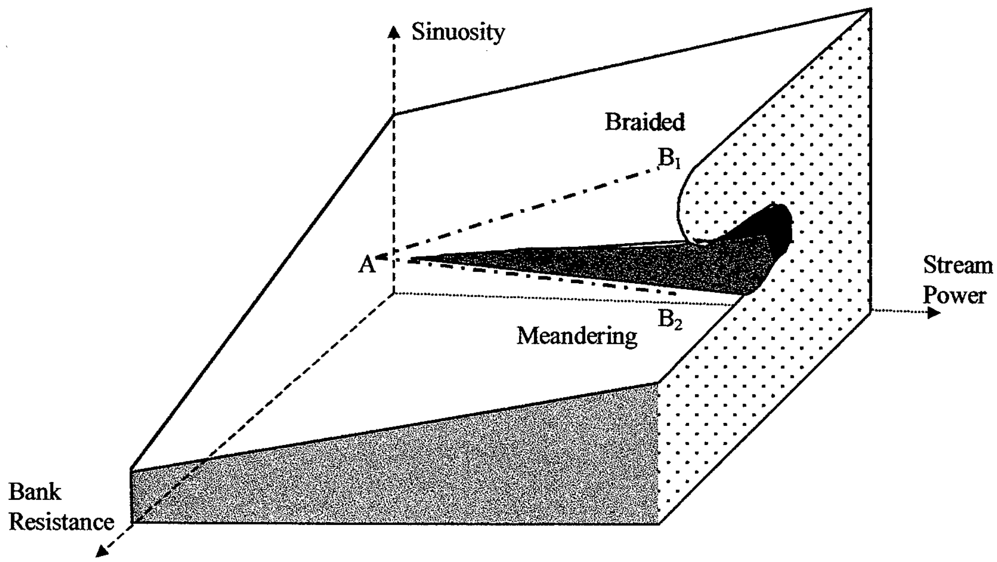

Figure 3. A qualitative model of non-linear change in equilibrium channel pattern between meandering and braided, demonstrating the theory of cusp catastrophe. Modified from Thorne et al. (1997).

Phillips (1992) argued that geomorphic systems and fluvial systems in particular might in actuality behave as **Nonlinear Dynamical Systems (NDS)**.
NDS has four characteristics that distinguish it from other types of approach. These are:

a) It is concerned with evolution of systems between equiliria;
b) Systems involved are dissipative structures;
c) System evolution is characterised by discontinuities such as bifurcations and catastrophes, and;
d) There is the possibility of deterministic chaos.

The concept of equilibrium is fundamental to geomorphology. Hack (1960) stated that a landscape is always moving towards a dynamic equilibrium, where forms are adjusted to the prevailing processes and environmental controls. But, viewed under the NDS approach geomorphic systems never approach a single equilibrium, but instead alter through abrupt changes of state (crossing thresholds).

Dissipative structures exist in any open system where processes are irreversible, i.e. where there is entropy. This obviously applies to geomorphic systems. Fluvial systems are therefore dissipative as energy is dissipated in maintaining order in states removed from equilibrium (for example consistent bedform states associated with changing hydraulic regimes).

NDS exhibit **bifurcations** which are analogous to thresholds in geomorphic systems (for example the threshold of sediment motion). Change between states may therefore be catastrophic rather than progressive (refer to Figure 3).

Chaos is linked to asymptotic instability which is defined as an exponential divergence from an equilibrium state following disturbance. Examples of this have been found in hydraulic geometry (Slingerland, 1981), and in sediment transport systems (Rhoads, 1988). Until recently geomorphic systems have been generally regarded as stochastic, so that if enough data could be gathered on them they could be understood deterministically. This may not be the case. If chaos is present it implies that at least long-term prediction is impossible and questions the concept of equifinality (the argument that landscapes tend toward the same final form under the same geomorphic controls).

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Streambed Erosion Mechanisms

The discussion of streambed erosion processes discussed below is extracted from Simons and Senturk (1992).

It is important to realise that all materials in a streambed will erode if subjected to sufficient stresses for long enough. Some materials such as granite may take hundreds of years to erode while sand-bed rivers may erode to the maximum depth of scour in a matter of hours.

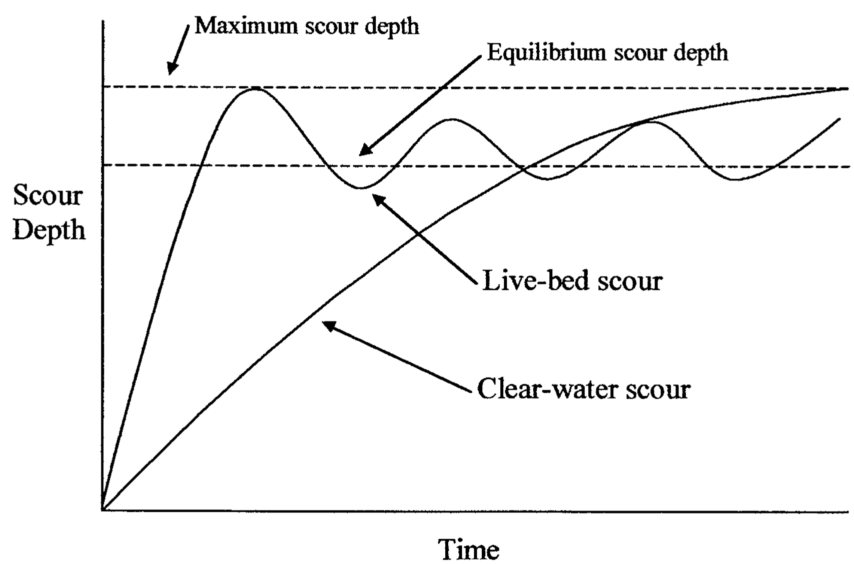
Total scour is composed of three components: Long-term Degradation; General Scour and Contraction Scour; and Local Scour.

Degradation is defined as the erosion of a streambed over an extended reach or throughout the watershed network over a long period of time and may be the natural trend of the stream or may be the result of some modification of the channel or the watershed. Degradation is not reversible in the short term. Degradation is often manifest by the presence of a **Knickpoint** or **Knickzone**, a distinct step or series of drops in the channel bed long profile, which migrates upstream as flow acceleration over the drop erodes the upstream face of the bed. Degradation can be initiated by factors such as sediment starvation downstream of dams, channelization, cutoff of meander bends, reduction of downstream base level either due to river dredging or through tectonic activity and diversion of water into a stream. Essentially any process that causes an increase in long term average shear stresses or which reduces the resisting frictional forces along an extended channel reach will initiate a degradation wave to move upstream.

General Scour can be caused by a decrease in channel width, either naturally or by man, which decreases flow area and increases velocity. This is also termed **Contraction Scour**. General scour can also be caused by short-term changes in the downstream water surface elevation that controls the backwater and hence the velocity through a contracted reach. This type of scour is reversible in the short term (daily, weekly, yearly or seasonally). General scour can be cyclic. That is, during a runoff event the bed scours during the rising stage and fills during the falling stage. General scour from a contraction occurs when the flow area of a stream is decreased from the normal either by a natural constriction or by a structure. With the decrease in flow area there is an increase in velocity and hence in stream power at the contraction and more bed material is transported through the contracted reach than is transported into the reach. The increase in transport of bed material lowers the bed elevation. As the bed elevation is lowered, the flow area increases and the velocity and shear stress decrease until equilibrium is once more reached. Factors that can cause contraction scour include: a natural stream constriction; ice formation or jams; berm formation along river banks; island or bar formation; debris; bridge abutments and piers; local levees; and the growth of vegetation in the channel.

Local Scour occurs when flow accelerates past or is forced to change direction around a discrete object in the flow such as a boulder, abutment or piling. Local scour does not extent to the reach scale and is often temporary or cyclic in nature. The obstruction in the flow causes the formation of a vortex at its base which results from the pileup of water on the upstream face and subsequent acceleration of the flow around the nose of the object. The action of the vortex is to remove bed material away from the base region and to dump it downstream. If the transport rate of sediment away from the local region is greater than the transport rate into the region, a scour hole forms. As with general scour, as the depth of scour increases, the strength of the vortex diminishes, the transport rate is reduced, and equilibrium is re-established. A common location of local scour is a the base of bridge piers. In this instance local scour is affected by the following factors: width of the pier; length of the pier; depth of flow; approach flow velocity; size of bed material; angle of approach flow to the pier; shape of the pier; bed forms; and ice and debris build-up around the pier.

There are two conditions of local scour. These are **Clear-Water Scour** and **Live-Bed Scour**. Clear-water scour occurs when there is no movement of bed material upstream of the object but the acceleration of the flow and vortices created by the object causes the material at it's base to move. Live-bed scour occurs when the bed material upstream of the object is also moving. Clear-water scour reaches its maximum over a longer period of time than live-bed scour (Figure *1*). The reason for this is that clear-water scour occurs mainly on coarse bed-material streams. In fact, clear water scour may not reach its maximum until after several flood events. Maximum clear water scour has been found to be about 10 percent greater than maximum live-bed scour. Live-bed scour in sand-bed streams with a dune bed configuration fluctuates about an equilibrium scour depth (Figure *1*). The reason for this is the fluctuating nature of sediment transport upstream of the object when there are dune formations. In this case maximum depth of scour has been found to be about 30 percent larger than the equilibrium depth of scour. If there is a plain bed with live-bed scour maximum scour is equal to equilibrium scour, while if there are antidunes with live-bed scour maximum scour depth has been found to be about 20 percent greater than equilibrium scour depth.



The Pool-Riffle Sequence

Pool-riffle sequences are characteristic of gravel and mixed load rivers of moderate gradient (<5 %) (Sear, 1996). Riffles are topographic high points in an undulating bed

long profile and pools are low points. Typically sediment grain size is coarser in riffles than in pools. The mechanism that has been evoked to explain this is as follows: Fine sediment is removed from riffles at low flows into pools because shear stress is at a maximum over riffles (Keller, 1971). As discharge rises there is thought to be a reversal of shear stress, whereby near-bed shear stress in the pool exceeds that over the riffle. However, evidence for this mechanism is conflicting. Ashworth (1987), Clifford (1990) and Petit (1987) have measured a shear stress reversal, while other studies indicate velocity equalisation at bankfull flows (Lisle, 1979; Carling, 1991). Sediment transport in pool-riffle sequences as envisaged by Iseya and Ikeda (1987) is shown in Figure 2.
Yalin (1971) suggests that turbulence generated at the boundary of a straight, uniform channel would produce large-scale eddies associated with alternate acceleration and deceleration of flow. The spacing between fast and slow zones is shown to average $2\pi w$ (w = width) (refer to figure 3) which is consistent with field observations of pool-riffle spacing at 5 to 7 channel widths (Keller and Melhorn, 1973).

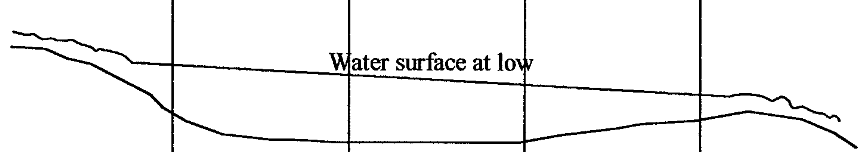
Bedform	Riffle	Pool-Head	Mid-Pool	Pool-Tail	Riffle
Longitudinal View					
Bed State	Congested	→	Smoothing	→	Congested
Sedimentary Structure Of Surface	Tightly packed. High frequency of particles in stable structures. Armoured. Open-work.	→	Loosely packed. High frequency of particles inn unstable positions in bed. Armoured. Increasing matrix.		
Surface D_{50}	●	●	●	●	●
Entrainment Threshold	High	Decreasing →		Low	High
Distrainment Opportunity	High	Decreasing →		Low	High
Bed Slope	+ High	Gentle →		- ive	+ High
Bedload Balance	Aggrading		Degrading →		Aggrading
Relative Exposure D_{50} Riffle Particle	Low	Increasing →		High	Low

Figure 2. Sediment transport processes in a pool-riffle sequence. Modified from Iseya and Ikeda (1987).

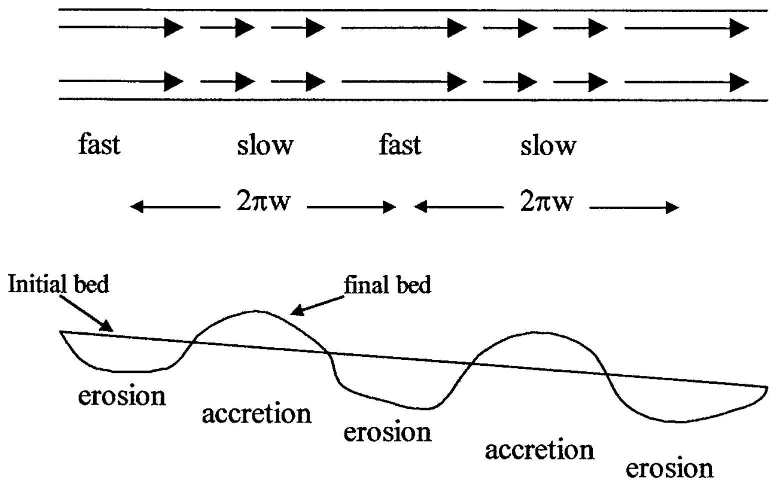


Figure 3. Erosion and accretion of a stream bed caused by alternate zones of fast and slow flow (modified from Richards, 1982)

The topography of a riffle-pool sequence is also maintained by the patterns of secondary circulation. In pools flow diverges at the bed, while in riffles flow converges at the bed. This has the effect of lifting shear stress from the bed in a riffle, which favours deposition, while flow divergence at the bed in pools increases shear stress, causing erosion (refer to Figure 4).

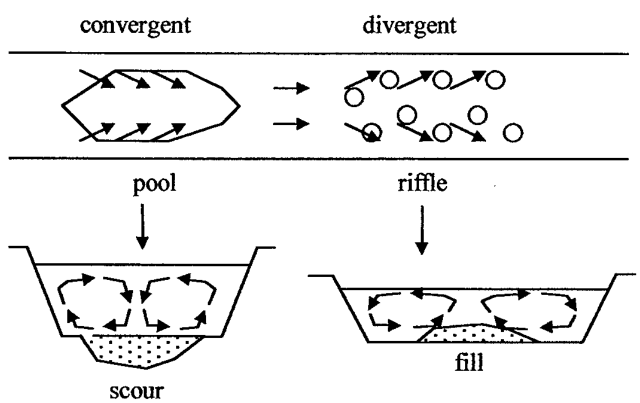


Figure 4. Secondary circulation in a pool riffle sequence and it's effect upon bed shear stress and consequently scour and deposition (Modified from Richards, 1982).

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Local Channel Morphological Variability: Empirical Equations

Meander Planform Variability

Equations for meander planform variability developed by various authors are given in section 3.1.3.3.
These equations present functions in terms of;
(1) Meander wavelength as a function of channel width;
(2) Meander wave length as a function of meander bend radius of curvature;
(3) Meander wavelength as a function of the mean annual flood;
(4) Meander wavelength as a function of average annual flood and the percentage of silt and clay in the channel boundary;
(5) Meander amplitude as a function of channel width;
(6) Meander width-depth ratio as a function of the percentage of silt and clay in the channel boundary, and;
(7) Meander bed slope as a function of mean annual discharge and the percentage of silt and clay in the channel boundary.

These equations can be used as the basis for restoring a straightened channel reach to a meandering course that mimics the planform of a natural river. However, it is important that the river engineer refer to the original references for these equations to determine whether they are applicable to the channel environment in question.

Meander Width Variability

In natural, stable channels, there may be considerable variation in bankfull width between meander crossings, bend apices, the width at the greatest scour location (found within the pool, usually offset downstream from the apex) and meander riffles. In contrast, width is usually not permitted to vary in two and quasi-three dimensional computer models that route flow and sediment through sinuous channels. This is due to an incomplete understanding of the governing processes that control width adjustment in non-straight alluvial channels.

Without a sound analytical technique to address the problem of planform width variation, empirical methods may be used to solve this neglected degree of freedom for different types of meander bends found in nature.

A summary of results by Thorne and Soar (1999) are presented here which have been derived from the 1980-1981 hydrographic survey data from the Red River between Index, Arkansas, and Shreveport, Louisiana, which is considered to be a typical alluvial river with a range of meander bend types and predominantly sand bed material. The initial data set was compiled by Thorne and Abt (1993) for a study of bend flow and maximum scour depth prediction, for which widths at meander crossings, W_x, were measured. For each meander bend in the data set, the width at the bend apex, W_a, width at the maximum scour location, W_s, and the offset distance between the bend apex and maximum scour location, or pool offset, $L_{a\text{-}s}$, were measured.

It is assumed that planform width variability found in natural channels is primarily a function of the complex flow patterns found in meandering rivers, rather than morphological variables such as boundary sediment type. This assumption is considered a fair statement on the basis that the general qualitative trends of planform width variability tend to hold true for different types of meandering channel with different boundary conditions.

Based on Thorne and Soar's (1999) thorough review of the literature on stream classification and typology, three types of meander bend were considered to be representative of the general types found in naturally stable meandering channels and may be considered as target channel types for river restoration design. A summary of this tripartite typology is given below.

i) *Equiwidth Meandering (e-type)*

Equiwidth refers to only minor variability in channel width around meander bends. These channels are generally characterised by: low width/depth ratios; erosion resistant banks; fine-grain bed material (sand or silt); low bed material load; low velocities, and; low stream power. Channel migration rates are relatively low because the banks are naturally stable.

Equiwidth meanders are denoted as 'Type-e' meanders, or 'T_e', in the equations presented below.

ii) *Meandering with Point Bars (p-type)*

Meandering with Point Bars refers to channels that are significantly wider at bendways than crossings, with well developed point bars but few chute channels. These channels are generally characterised by: intermediate width/depth ratios; erosion resistant banks; medium grained bed material (sand or gravel); medium bed material load; medium velocities, and; medium stream power. Channel migration rates are likely to be moderate unless banks are stabilized.

Meanders with point bars are denoted as 'Type-p' meanders, or 'T_p', in the equations presented below.

iii) *Meandering with Point Bars and Chute Channels (c-type)*

Meandering with Point Bars *and* Chute Channels refers to channels that are very significantly wider at bendways than crossings, with well developed point bars *and* frequent chute channels. These channels are generally characterised by: moderate to high width/depth ratios; highly erodible banks; medium to coarse grained bed material (sand, gravel and/or cobbles); heavy bed material load; moderate to high velocities, and; moderate to high stream power. Channel migration rates are likely to be moderate to high unless banks are stabilized.

Meanders with point bars and chutes are denoted as 'Type-c' meanders, or 'T_c', in the equations presented below.

Three morphological variables are defined as being important for the correct design of meander bends. These are:

1) Bend apex width

2) Width at the maximum scour location
3) Pool offset ratio (distance between bend apex and maximum pool depth)

The ratio of bend width and width at maximum scour location to crossing width at the 2-year flow stage are presented in equations 1 and 2. The ratio of distance from bend apex to maximum scour width to distance from bend apex to downstream crossing at the 2-year discharge is presented in equation 3:

Bend Apex (for P>=1.2):

$$\frac{W_a}{W_x} = 1{\cdot}05T_e + 0{\cdot}30T_p + 0{\cdot}44T_c \pm u \qquad (1)$$

Maximum Scour Location (for P>=1.2):

$$\frac{W_s}{W_x} = 0{\cdot}95T_e + 0{\cdot}20T_p + 0{\cdot}14T_c \pm u \qquad (2)$$

Pool offset (for P>1.0):

$$\frac{L_{a\text{-}s}}{L_{a\text{-}x}} = 0{\cdot}36 \pm u \qquad (3)$$

where, P = channel sinuosity, W_a = bend apex width, W_x = crossing width, W_s = width at maximum scour location, $L_{a\text{-}s}$ = distance from bend apex to maximum scour location, $L_{a\text{-}x}$ = distance from bend apex to downstream crossing, T_e = e-type bend, T_p = p-type bend, T_c = c-type bend, and u = confidence limits of the mean response and confidence limits of an individual response (prediction limits). Values of u are given as:

		Ratio		
Confidence Limits (%)		W_a / W_x	W_s / W_x	$L_{a\text{-}s} / L_{a\text{-}x}$
Mean Response*	99	0.07	0.17	0.11
	95	0.05	0.12	0.08
	90	0.04	0.10	0.07
Individual Response*	99	0.31	0.76	1.00
	95	0.22	0.53	0.75
	90	0.17	0.43	0.63

* mean response = The confidence interval on the mean response describes upper and lower bounds, or *limits*, on the *expected value* of the dependent variable with 100(1-α)% confidence. With repeated sampling from a population consisting of a specific river *type*, these confidence limits will contain the regression line or regression surface for each data set at the required level of fixed statistical probability, α.

* single response = The confidence interval on a single response, or *prediction interval*, describes upper and lower bounds, or *limits*, on a future *single value* of the dependent variable with 100(1-α)% confidence. With repeated sampling from a population consisting of a specific river *type*, these confidence limits will contain the spread of data about a regression line or regression surface for each future data set at the required level of fixed statistical probability, α. The confidence interval on a single response is significantly wider than the mean response interval, reflecting the additional variance of the sample data.

In equations 1 and 2, the additive effects of 'e-type', 'p-type' and 'c-type' bends are represented by the binary parameters, T_e, T_p and T_c, respectively. The value of T_e has a value of '1' for all three types of bend and represents the smallest planform width ratio. If point bars are present but chute channels are rare, then T_p is assigned a value of '1' and T_c is assigned a value of '0'. If point bars are present and chute channels are common, then both T_p and T_c are assigned values of '1'. Obviously T_c can only be given a value of '1' when T_p has a value of '1'.

Pool-Riffle Geometry Variability

It has been noted that the pool-riffle sequence in straight channels is spaced fairly evenly along the channel. Leopold et al. (1964) suggested that riffle spacing was 5 to 7 times the channel width. Hey and Thorne (1986), working in British rivers with a mixture of straight, sinuous and meandering planforms found that riffle spacing (measured along the channel centreline) could be defined by:

$$z = 6.31w \qquad r^2 = 0.88 \qquad (4)$$

where, z = riffle spacing (m) and w = bankfull width (m).
Local width and depth variability associated with riffles has also been characterised by Hey and Thorne (1986) in the form of modifications to the equations defining the stable hydraulic geometry of a channel:

$$R_w = 1.034w \qquad r^2 = 0.97 \qquad (5)$$

$$R_d = 0.951d \qquad r^2 = 0.97 \qquad (6)$$

$$R_{dm} = 0.912d_m \qquad r^2 = 0.96 \qquad (7)$$

$$R_v = 1.033v \qquad r^2 = 0.92 \qquad (8)$$

where, R_w = riffle bankfull width (m), w = channel bankfull width (m), R_d = riffle bankfull mean depth (m), d = channel bankfull mean depth (m), R_{dm} = riffle bankfull maximum depth (m), d_m = channel bankfull maximum depth (m), R_v = riffle bankfull mean velocity (m/s), and v = channel bankfull mean velocity (m/s). These relationships show that, morphologically, riffles are a little shallower and wider than the average dimensions of the channel. This variability is much greater at low flows but the difference between pools and riffles decrease as flow stage increases and probably disappears at bankfull flow.

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NUMERICAL MODELS FOR RIVER CHANNEL ADJUSTMENT

Numerous 1D, 2D and 3D numerical models have been developed in recent years to describe open channel flow with sediment transport and bank retreat. Twelve of these models were reviewed by Darby (1998). The main characteristics of these models in terms of flow routing, sediment routing and bank retreat algorithms are presented in Tables 1, 2 and 3. Numerical models create conceptions of physical reality that result in quantitative predictions. Given the complexity of natural systems, the gaps in our knowledge, the modelling 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 of subsequent efforts. Potential users must judge if in fact the model will solve a particular problem.

Table 1. Features of Flow Routing Sub-models in Reviewed Models

Model	Dimension	Discharge variation over time	Secondary flow	Lateral shear	Friction factor	Flow resistance formula[b]
Darby-Thorne[1]	quasi 2D[a]	Stepped hydrograph	No	Yes	Time and space variable	Strickler
CCHEBank[2]	3D	Unsteady flow	Yes	Yes	Constant	Keulegan
Kovacs-Parker[3]	2D	Steady flow	No	Yes	Constant	Keulegan
Wiele[4]	2D	Steady flow	No	Yes	Constant	Keulegan
RIPA[5]	2D	Stepped hydrograph	Yes	No	Constant	Specified
Simon et al.[6]	quasi 2D[a]	Stepped hydrograph	No	No	Time and space variable	Strickler, Darcy, and Chezy
Pizzuto[7]	2D	Steady flow	No	Yes	Constant	Einstein
STREAM2[8]	1D	Stepped hydrograph	No	No	Constant	Specified
GSTARS[9]	quasi 2D[a]	Stepped hydrograph	No	No	Time and space variable	Strickler, Darcy, and Chezy
FLUVIAL-12[10]	1D	Unsteady flow	Yes	No	Time and space variable	Strickler and Brownlie
Alonso-Combs[11]	1D	Stepped hydrograph	No	No	Constant	Specified
WIDTH[12]	1D	Stepped hydrograph	No	No	Time and space variable	Strickler

Note: Strickler = Strickler (1923); Keulegan = Keulegan (1938); Einstein = Einstein (1950); Brownlie = Brownlie (1983).

[a] quasi 2D 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 effect of bed forms.
[1] Darby & Thorne (1996); [2] Li & Wang (1993); [3] Kovacs & Parker (1994); [4] Wiele (1992); [5] Mosselman (1992); [6] Simon et al. (1991); [7] Pizzuto (1990); [8] Borah & Bordoloi (1989); [9] Yang et al. (1988); [10] Chang (1988); [11] Alonso & Combs (1986); [12] Osman (1985).

Table 2. Features of Sediment Routing Submodels in Reviewed Models

Model	Routing method	Steam-wise flux difference	Trans-verse flux difference	Bed load	Susp-ended load	Transport equations	Sorting	Bed material
Darby-Thorne	quasi 2D	Yes	Yes	Yes	Yes	Engeland & Hansen (1967)	Yes	Sand
CCHEBank	2D	Yes	Yes	Yes	No	Meyer-Peter & Muller (1948)	No	Gravel
Kovacs-Parker	2D	No	Yes	Yes	No	Kovacs & Parker (1994)	No	Gravel
Wiele	2D	No	Yes	Yes	No	Parker (1979) & Meyer-Peter & Muller	No	Sand & gravel
RIPA	2D	Yes	Yes	Yes	No	Egelund & Hansen (1967) & Meyer-Peter & Muller (1948)	No	Sand & gravel
Simon et al.	quasi 2D	Yes	No	Yes	Yes	Yang (1973), 1984), Ackers & White (1973), Engelund & Hansen (1967)	Yes	Sand & gravel
Pizzuto	2D	No	Yes	Yes	No	Parker (1983)	No	Sand
STREAM2	1D	Yes	No	Yes	Yes	Yang (1973), Graf (1971) & Meyer-Peter & Muller (1948)	Yes	Sand & gravel
GSTARS	quasi 2D	Yes	No	Yes	Yes	Yang (1973, 1984), Ackers & White (1973), & Engelund & Hansen (1967)	Yes	Sand & gravel
FLUVIAL-12	1D	Yes	No	Yes	Yes	Yang (1973), Paker et al. (1982), Ackers & White (1973), Engelund & Hansen (1967) & Graf (1971)	Yes	Sand & gravel
Alonso-Combs	1D	Yes	No	Yes	Yes	Alonso et al. (1981)	Yes	Sand & gravel
WIDTH	1D	Yes	No	Yes	Yes	Engelund & Hansen (1967)	No	Sand

Table 3. Features of bank mechanics sub-models of reviewed models

Model	Bank Process				Bank Material			
	Depos-ition	Fluvial entrain-ment	Types of bank failure	Longitud-inal extent of failure	Cohesive	Nonco-hesive[a]	Layer-ed	Hetero-geneous
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 FLUVIAL-12	Bank mechanics submodels are not included in these models, which are instead based on extremal hypotheses							
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

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The Role of Pore Water Pressure

An important consideration in bank stability analysis is the role of pore water pressure. The following discussion of this component of bank stability is taken from Casagli et al. (1999).

Pore water pressure is defined as the pressure of water filling the voids between solid particles. In a fully saturated soil all voids are filled with water and, under these conditions, the effective stress (σ') is given by the difference between the total normal stress and the pore water pressure (u_w). In partially saturated soils the voids are occupied by air and water and pore water pressure is less than pore air pressure (u_a) due to surface tension at the air-water interface. In this condition pore water pressure above the water table is negative and the difference between the two quantities is ($u_a - u_w$) is defined as matric suction.

Matric suction increases the shear strength of an unsaturated soil. In this case the failure criterion can be expressed in terms of normal stress (σ - u_a), and the matric suction ($u_a - u_w$) in an equation of the form:

$$\tau = c' + (\sigma - u_a) \tan \phi' + (u_a - u_w) \tan \phi^b \qquad (1)$$

where τ = shear strength, c' = effective cohesion, ϕ' = friction angle in terms of effective stress and ϕ^b = angle expressing the rate of increase in strength relative to matric suction.

This criterion represents an extension of the Mohr-Coulomb failure envelope in terms of effective stress because as the soil approaches full saturation the pore water pressure tends to equal to pore air pressure and the suction component disappears. The extended Mohr-Coulomb failure envelope for an unsaturated soil can be plotted in a three-dimensional diagram with the shear stress and the two stress state variables on the three axes. The extended failure envelope can be presented as a projection onto the shear stress versus (σ - u_a) plane. For a given matric suction value, the projection is a line given by the equation:

$$\tau = c_a + (\sigma - u_a) \tan \phi' \qquad (2)$$

where c_a = total cohesion intercept, which incorporates the effects of the matric suction and of the effective cohesion:

$$c_a = c' + (u_a - u_w) \tan \phi^b \qquad (3)$$

Pore water pressure distribution within a soil also controls the seepage both in the saturated and unsaturated zone. In unsaturated flow the permeability is a function of the degree of saturation, and the effects of matric suction have to be included in the expression of hydraulic head, by considering the negative pressure head (matric head) h_m:

$$h_m = -\frac{(u_a - u_w)}{\gamma_w} \qquad (4)$$

where γ_w = unit weight of water
Total head h is the sum of elevation head and matric head:

$$h = z - \frac{(u_a - u_w)}{\gamma_w} \tag{5}$$

Apparent cohesion due to matric suction can represent a large component of a river banks' total shear strength. Thus, if a bank is composed of fine material the high shear strength term due to matric suction allows the bank to remain stable during low-flow periods at angles much higher than the effective friction angle. During high flow periods or during heavy rainfall water content in a bank will rise as will pore water pressure so that apparent cohension may disappear altogether and positive pore water pressures occur. In these circumstances it might be expected that the bank will fail, but stability can be maintained by the confining pressure of the high flow. Bank failure is therefore most likely as high flows recede and the bank material is still at or near saturation.

Simon et al. (1999) also demonstrate how matric suction increases the resistance of banks to mass failure as well as failed cohesive blocks to entrainment by fluid shear. They have found that a stable bank is transformed into an unstable one during periods of prolonged rainfall through;

(1) increase in soil unit weight;
(2) decrease of matric suction and therefore apparent cohesion;
(3) generation of positive pore water pressures and therefore loss of frictional strength;
(4) entrainment of insitu and failed material at the bank toe, and;
(5) loss of confining pressure during recession of the storm hydrograph.

They conclude that it is rapid drawdown conditions which represent the dominant condition for bank mass failure, and also note that it is not the largest storms and greatest floods that induce bank failures but prolonged wet periods which weaken *in situ* bank materials.

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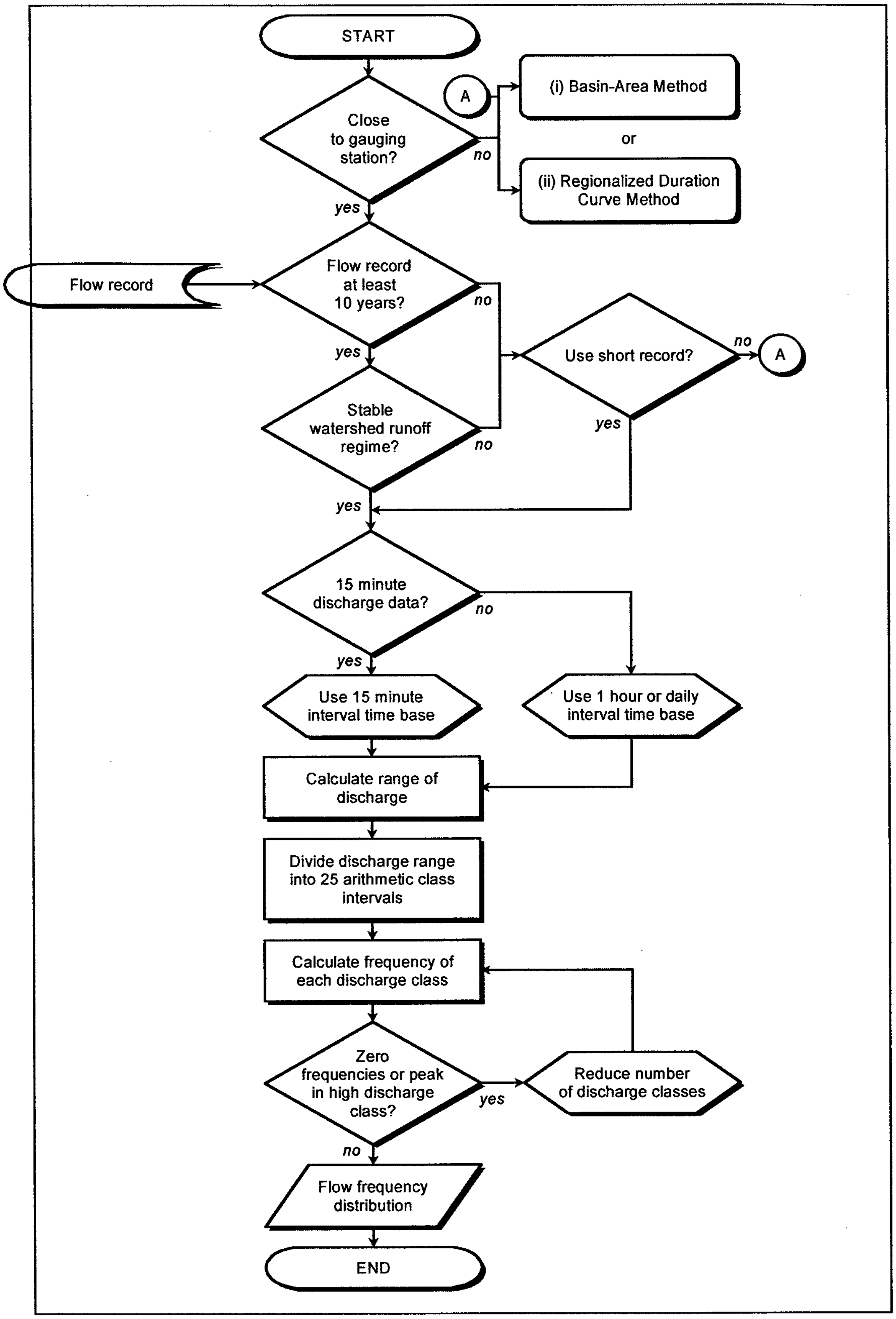
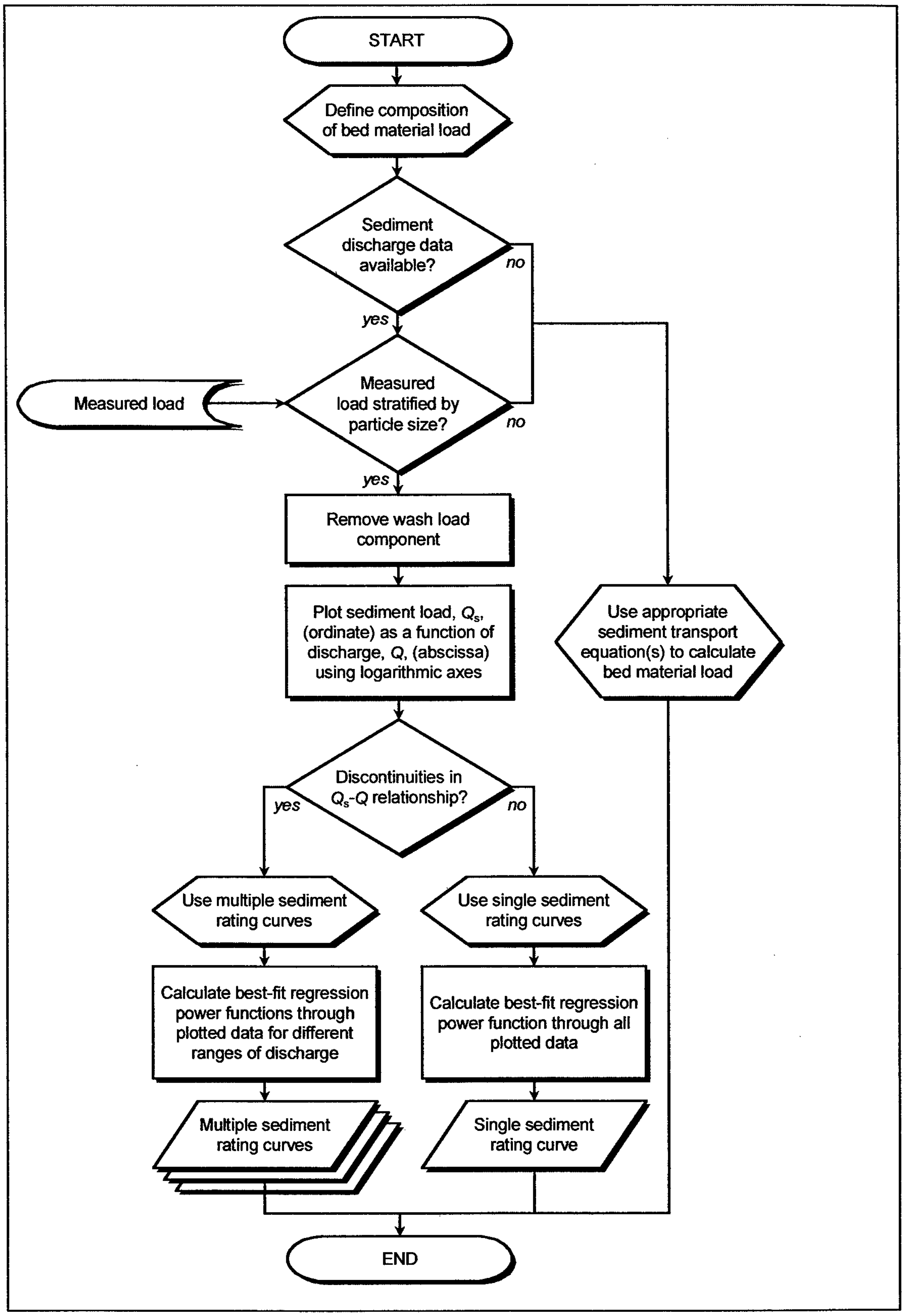
Calculation of Effective Discharge

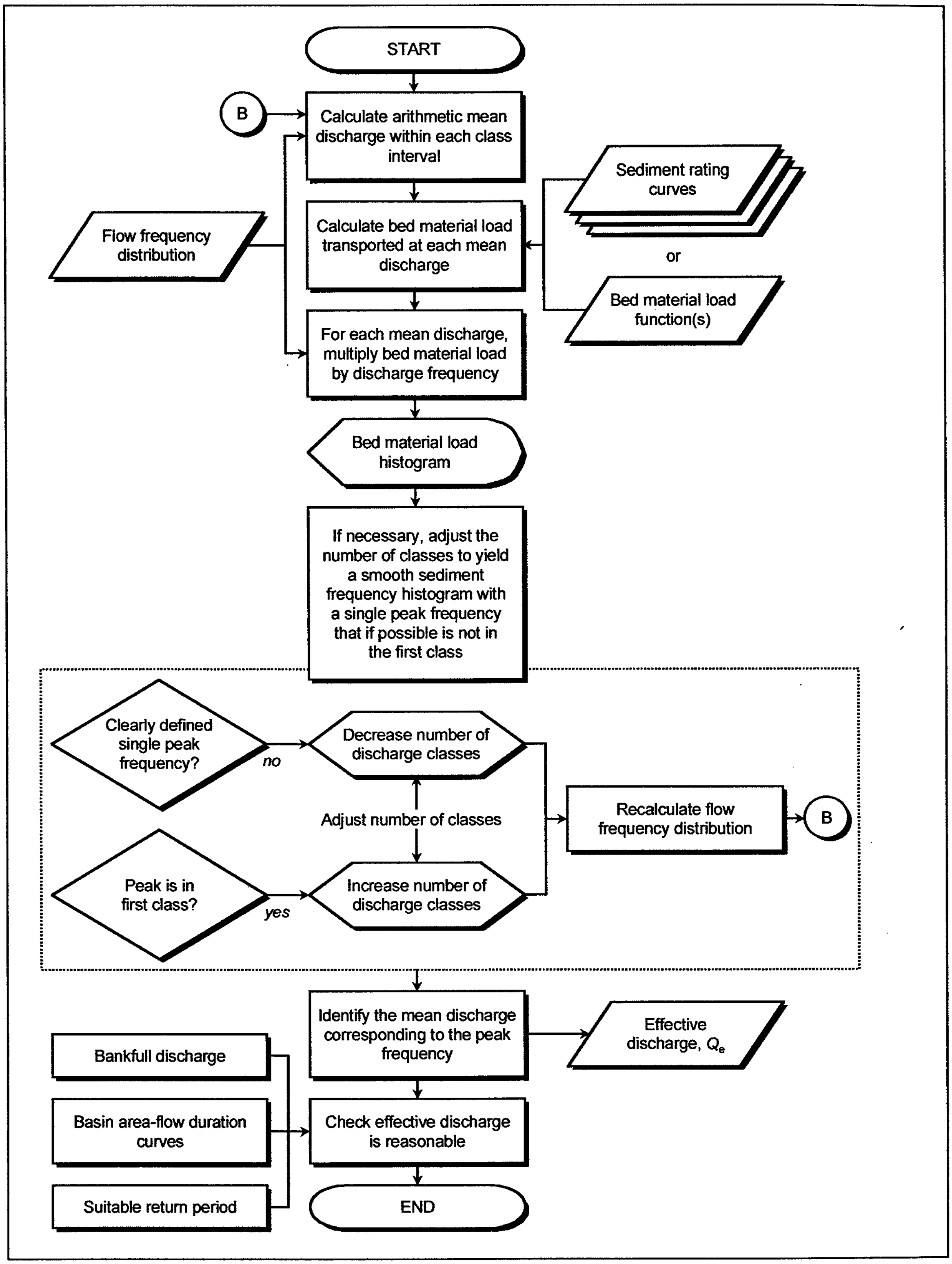
Since the DEC Manual was completed, there has been further research on the best procedure for calculation of the effective discharge. The new procedure is encapsulated in three flow charts that replace those in the earlier version of the Manual. These charts are reproduced overleaf.



Procedure for Generating a Flow Frequency Histogram



Procedure for Generating a Bed Material Load Rating Curve



Procedure for Generating a Bed Material Load Histogram

Impact of Large Woody Debris

Large Woody Debris (LWD) is an important channel independent variable in many fluvial systems (Hogan, 1987). For example, Bevan (1948; quoted in Macdonald and Keller 1987) concluded that in the Middle Fork Willamette River, Oregon, woody debris was responsible for more channel changes than any other factor.

It is important to recognise that processes are scale dependent and that the influence of LWD on channel and valley morphology may change systematically downstream through the network (Abbe & Montgomery, 1993).

Keller & Tally (1979) suggest that debris loadings increase with stream size. While, Wallerstein (1997), found that, in degrading streams, differences in the volume of debris and spatial frequency of jams could be related to the Schumm et al. (1984) Channel Evolution Model (CEM). Analysis showed that the relative volumes of debris associated with each stage of the CEM could be related to the processes occurring during that stage. Specifically, the volume of debris in reaches was low in stable, Stage 1 reaches, rose rapidly in Stage 2 reaches (where knickpoints are active), was highest in Stage 3 reaches (due to lateral channel adjustment following incision), and fell in Stages 4 and 5 (due to the cessation of bank erosion and the burial of LWD by alluvial deposition).

Gregory et al. (1985), have characterised jams into three types :

1) Active (form a complete barrier to water and sediment movement, and create a distinct step or fall in the channel profile)
2) Complete (a complete barrier to water/sediment movement, but no step formed)
3) Partial (only a partial barrier to flow)

They suggest that these types become sequentially more prevalent as channel size increases.

Once trees fall into a stream, their influence on channel form and process may be quite different from that when they were on the banks, changing from stabilising to destabilising through causing local bed scour and basal erosion of the banks. Thus, jams represent a type of auto-diversion, that is, a change in channel morphology triggered by the fluvial process itself (Keller & Swanson, 1979). The type and degree of impact on channel morphology depends primarily on the channel width/tree height ratio and on debris orientation relative to the flow. Mean discharge and the dominant discharge recurrence interval are also important because the higher the flow is relative to jam size, the smaller will be the jam's impact in terms of acting as a flow diverter and roughness element. The principal effects of debris upon channel morphology are described below.

LWD influences the geomorphology of rivers on three levels (Gray, 1974); the overall channel form; detailed features of the channel topography; and channel roughness.

Heede (1985), Smith et al. (1993b), Andrus et al. (1988) and Mosley (1981) have all observed that the spatial distribution and number of pools, riffles and gravel bars is positively related to the distribution and volume of LWD in the channel. This relationship has been explained through laboratory experiments by Smith & Beschta (1994), who found that the pool-riffle sequence in gravel-bed rivers is maintained by a combination of mean boundary shear stress and intermittent lift and drag forces due to velocity fluctuations around debris. Random debris input will also distort the pool-riffle sequence, making it less systematic, so that the long- profile has very little spatial memory, or periodicity (Robinson & Beschta, 1990). Other studies have shown that a considerable proportion of the vertical fall of channels can occur at the sites of debris jams, accounting for a 4% of the vertical drop along a 412m reach of channel in Vermont (Thompson, 1995) and 60% of the total drop in Little Lost Man Creek in Northern California (Keller & Tally, 1979).

Debris jams, therefore, act as local base levels and sediment storage zones which provide a buffer in the sediment routing system (Heede, 1985, Bilby, 1981). Thompson (1995) found that LWD causes an important negative feedback mechanism, where, in the case of channel degradation, there is an increase in debris input due to mass bank failure, which in turn causes greater sediment storage. Channel bed elevation is consequently raised once more and the rate of bank failure and debris input is thereby reduced. On this basis, Klein et al. (1988) argue that jam removal can reduce the base level for the channel upstream and may trigger bank erosion. However, in an experimental study by Smith et al. (1993a and b) it was found that, while the removal of debris from a small gravel bed stream initially caused a four fold increase in bed load transport at bankfull flow, the associated loss of scour turbulence and greater flow resistance imparted by alternate bars actually resulted in a reduction in stream power which was compensated for by sediment deposition and net channel aggradation.

Potential energy is dissipated at jams, with energy loss being as much as 6% of total potential energy (MacDonald et al., 1982). Shields & Smith (1992) found that the Darcy-Weisbach friction factor was 400 % higher at base flow in an uncleared river reach compared to a clear condition, but that this value declined to 35% at high flows. The velocity distribution is also far more heterogeneous in debris-filled reaches, especially at low flow. Changes of stream power distribution due to flow resistance effects in turn give jams the ability to influence the location of erosional and depositional processes. Also the backwater effect created by jam back-pools may induce local silting (Keller & Swanson 1979). Thus, in small, stable channels log steps generally increase bank stability and reduce sediment transport rates by creating falls, runs and hydraulic jumps. The localised dissipation of energy can, however, result in associated local scour and bank erosion that causes channel widening. Bank failure may also occur through flow diversion around a debris obstruction. Davis & Gregory (1994) have also suggested a mechanism whereby bank failure is induced through the erosion of a porous, gravel, bank subsurface due to the greater hydrostatic pressure caused by debris dammed flow. Conversely, Keller & Tally (1979) have observed that flow convergence under logs may cause channel narrowing, with sediment storage upstream and a scour-pool downstream of the log step.

As drainage area increases, and the channel width/tree size ratio exceeds unity, flow is diverted laterally, inducing bank erosion through local basal scour. Hogan (1987) found that in undisturbed channels in British Columbia organic debris orientated diagonally across the channel resulted in high width and depth variability. However, in catchments where there had been logging operations the majority of in-channel discarded timber was orientated parallel to the flow and it subsequently became incorporated into the stream banks, protecting them from erosion. Nakamura & Swanson (1993) and Keller & Swanson (1979) have suggested that there is a progression of types of interaction between debris jam and channel processes, ranging from local base level control and possible local widening in low-order streams, to lateral channel shifts and even meander cut-off in middle-order channels, where debris is moved into larger more coherent jams which may either increase or decrease the channel stability depending upon the erodibility of bed and banks. In larger channels still, bars may form and flow bifurcate around debris obstructions. This last process has been documented by Nanson (1981) in British Columbia, who found that organic debris deposited at low flow provided the nuclei for development of scroll bars, through the local reduction of stream power. Hickin (1984) also observed crib-like bar-head features, but was undecided regarding whether the debris caused bar formation, or whether the bars pre-dated and trapped the debris. In either case, organic debris would, at the very least, enhance sediment deposition and bar formation.

Wallerstein (1997) conducted a study of the geomorphic impact of LWD in degrading unstable, sand bed, streams in Northern Mississippi and, like others, found that jam type changed as the ratio of average channel width to average tree height became larger in the downstream direction. Each jam type (see Figure 1) has a distinctive impact upon channel morphology. Wallerstein concluded that LWD did not have a detrimental impact in terms of stabilization of degrading systems because it dissipates energy and causes sediment retention. Sediment storage volumes were found to be only small however, as compared to that documented by Thompson (1995) in gravel bed rivers, because energy dissipation at the debris quickly leads to scour and re-routing of the flow that prevents significant backwater pools from forming.

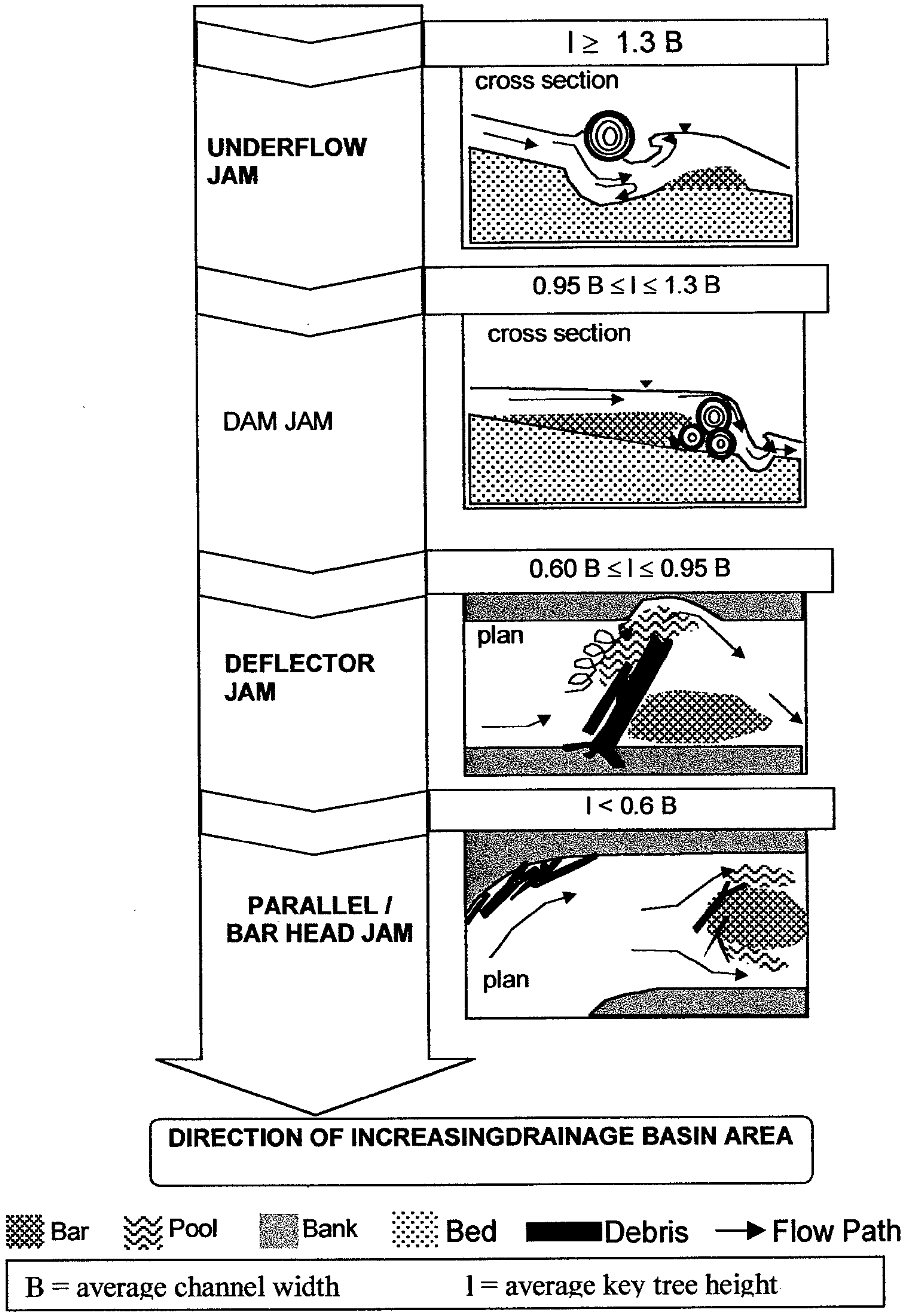


Figure 1. Debris Jam classification Model (Wallerstein, 1997)

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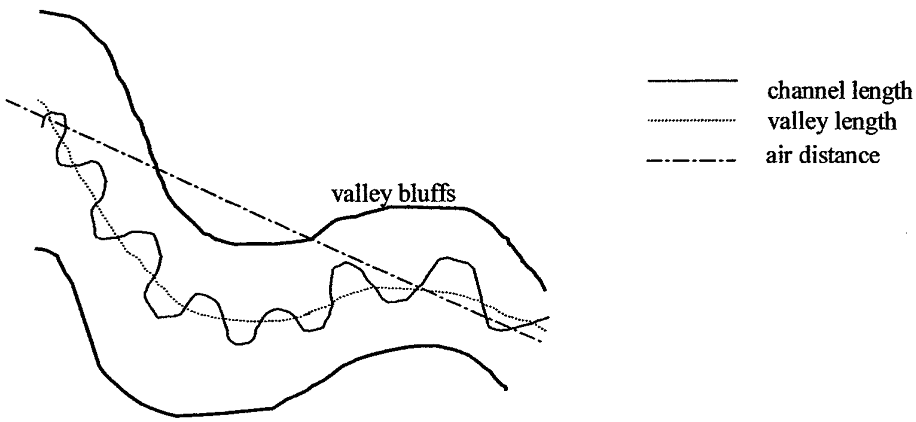
Sources of GIS and Catchment Data in the UK and Europe

Organizations in the U.K and Europe that offer GIS and other data resources include:

British Geological Survey	www.bgs.ac.uk
European Environment Agency	www.eea.eu.int
Government Information Service	www.open.gov.uk
Ministry of Agriculture Fisheries and Food	www.maff.gov.uk
Ordnance Survey	www.ordsvy.gov.uk

Defining Sinuosity and Total Sinuosity

Sinuosity (P) is a commonly used parameter to describe the degree of meander activity in a stream. However there are several different definitions of sinuosity and the distinction should be explicitly made between total sinuosity, valley sinuosity and channel sinuosity (refer to figure 1).



1) Total sinuosity: channel length divided by air distance.
2) Valley sinuosity: valley length divided by air distance.
3) Channel sinuosity: channel length divided by valley length

Figure 1. Definition of meander parameters (after Richards, 1982).

Channel sinuosity (symbol P) is the most commonly used parameter to define morphology of meandering channels. However if this measure is strictly applied to braided channels sinuosity approaches one as for straight channels, even though the morphology of the channels is quite different. Robertson-Rintoul and Richards (1993) point out that a quantitative index of channel pattern, such as sinuosity, needs to be applicable to both meandering and braided rivers in order to be representative of the morphological continuum. They therefore suggest a measure of total channel sinuosity defined by:

$$\text{Total channel sinuosity} = \frac{\text{total active channel length}}{\text{valley length}} \qquad (1)$$

They also make reference to a definition suggested by Le Ba Hong and Davies (1979), a total sinuosity index (ΣP), in which the cumulative length of channels in a reach is divided by the reach length.

Both these definitions require an operational definition of what constitutes a 'channel' to enable their use in multi-thread channels. Robertson-Rintoul and Richards provide the definition that active channels are those diverted around the channel macro-forms (large bars or bar complexes) according to the consistent hierarchical classifications of Church and Jones (1982) and Parker (1976) These classifications are consistent with the first order channels defined by Williams and Rust (1969) (refer to figure 2).

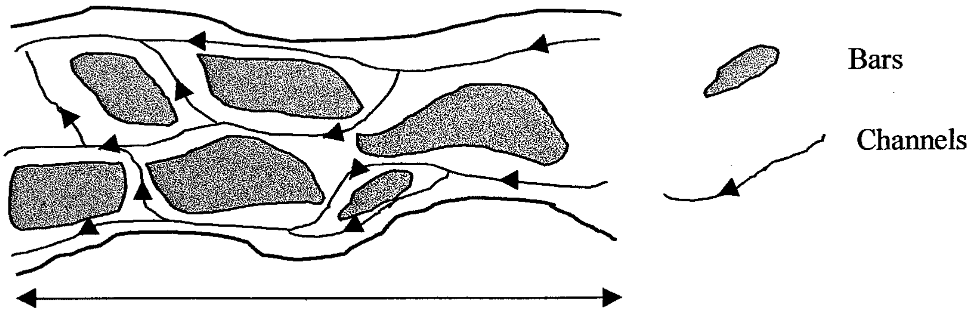


Figure 2. Definition diagram of total channel sinuosity in braided rivers (modified from Robertson-Rintoul and Richards (1993).

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RIVER WIDTH ADJUSTMENT & BANK STABILITY ANALYSIS

Fluvial bank erosion

Bank material is entrained due to the drag and lift forces exerted by the adjacent flowing water. These erosive forces are resisted to varying degrees by the normal component of the bank sediments weight, packing of grains, and electrochemical bonding forces. The characteristics of bank sediments generally vary vertically with a fining-upward trend because they are usually fluvially deposited in the first place. The nature of bank sediments may also vary laterally along the river due to clay plugs and gravel lenses. As a consequence the extent of bank erosion may be spatially quite varied.

The mobility of non-cohesive bank material can be determined using a Shields-type entrainment function, but which must also take into account channel bank slope and sediment packing.

Fine-grained silts and clays resist erosion primarily through electrochemical bonding, so when such material is entrained by the flow it is generally as aggregates of material. Near-bank shear stresses must therefore exceed the threshold for the aggregate size not for individual particles size. Because aggregate size and stability depends to a certain degree on the recent antecedent conditions of wetting and drying conditions determining the conditions for incipient motion on cohesive river banks is complex and time dependent. In however, erosion rates for cohesive banks tend to be lower than for non-cohesive ones.

Notable papers on the erosive resistance of clayed soils include Grissinger (1982), Kamphuis and Hall (1983), Parchure and Mehta (1985), Springer et al. (1985) and Kranenburg and Winterwerp (1997).

The erodibility of bank materials is increased by the processes of weakening and weathering, which are primarily driven by soil moisture conditions. Following rapid flow draw-down banks may be poorly drained which can increase pore water pressure and induce bank failure. Conversely, rapid immersion of a dry bank can cause slaking, which is the detachment of aggregates by positive pore pressures caused by compression of trapped air. swelling and shrinkage of soils during cycles of wetting and drying can cause cracking which increases erodibility. The growth and melting of ice crystals has a similar effect.

Because the operation of weakening forces on banks are temporally variable the effectiveness of a given flow to remove bank material depends not only on the magnitude and duration of the event but also on antecedent conditions (Thorne, 1998).

Mechanics of Bank Failure

Fluvial erosion causes channel widening though the removal of material from the bank, however, it is also the cause of more catastrophic, mass failure.

Bank stability with respect to mass failure depends on the balance between gravitational driving forces and frictional and cohesive resisting forces. Bank failure occurs when fluvial erosion of the bank toe causes either an increase in bank height, or causes undercutting which increases bank angle, to the point where driving forces exceed resisting forces and mass failure occurs.

Failure mechanics are different in cohesive and non-cohesive materials. In non-cohesive banks shear strength increases more rapidly with depth than shear stress so critical conditions are more likely to occur at shallow depths. Non-cohesive banks tend to fail by dislodgement of particles or by shear failure along shallow, curved surfaces. Conversely, in cohesive banks shear stress increases more rapidly than shear strength with depth so critical surfaces tend to be located deep within the bank. This causes a block of, sometimes intact, material to slide into the channel. There are two main types of deep-seated failure:

a) Rotational Failure: Occurs in high banks with shallow slope angles (< 60°). The failure surface is curved and the block tends to rotate back towards the bank as it slides.
b) Planar Slip Failure: Occurs in lower banks with steep slope angles (> 60°). The detached block tends to slide downward and outward as it fails.

Computation of the position of rotational failure slip surfaces is usually determined using the slice method (Bishop, 1955) where the soil within the failure arc is divided into segments and the factor of safety determined for each. The critical slip surface is usually determined using a computer program as there are a large number of possible solutions.

River banks which are undergoing active fluvial erosion tend to be steep and therefore fail by planar slip. The factor of safety on such banks can be determined using the Culman method in which forces acting on the potential failure block are resolved normal to and along the failure plane.
The resisting force is derived from the shear strength of the soil and keeps the slope from moving. The shear strength of the soil is defined as:

$$\tau = c' + \sigma' \tan \varphi' \tag{1}$$

where, τ = shear strength of the material, c' = cohesion of material, σ' = normal stress on the failure surface, and φ' = angle of internal friction of the soil.
The primes indicate that these parameters are measured with respect to the effective normal stress, which is determined by:

$$\sigma' = \sigma - u \tag{2}$$

where, σ = normal stress minus u the bank pore water pressure. The parameters c' and φ' required laboratory testing using for example a direst shear apparatus.
Forces tending to cause movement of the slope, the driving forces, include the weight of the soil mass and any external loading. The ratio between resisting and driving forces define the factor of safety (FS) which is determined by:

$$FS = \frac{F_R}{F_D} \tag{3}$$

The resisting forces (F_R) is determined by:

$$F_r = c'L + N \tan \varphi' \tag{4}$$

where, $L = H_c/\left(\frac{\theta}{2}+\frac{\varphi}{2}\right)$, θ = bank angle, H_c = critical bank height, $N = W\cos\left(\frac{\theta}{2}+\frac{\varphi}{2}\right)$, W = weight of a unit width of bank.

The driving force (F_D) is determined by:

$$F_D = W\sin\left(\frac{\theta}{2}+\frac{\varphi}{2}\right) \qquad (5)$$

The various parameters are shown in Figure 1.

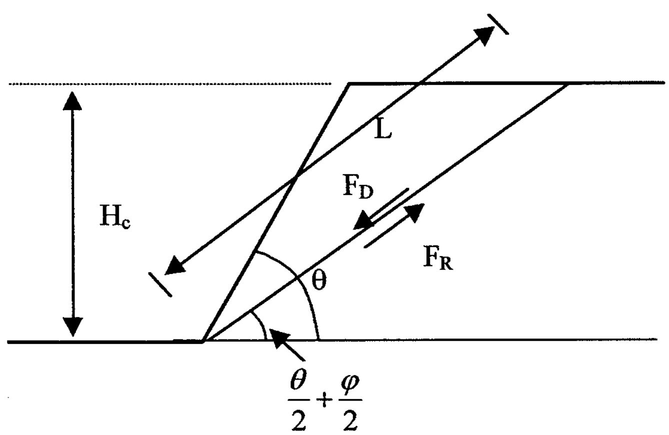

Figure 1. Culman Analysis for Plane Slip Failure (modified from Thorne, 1998).

The factor of safety can also be considered as the ratio of the critical bank height to the actual bank height as represented by:

$$FS = \frac{H_c}{H} \qquad (6)$$

where, $H_c = 4c(\sin\theta \quad \cos\varphi)/\gamma[1 - \cos(\theta - \varphi)]$, γ = bank material unit weight, and H = actual bank height.

For both failure ratio methods failure is anticipated when the factor of safety is less than unity.

Often, just prior to failure, a tension crack will develop parallel to the stream bank and can be observed from the top bank. A tension crack is a vertical separation of the soil resulting in a cavity. Vertical tension cracks at the surface of a slope, possibly occuring along natural cleavage planes, reduce the overall stability of a slope. The presence of tension cracks reduces the critical bank height (see Figure 2). At failure, tension cracks may quickly develop to depths greater than half the bank height. As a conservative, measure, Thorne and Abt (1989) recommend using a tension crack depth of half the bank height if no site-specific data are available. generally, varying a tension crack depth from 30 to 50% of the bank height is a realistic range and does not change the factor of safety by more than 10%.

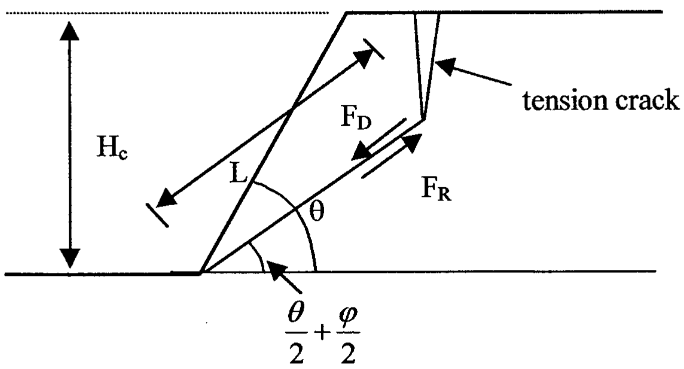

Figure 2. Shear Failure along a Planar Slip Surface through the toe of the slope with a tension crack

A third type of bank colapse is canalever failure. This occurs when an erodible layer of a stratified bank is removed, undermining an overlying, erosion resistant layer. Cantilevered banks may fail by shear, beam or tensile collapse (Thorne and Tovey, 1981). Shear failure occurs when the weight of the cantilever block exceeds soil strength, causing the overhanging block to slip downwards along a vertical plane. Beam failure occurs when a block rotates forward about a horizontal axis within the block and tensile failure occurs when the tensile stress exceeds soil tensile strength and the lower part of the overhanging block falls away. Cantilever blocks are strengthend by root reinforcement.

To summarize, in rotational, planar and cantilever failure the main force causing failure is the tangential component of the weight of the block. Fluvial erosion can increase the driving forces by scouring the bed, which increases bank height, and/or by lateral erosion of the bank toe, which increases bank angle.
banks are also destabilised by:
a) loss of confining pressure provided by high water;
b) positive pore water pressure due to poor drainage, and;
c) increase in the effective unit weight of the soil due to saturation.

The Role of Pore Water Pressure

An important consideration in bank stability analysis is the role of pore water pressure. The following discussion of this component of bank stability is taken from Casagli et al. (1999).
Pore water pressure is defined as the pressure of water filling the voids between solid particles. In a fully saturated soil all voids are filled with water and, under these conditions, the effective stress (σ') is given by the difference between the total normal stress and the pore water pressure (u_w). In partially saturated soils the voids are occupied by air and water and pore water pressure is less than pore air pressure (u_a) due to surface tension at the air-water interface. In this condition pore water pressure above the water table is negative and the difference between the two quantities is ($u_a - u_w$) is defined as matric suction.

Matric suction increases the shear strength of an unsaturated soil. In this case the failure criterion can be expressed in terms of normal stress $(\sigma - u_a)$, and the matric suction $(u_a - u_w)$ in an equation of the form:

$$\tau = c' + (\sigma - u_a) \tan \phi' + (u_a - u_w) \tan \phi^b \qquad (7)$$

where τ = shear strength, c' = effective cohesion, ϕ' = friction angle in terms of effective stress and ϕ^b = angle expressing the rate of increase in strength relative to matric suction.

This criterion represents an extension of the Mohr-Coulomb failure envelope in terms of effective stress because as the soil approaches full saturation the pore water pressure tends to equal to pore air pressure and the suction component disappears. The extended Mohr-Coulomb failure envelope for an unsaturated soil can be plotted in a three-dimensional diagram with the shear stress and the two stress state variables on the three axes. The extended failure envelope can be presented as a projection onto the shear stress versus $(\sigma - u_a)$ plane. For a given matric suction value, the projection is a line given by the equation:

$$\tau = c_a + (\sigma - u_a) \tan \phi' \qquad (8)$$

where c_a = total cohesion intercept, which incorporates the effects of the matric suction and of the effective cohesion:

$$c_a = c' + (u_a - u_w) \tan \phi^b \qquad (9)$$

Pore water pressure distribution within a soil also controls the seepage both in the saturated and unsaturated zone. In unsaturated flow the permeability is a function of the degree of saturation, and the effects of matric suction have to be included in the expression of hydraulic head, by considering the negative pressure head (matric head) h_m:

$$h_m = -\frac{(u_a - u_w)}{\gamma_w} \qquad (10)$$

where γ_w = unit weight of water

Total head h is the sum of elevation head and matric head:

$$h = z - \frac{(u_a - u_w)}{\gamma_w} \qquad (11)$$

Apparent cohesion due to matric suction can represent a large component of a river banks' total shear strength. Thus, if a bank is composed of fine material the high shear strength term due to matric suction allows the bank to remain stable during low-flow periods at angles much higher than the effective friction angle. During high flow periods or during heavy rainfall water content in a bank will rise as will pore water pressure so that apparent cohension may disappear altogether and positive pore water pressures occur. In these circumstances it might be expected that the bank will fail, but stability can be maintained by the confining pressure of the high flow. Bank failure is therefore most likely as high flows recede and the bank material is still at or near saturation.

Simon et al. (1999) also demonstrate how matric suction increases the resistance of banks to mass failure as well as failed cohesive blocks to entrainment by fluid shear. They have found that a stable bank is transformed into an unstable one during periods of prolonged rainfall through;

(6) increase in soil unit weight;
(7) decrease of matric suction and therefore apparent cohesion;
(8) generation of positive pore water pressures and therefore loss of frictional strength;
(9) entrainment of insitu and failed material at the bank toe, and;
(10) loss of confining pressure during recession of the storm hydrograph.

They conclude that it is rapid drawdown conditions which represent the dominant condition for bank mass failure, and also note that it is not the largest storms and greatest floods that induce bank failures but prolonged wet periods which weaken *in situ* bank materials.

Basal Endpoint Control

Fluvial erosion and geotechnical failure control different aspects of bank retreat but are actually linked because while mass waisting delivers material to the toe of the slope (basal area) it does not remove it frrom the bank profile. Removal is only achieved by fluvial entrainment. The concept of basal endpoint control explains how medium to long-term retreat of a bank is determined by the rate of sediment entrainment and removal from the toe. This theory was first appliad to hillslopes but Thorne (1982) appliad it to the river bakn environment. He proposed that bank retreat would only occur when near-bank flow velocities are sufficient to remove the failed debris. When velocities are not sufficient to remove material basal debris accumulates and a berm is formed which protects the bank from further attack.
Figure 3 shows a schematic diagram of sediment fluxes in the near bank zone. There are three states of basal endpoint control, and therefore lateral channel stability

1) INPUT > OUTPUT (Impeded removal):	Basal sediment wedge accretion, berm formation and increasing bank stability.
2) INPUT = OUTPUT (Unimpeded removal):	Equilibrium between processes delivering and removing sediment, bank retreat limited.
3) INPUT < OUTPUT (Excess basal capacity):	Removal of sediment wedge by basal scour causing basal lowering, a reduction in bank stability, and an increased bank retreat rate.

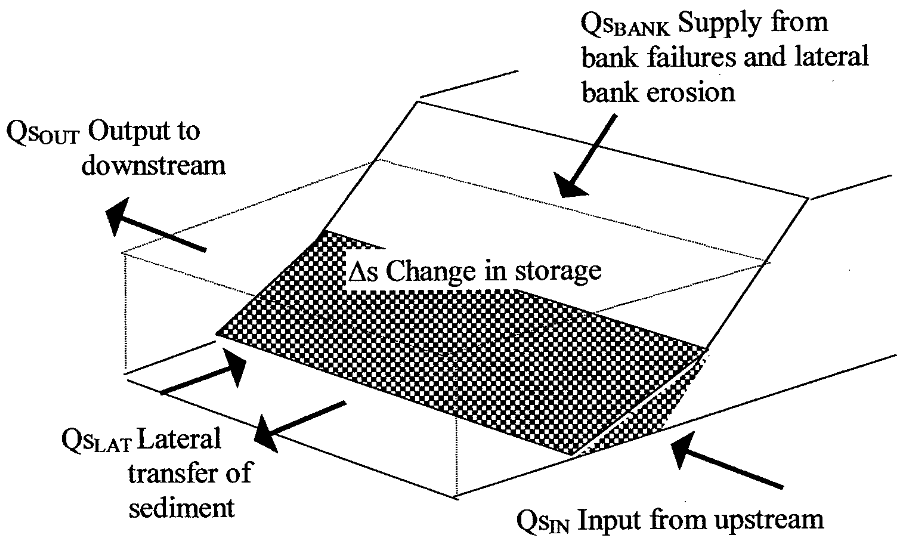

Figure 3. Basal Endpoint Control: Sediment flux in the near bank zone (modified from Thorne & Osman, 1988)

This concept highlights the importance of considering the response of near-bank morphology to bank stabilization as, for example, a programme of bank stabilization mat reduce the supply of debris to the basal area causing excess basal capacity which results in toe scour.

Vegetation Effects

The impact of vegetation on bank stability can be either positive or negative with respect to mass failure (Thorne et al. 1997). Depending on the dominance of either fluvial processes or mass wasting, vegetation can produce either a net increase or decrease in the rate of bankline retreat.

Vegetation can have a major impact in terms of limiting fluvial entrainment of grains and loose aggregates as:

a) Foliage and plant residues intercept and absorb rainfall energy and prevent soil compaction by raindrop impact.
b) Root systems physically restrain soil particles.
c) Plant stems dampen turbulence to reduce peak shear stresses.
d) Roots and humus increase permeability that reduces excess pore water pressure.
e) Depletion of soil moisture through plant uptake reduces water-logging.

Vegetation may also affect the balance of forces that determine mass wasting (Gray and Leiser, 1982). Roots mechanically reinforce soil by transferring shear stresses in the soil to tensile stresses in the roots. This effect obviously only occurs to the depth to which roots penetrate.

Vegetation also intercepts rainfall, which reduces soil moisture, and therefore pore water pressure.

Negative effects of vegetation include the fact that roots may enter fissures and expand them and trees growing close to a bank edge may surcharge the bank by adding extra weight or provide a destabilizing moment in high winds. Fallen trees, in the form of in-channel Large Woody Debris, may also cause flow deflection against the bank but can, on the other hand, act as bank protection against fluvial attack.

The Effect of Seepage

Water flows into banks during high flow events, then flows out again as flows recede, this out-flowing water may become concentrated also seepage lines, resulting in the erosion of bank material. This process is known as piping (Hagerty, 1991a / 1991b). Piping may result in the erosion of cavities in the bank which are deep enough to result in tensile failure of overlying layers.
It is important that the river engineer considers the effect of piping when designing bank stabilization works as, for example, rip-rap may be adequate to protect a bank from fluvial attack but the bank may still fail due to internal erosion through piping.

Channel Narrowing through Bank Advance

Riverbanks may advance through berm formation against the bank base. Conditions suitable for river bank advance include river sections where the width is greater than mean bank-full width and in channels which are returning to equilibrium dimensions after widening due to degradation (Schumm et al., 1984). Harvey and Watson (1988) proposed a mechanism for berm formation in sand bed channels whereby dunes, which form during high flows, are left as remnant formations along the channel margins as the flow recedes. These remnant dunes are then covered by mud drapes, deposited in low flow condition, which enables pioneer species to colonize the incipient berms. The vegetation then reduces flow velocities during future high flow events, causing further sediment deposition on the berm surface.
In essence berms will form when there is impeded removal in the bank basal region.

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NEW CHAPTER 7

PROJECT MONITORING AND MAINTENANCE

7.1 Introduction

The river environment is dynamic while the majority of channel stabilization and rehabilitation works are static in nature so maintenance and repair of structures is an inevitable part of the river engineering process.

Works must therefore be maintained on a regular basis, post construction, to enable effective remedial work to be carried out in good time before major failures occur.

Despite this knowledge it is a fact that the majority of stream engineering programmes do not, at present, have funds set aside for a structured programme of post construction appraisal and maintenance.

Reasons for the absence of such programmes can therefore only be attributed to a lack of awareness of the sometimes rapidly changing nature of the fluvial environment among the engineering community, but also, more significantly, may be due to a project funding culture which is unwilling to spend significant sums of money on what is currently seen as a peripheral luxury compared to the primary engineering work.

7.2 Monitoring

7.2.1 Why Monitor?

Three primary reasons for monitoring are:

a) to detect the need for maintenance with adequate time;

b) to provide a basis for repair design, and;

c) to provide a body of literature on the performance of river stabilization and rehabilitation work which will benefit future projects.

7.2.2 Key Considerations

For a river engineering project to be successful it is essential that every aspect of the work is documented, from site geomorphic and hydraulic analysis, to catchment considerations, to construction plans and, more importantly, the as-built condition of the work. This information will provide the basis for determining project success. But key to the determination of success is a carefully thought out definition of the reasons for the work being carried out in the first place and what the project should achieve long-term. Without careful documentation of success criteria it is impossible to determine how well a project has performed post construction. This consideration is often overlooked or undervalued in current river engineering practice.

A second key consideration is that the river environment changes in response to what, for engineering time scales, may be rare events so monitoring cannot be performed for only a few years and then forgotten. A long-term assessment programme must be budgeted for, at least as long as the design life of the structure.

It must also be recognised that a project can only be deemed a 'success' relative to the actual flow conditions experienced over the structures life. There must, therefore, be documentation of the hydrologic conditions, pre and post construction.
A third consideration is that a consistent method must be used to monitor a project in terms of survey techniques and observation criteria so that records are comparable. The timing of monitoring must also be consistent as a site may appear quite different through the year due to changes in vegetation cover and changes in average flow stage.

7.2.3 Monitoring Data Requirements

The type of monitoring carried out obviously depends upon the type of construction in place so different projects have different data requirements. The frequency and intensity of data collection is an important consideration and is essentially site and catchment specific depending upon the hydrologic regime and the nature of the construction. A monitoring protocol and timetable must therefore be planned in the light of these considerations prior to construction.
data requirements have to be balance against long-term resource availability and a compromise will probably have to be sought between maximum data coverage and maximum monitoring lifetime.

Specific types of data that should be collected include:

a) Survey Data
This provides the primary data source. Surveys should include channel cross sections, the spacing of which depend on the nature of the construction, and the channel thalweg. These data will show rates of bed-elevation changes due to, for example, the emplacement of a grade control structure, and rate and location of width adjustment through bank failure and accretion which may result from structure induced flow deflection.
Surveys may have to be carried out under different flow conditions as the extent of scour, say for example, around a flow deflector or at the toe of bank protection may only be evident during high, sediment transporting, flows while the extent of deposition, for example, downstream of bendway weirs may only be evident during low flow conditions. Surveying during high floes is obviously more difficult and potentially dangerous but may be necessary none-the-less to obtain a true picture of the extent of channel scour and fill.

b) Geomorphic Data
Geomorphic assessment may be the most cost-effective form of monitoring. A well thought out geomorphic assessment sheet needs to be devised prior to project monitoring. Geomorphic assessment criteria will vary depending on the channel environment and dynamics and type of construction so careful consideration must be given, based upon documentation of previous projects, as to what morphological changes are most likely to occur and therefore what aspects of the channel environment to inspect the most closely.
Stream reconnaissance techniques and guidelines are presented in Thorne (1998). Techniques include probing the channel bed to determine sediment and scour hole depths, measurement of bank angles, identification of bed material grain size, identification of bank failure mechanisms, and identification of the location of zones

of aggradation. Other more sophisticated techniques include the use of dendrochronology to date bank failure and berm aggradation rates, the use of aerial photography to document channel migration rates and the use of stereoscopic pairs of photographs to determine changes in bar height elevations. Aerial photography is obviously more expensive than ground surveys but can be invaluable for obtaining the bigger picture of the catchment. A good ground-based photographic record is also essential as it provides information that may be overlooked at the time of survey or forgotten post survey. Another useful approach is to interview local landowners who may notice significant changes in morphology at and away from the construction site and to encourage such observations.

It cannot be over-emphasised how important catchment-wide assessment is to project evaluation as long-term project performance depends greatly upon the large scale geomorphic and environmental processes that are occurring such as tectonic uplift or subsidence, human or naturally induced changes in vegetation cover and changes in climatic regime. Consideration therefore needs to be given to the distance that monitoring should extend both upstream and downstream of the project site. Of primary importance is the identification of pre-existing channel instability such as knickpoint and meander migration; i.e. factors which may threaten structures in the future. Thus the rate of movement of geomorphic instabilities and potential for channel movement through geomorphic thresholds (Summerfield, 1991), such as the switch from a meandering to braided channel planform, must be monitored and assessed post-construction.

Of equal importance is consideration of the impact that structures themselves have upon the dynamics of a channel. Geomorphic assessment can identify the onset of downstream changes, such as degradation caused by sediment retention at a structure, and upstream changes, such as berm formation caused by backwater velocity reduction.

c) Hydraulic Data

Hydraulic data such as stage discharge relationships and flow velocities at the design discharge will have been collected or calculated prior to project design and construction but need to be monitored throughout the works design life.

Velocity data is important for determination of structural stability, say for example where rip-rap has been placed and for determination of sediment transport rates. The measurement of the spatial variation of velocities and therefore shear stresses enables the determination of where structures are most at risk from fluvial attack, where there is likely to be scour and fill and, in the case of habitat-based projects, the degree to which velocities have been varied through the site to induce flora and fauna diversity.

Measurement of discharge at given stages enables the calculation of channel roughness coefficients which may alter over time due to changes in vegetation cover, bed-material grain size and channel geometry. Long-term stage-discharge relationships, combined with knowledge of roughness coefficient changes also indicates whether the channel bed is degrading or aggrading in response to structure induces alteration of sediment continuity.

As monitoring of shear stress intensity and distribution is time consuming it will obviously have to be less spatially extensive than surveying, being concentrated around the structures themselves. Stage-discharge relationships can easily be determined if gauging stations are already in place in the locality of the works but this is often not the case so it may be necessary to install gauges specifically for the project. If possible as many as three gauges should be installed to determine stage-discharge relationship changes downstream, at and upstream of the construction site.

d) Geotechnical Data
The degree of geotechnical monitoring required will be highly site specific and depends upon the type of work constructed. For example, rip-rap bank protection may require a close inspection of ground water properties to mitigate against pop-out failure. geomorphic evaluation of channel banks can provide a good deal of information however, such as the location of failure, position of tension cracks and position of sub-surface drainage outlets in the bank profile. It may be necessary however to install piezometers and tensiometers to determine temporal changes in in-bank pore water pressure and matric suction which may result from changes in channel-bank water transfer.

7.2.4 Frequency of Monitoring

The first few years after construction and the first significant flow event are the two critical periods in any channel works. Monitoring should therefore be most intense immediately post construction and after the first flood flow.
As a minimum the site should, in the first few years, be inspected at least twice a year, once after the high flow season (if there is one) and once during low flow conditions. It is likely that most serious problems with the construction will become evident during this initial time period so if repairs are required the process of design and construction can be completed before the next high flow event.
Geomorphic adjustment of the channel is likely to be more long-term so any reasonable interval of monitoring will suffice for that element.
Beyond the initial period, frequency of monitoring must be determined either by regulation or through engineering judgement. Engineering judgement should take into account factors such as:
a) predicted return period of flood events;
b) predicted rate of change of channel geomorphology, both where it is unaffected by the construction but could impinge on it in future, and that caused by the works;
c) design life of the project, and;
e) factor of safety in the design and consequences of structural failure.

Vrijling (1995) suggests a 'pulsed monitoring regime where a site is monitored on a long-term basis with varying levels of effort.
Brookes (1996) suggests that monitoring should be undertaken for at least 10 years after engineering works have been completed.

7.2.5 Levels of Monitoring

Vrijling (1995) suggests the following five levels of monitoring. These guidelines were written specifically for riprap bank protection but can just as easily be applied to any other type of channel work.

Level 1: Field reconnaissance and visual observation of the site including a comparison between the present existing conditions and previous findings, plus aerial photographs.

Level 2: All activities included in Level 1 plus a permanent photographic and/or videotape record of the project area. All photos should be shot from fixed positions and be accompanied by written descriptions.

Level 3: All activities included in Level 2 plus measurement of scour/fill locations and depths.

Level 4: All activities included in Level 3 plus measurement of cross sections and thalwegs in the construction reach and downstream in degradation is a concern.

Level 5: All activities included in Level 4 plus additional data on bed material size and gradation, water quality, roughness, habitat and biomass analysis.

7.2.6 The PPA Approach

Skinner (1999) used the phrase 'Post Project Appraisal' (PPA) to describe all aspects of the monitoring process. The notion of PPA suggests that river rehabilitation projects require an explicit set of aims and objectives at the outset that can be evaluated once the project is finished. And it is essential, therefore, that monitoring is undertaken with a clear understanding of the scheme's objectives (Skinner, 1999).
Schemes therefore requires both pre-project data and post project long-term monitoring to evaluate success. This is rarely done as yet except in the case of demonstration projects.
From the analysis of thirteen semi-natural, rehabilitated and channelized rivers in the U.K. Downs and Skinner (1999) conclude that PPA practices could be improved by:

a) Defining success criteria for the project at the pre-initiation stage;
b) Establishing variables that are to be monitored;
c) Ensuring consistent monitoring post installation;
d) Documenting each stage of project construction;
e) Setting aside resources for project monitoring and evaluation;
f) Changing legislation (in the U.K.) to make PPA's mandatory;
g) Enabling evaluation results to be disseminated, regardless of success so that practitioners can learn from other schemes, and;
h) Using multidisciplinary teams in planning, design, installation and monitoring.

Downs and Skinner (1999) observe that, "River rehabilitation is a multidisciplinary subject with aspects of hydrology, geomorphology and ecology providing the basis for an integrated approach to a successful project.... Ideally (therefore), the integration of various disciplines would form the basis for the evaluation of a (river rehabilitation) scheme's success". At present this is not usually the case, with one discipline (usually engineering) controlling a river rehabilitation program. In the absence of a multidisciplinary approach to project monitoring Downs and Skinner (1999) put forward the case for a geomorphologically based Post Project Appraisal (PPA) system. They argue that such an approach is appropriate as most rehabilitation schemes (in the U.K.) are for the benefit of fisheries habitat creation and success in such projects can be determined through the quantification of geomorphological heterogeneity as this has been positively linked with habitat diversity. Their procedure for PPA is outlined in Figure 1. Figure 2 shows the approach developed by Skinner

(1999) for the overall design, monitoring and appraisal of a river rehabilitation scheme.

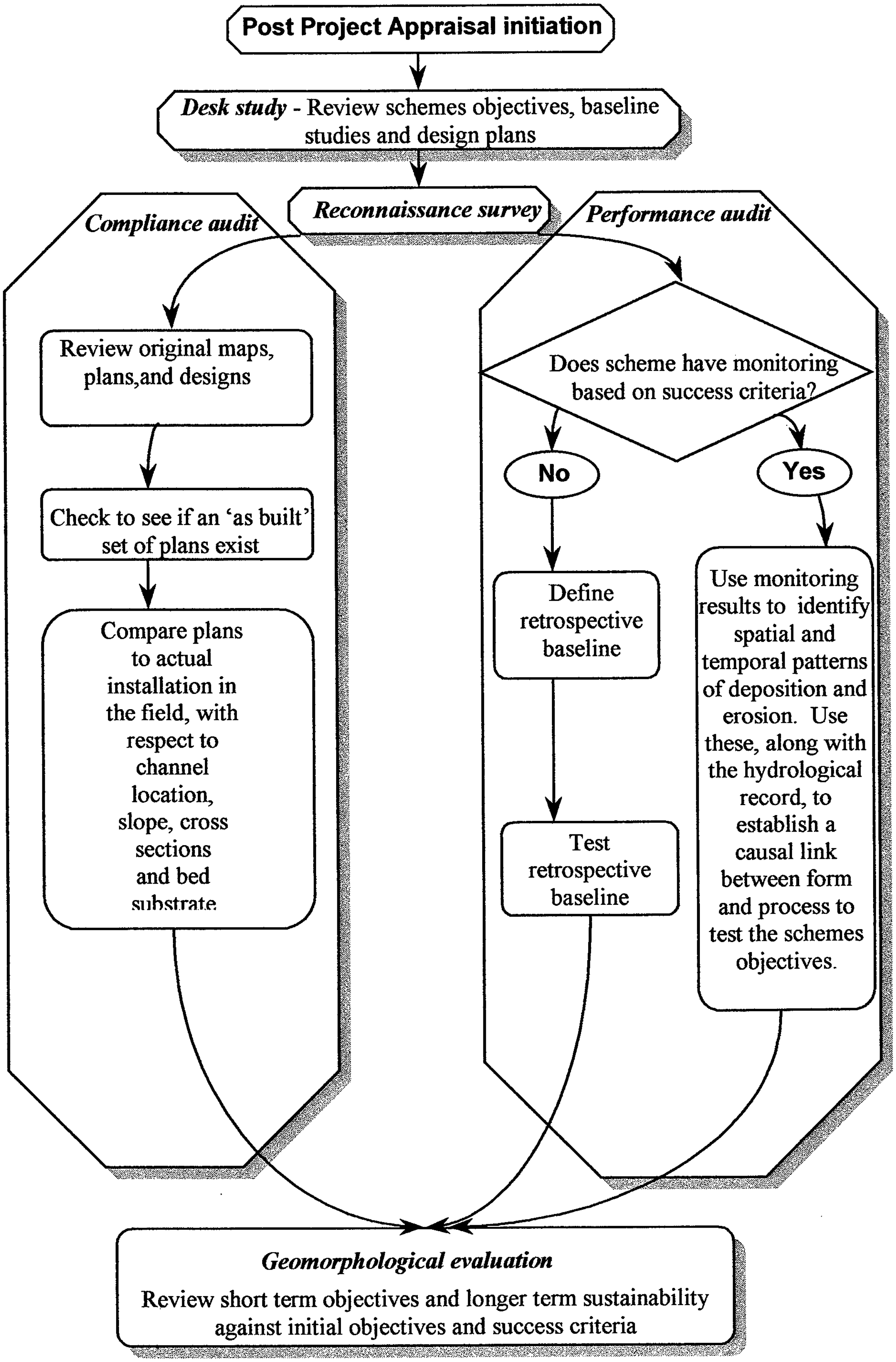

Figure 1. River Rehabilitation Post Project Appraisal procedure (modified from Downs and Skinner, 1999).

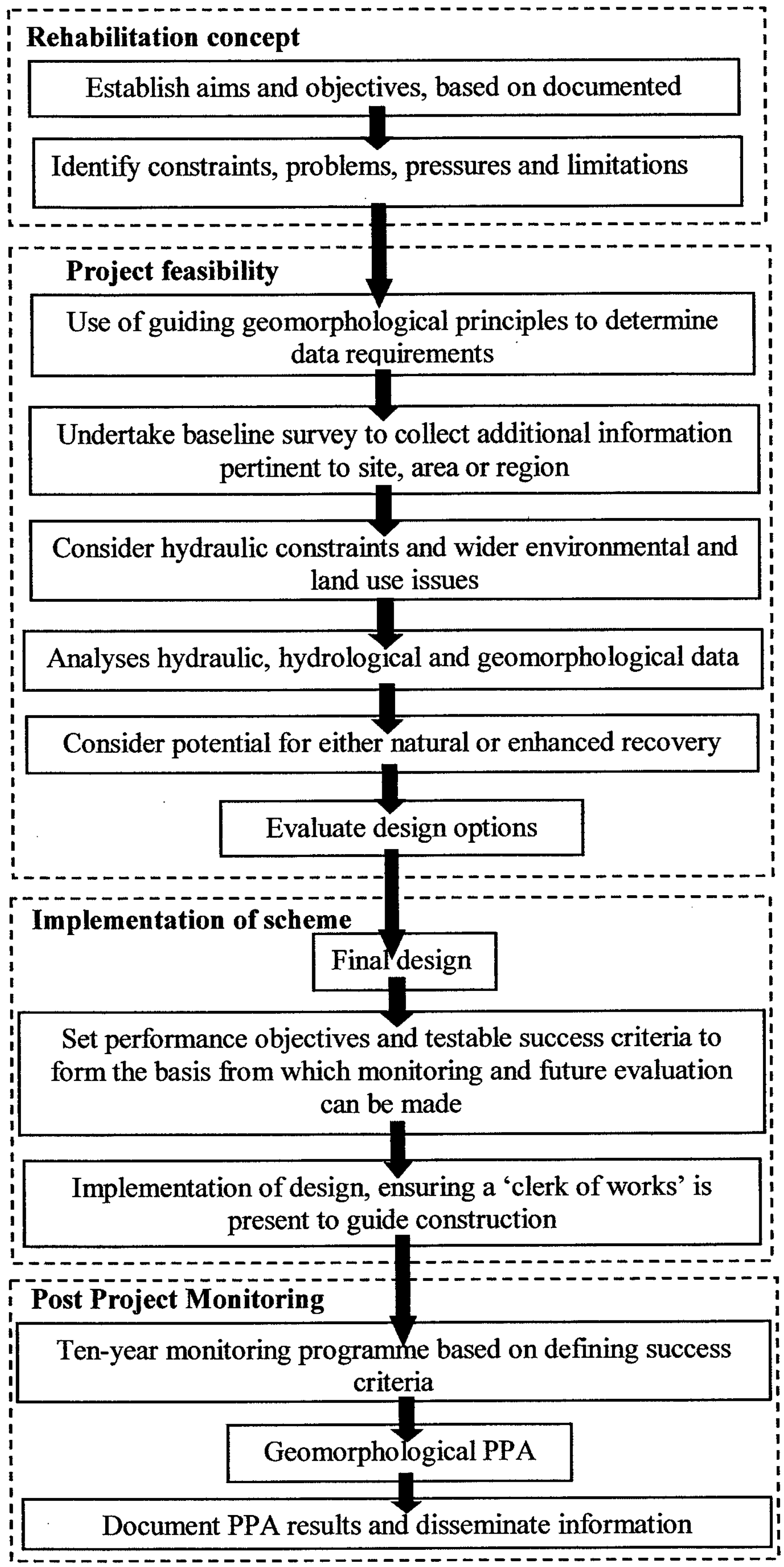

Figure 2. Approach to design, monitoring and appraisal of river rehabilitation schemes (modified from Skinner, 1999).

7.3 Maintenance

7.3.1 Introduction

According to Vrijling et al. (1995) a maintenance programme will include the following elements:
a) monitoring and inspection of environmental conditions and structural state;
b) appraisal of collected data to predict structural deterioration, and;
c) repair or replacement of components of the structure which have failed or which have a life expectancy less than the overall structure.

Key to such a programme is the necessity of developing a maintenance strategy at the structure's design stage. An example of a maintenance strategy outline is shown in Figure 3.

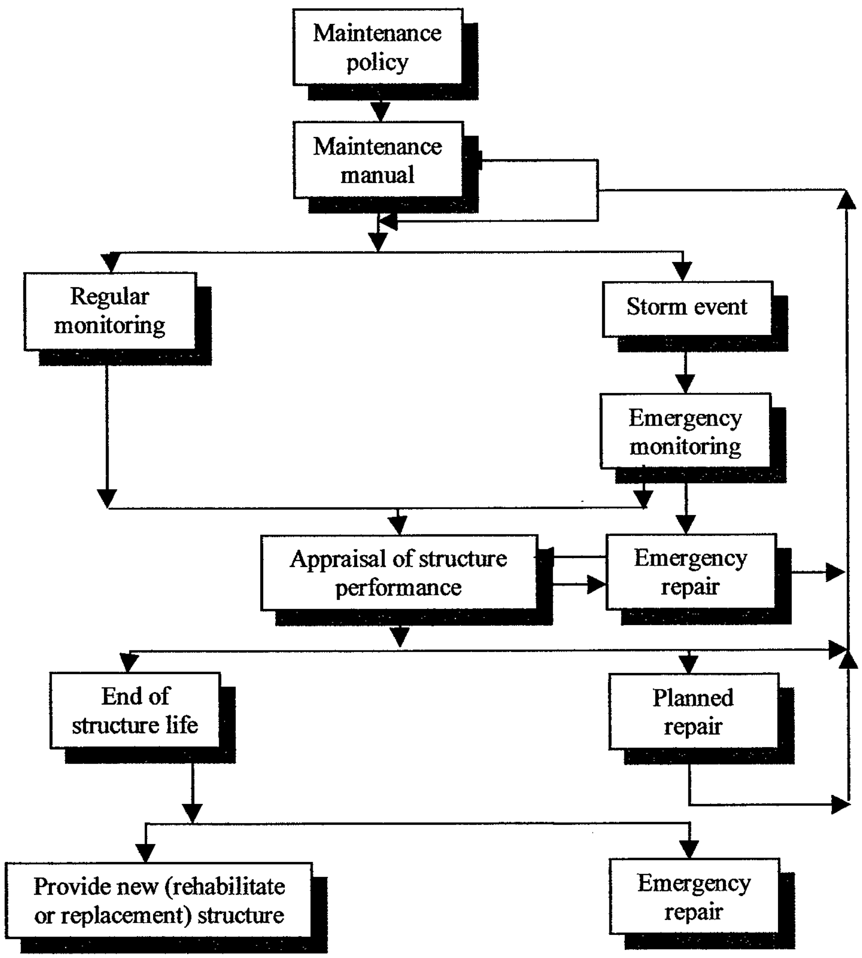

Figure 3. Example of a maintenance strategy (modified from Vrijling et al., 1995)

There are two classes of maintenance: corrective and preventative. Corrective maintenance implies that a structure is repaired after failure. Preventative maintenance means that a structures is inspected at specific intervals and repaired if necessary. According to Vrijling et al. (1995) the optimal cycle of inspection and repair is found by minimization of the present value of the sum of failure costs, inspection costs and repair costs. The conceptual framework linking design, maintenance and risk of failure is the minimal lifetime cost:

$$\text{Minimize } \{I + PV(M) + PV(R) + PV\,(p_t c_t)\} \tag{1}$$

where, I = investment in the structure, PV = present value operator, M = cost of monitoring, R = cost of repair, p_t = probability of failure per year, c_t = cost of failure.
It must be noted however that this model is derived from mechanical engineering practice and there are problems in applying it to hydraulic structures because;
a) in mechanical engineering failure rate is not related to environmental conditions, and;
b) hydraulic engineers cannot draw on large numbers of failed structures as a way to provide reliable data to construct failure rate curves.
Consequently this model needs refinements in the future to be applied to hydraulic structures.

7.3.2 Types of Maintenance

There are several approaches that can be taken to the maintenance procedure:

1) Failure based maintenance: Repair is undertaken when the structure fails. applicable only if the consequences of failure are limited.

2) Time based maintenance: Assumes that the structural state deteriorates according to a known function of time. Repair is due after a certain time has elapsed.

3) Use based maintenance: Assumes that structural state deteriorates as a known function of use. Usage must be monitored and repair is due after a certain number of times that the structure has been operated.

4) Load based maintenance: Attributes structural deterioration to loading (e.g. storms). Loading must be monitored and repair is due after a certain number of critical events.

5) State based maintenance: Based on the state of the structure. If the structures state appear to be no longer sound, repair is carried out.

Given these five maintenance types it is evident that choice of maintenance strategy depends on:

a) predictability of structural deterioration;
b) cost of inspection and monitoring;
c) cost of repair;
d) consequence of failure, and;
e) availability of methods to measure the structural state.

The choice of strategy can be determined using the flow chart in Figure 4.

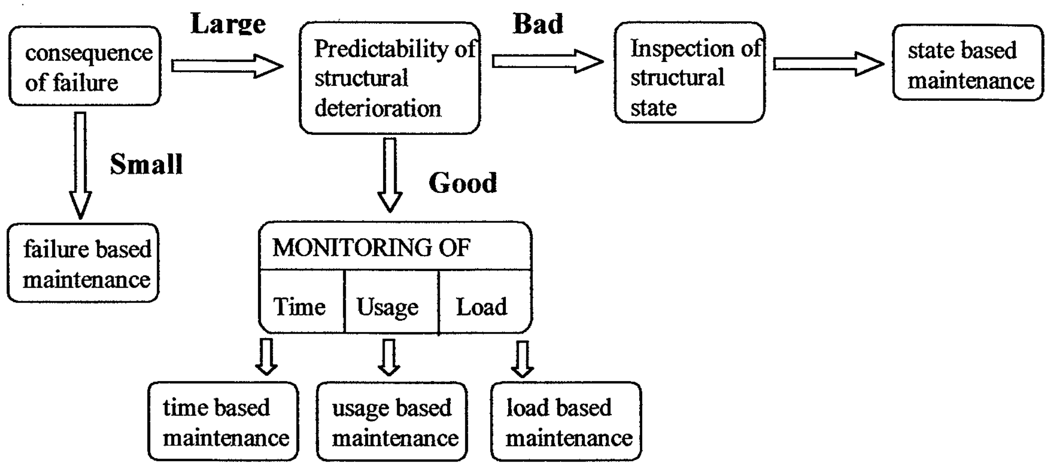

Figure 4. Choice of maintenance strategy (modified from Vrijling et al., 1995)

7.3.3 Structural Inspection Methods

It is important to note that inspection and monitoring of a structure is only required in preventative maintenance and is not required in failure based maintenance. Inspection methods selected must relate to the predicted failure modes of the structure.
There are three main inspection and monitoring methods relating to structures:

a) Measurement of structural state: this relates to the resistance or strength of the structure and should relate to a particular measurable variable (e.g. rip-rap D_{50}). Measurement can be performed on different levels:

(i) Visual inspection and photography;
(ii) Survey profiles;
(iii) Measurement of special variable, e.g. rock rounding

b) Measurement of environmental loading conditions: those factors discussed in the previous section on site monitoring.

c) Measurement of deformation, erosion or damage: includes measurement of internal response of the structure e.g. stresses in concrete, pore pressures, and the effect the structure has on the surrounding environment.

7.3.4 Frequency of Inspection

The method and frequency of inspection of a structure depends on the balance between investment in the structure, cost of inspection, cost of repair and consequences of failure.
Vrijling et al. (1995) advise a two tiered approach consisting of monitoring as a simple cheap continuous type of measurement and inspection as a more elaborate in-depth measurement of variables. Monitoring is covered in section 1. The inspection regime can be determined by following the same classification as given for maintenance:

a) Time based monitoring: inspection performed at regular intervals. The suggested minimum is between 6 and 12 months;
b) Load based inspection: used when structural deterioration depends on the rate of environmental loading (e.g. rate of scour hole development), and;
c) State based inspection: incremental in nature, where monitoring forms the basis of a decision to perform an in-depth inspection involving more resources.

7.3.5 Method of Repair

There are several approaches that can be adopted when it is recognized that a structure needs to be repaired, these are:

a) Restore the structure using the original approach;
b) Increase the factor of safety of the structure using the same materials but with a more severe design criteria, and;
c) Select a different method of construction.

Selection of the correct approach depends on the reasons for the structure's failure. If the structure needs repair because of conditions that are not expected to recur, for example slope settlement under rip-rap, then the same techniques as originally used may be adequate. If it is apparent that the original design criteria were not conservative enough, for example, rip-rap displacement by hydraulic forces, then the same method can be used as before but with a higher factor of safety. However, if the structures design seems have a fundamental fault then it may be necessary to entirely re-build the work using different material and/or techniques.

7.3.6 Conclusion

It is essential that river based structures have a well thought out inspection and maintenance programme set in place at the initial design stage as even the most robustly designed works will usually require some level of maintenance during their lifespan.

'Foresight is essential, because it is too late to begin an effective monitoring program once unforeseen damage requires major repair' (X, 19..)

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Conclusions

The significant features of the Manual to have benefited from the outcomes of this project are:

- Introduction of a logical structure dealing with the entire design process for river channel rehabilitation projects from feasibility studies, through project initiation, background studies, selection of structural and non-structural project elements, design, construction and post-project monitoring and appraisal;
- Addition of case studies selected to illustrate application of the principles underlying the manual;
- More extensive use of flowcharts to present guidance in a clear and logical fashion that is sufficiently robust to support a wide range of applications in different hydrologic, hydraulic and morphological environments.
- Expanded coverage of the application, utility, advantages, disadvantages and limitations of new and traditional channel stabilisation measures and materials including bio-engineering and non-structural solutions to instability problems.
- Increased applicability of the manual based on wider basis in theory and practice making it now of use both nationally and internationally.