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STABILITY OF COMPOSITE RIVER BANKS

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ABSTRACT

The stability of a river bank depends on the balance of forces, motive and resistive, associated with the most critical mechanism of failure. Many mechanisms are possible and the likelihood of failure occurring by any particular one depends on the size, geometry and structure of the bank, the engineering properties of the bank material, the hydraulics of flow in the adjacent channel and climatic conditions.

Rivers flowing through alluvial deposits often have a composite structure of cohesionless sand and gravel overlain by cohesive silt/clay. Bank erosion occurs by fluvial entrainment of material from the lower, cohesionless bank at a much higher rate than material from the upper, cohesive bank. This leads to undermining that produces cantilevers of cohesive material. Upper bank retreat takes place predominantly by the failure of these cantilevers. Three mechanisms of failure have been identified: shear, beam and tensile failure.

The stability of a cantilever may be analysed using static equilibrium and beam theory, and dimensionless charts for cantilever stability constructed. Application of the charts requires only a few simple measurements of cantilever geometry and soil properties. In this analysis the effects of cracks and fissures in the soil must be taken into account. These cracks seriously weaken the soil and can invalidate a stability analysis by affecting the shape of the failure surface.

Following mechanical failure, blocks of soil must be removed from the basal area by fluvial entrainment if rapid undermining and cantilever generation are to continue. Hence, the rate of bank retreat is fluvially controlled, even though the mechanism of failure of the upper bank is not directly fluvial in nature.

This cycle of bank erosion: undermining, cantilever failure and fluvial scour of the toe, operates over several flood events and has important implications for river engineering, channel changes, and the movement of sediment through fluvial systems.

KEY WORDS Bank stability Composite structure Fluvial processes Mechanisms of failure River engineering Slope stability charts Soil mechanics

INTRODUCTION

The stability of river bank depends on the balance of forces, motive and resistive, associated with the most critical mechanism of failure. Many mechanisms are possible and the likelihood of failure occurring by any particular one depends on the size, geometry and structure of the bank, the engineering properties of the bank material, the hydraulics of flow in the channel and climatic conditions (Thorne, 1978). An analysis based on static equilibrium may be used to assess the stability of a bank with regard to a particular mechanism of failure. In this approach motive and resistive forces acting on the potential failure plane are compared to produce a factor of safety. A value of unity for the factor of safety defines the critical case, with the bank on the point of failure. Safety factors greater than one indicate stability whilst those less than one suggest that failure should already have occurred.

Rivers flowing through alluvial deposits often have composite banks composed of non-cohesive and cohesive materials. The former are sandy gravel deposits formed from relic channel bars. The latter are sandy silt/clays deposited by overbank flow, in abandoned channels and on emergent bars. The characteristic bankform shows imbricated gravel with closely packed interstitial sand, overlain by sandy silt/clay. The interface between the non-cohesive lower bank and the cohesive upper bank is usually well defined (Figure 1). The sandy silt/clay of the upper bank has a ped fabric due to lateral swelling and shrinkage

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Figure 1. Composite bank structure. Non-cohesive sand and gravel overlain by cohesive sandy silt/clay. The vertical cut extends well into the closely packed and imbricated sand and gravel. The basal concavity is formed from a wedge of loosely packed gravel which has no imbrication. River Severn at Maes Mawr, Powys, Wales

during wetting and drying. Peds are separated by near vertical fissures which have a polygonal pattern in plan view. These fissures are planes of weakness in the soil and have important effects upon the stability of the bank and the critical mechanism of failure.

In the case of low composite river banks conventional slope stability analyses, developed in the field of civil engineering, are inapplicable. To explain why, it is necessary to examine the processes of erosion responsible for bringing composite banks to an unstable state. Initially the processes which operate on the lower and upper sections of the bank are considered separately. Then it is shown how the contrast in the rate of bank retreat between the upper and lower layers can generate cantilevers in the upper bank. A new analysis, based on static equilibrium, is presented to assess the stability of cantilevered banks. Subsequent failure of overhanging blocks of soil, and their removal from the toe of the bank by the flow, completes the cycle of erosion.

This cycle of events has important implications for river engineering, channel changes and sediment movement in gravel-bed rivers.

PROCESSES OF EROSION OF COMPOSITE RIVER BANKS

Non-cohesive lower bank

If no pore pressure or external force acts, the stability of a non-cohesive bank depends only on the angles of slope and internal friction (Taylor (1948)). Failure may be brought about by increasing the slope angle (oversteepening), or by reducing the friction angle. In rivers, basal scour leads to oversteepening, while processes of subaerial and subaqueous erosion act on alluvial deposits to reduce their packing density, destroy any imbrication and, consequently, reduce the friction angle (Carson (1971); Carson and Kirkby (1972)).

Non-cohesive materials are relatively coarse grained and are usually well drained. As a result, pore water pressure is seldom a significant factor. However, external forces of fluid drag and lift due to flow in the channel cannot be neglected. Various equations, of increasing complexity, have been developed to describe the stability of a particle at the bank surface (Lane (1955); Sundborg (1956); Task Committee on Sedimentation (1966); Graf (1971); Yen (1975); Yalin (1977)). Application of equations of this type and observations of erosion of cohesionless banks make it clear that particles in the sand and gravel size range are highly susceptible to erosion by fluvial entrainment (Thorne (1978); Thorne and Lewin (1979)). Fluvial erosion of the lower part of a non-cohesive bank can cause oversteepening and slip failures higher up the bank.

The two principal processes of erosion, fluvial entrainment and loosening, give the cohesionless lower bank a bipartite profile. The higher section is formed in closely packed and imbricated sandy gravel. The lower section consists of a wedge of loosely packed gravel that has accumulated at the foot of the bank after failure. This material has had the sand winnowed out and has lost its close packing and imbrication. Friction angles for the two bank sections are very different. When closely packed, the non-cohesive sandy gravel has a high friction angle and so it can stand at a steep bank angle. Indeed, it is even possible for closely packed imbricated gravel with a predominatly 'platy' shape to form an overhanging bank. Under such circumstances, it is difficult to assign a value to the friction angle for the closely packed material. The friction angle for the loosened material of the lower bank is reduced considerably. The change in packing density alone can cause a 20 to 30 per cent reduction in the friction angle (Carson (1971)), while if imbrication is lost too, reductions may be even greater. Generally friction angles for loosely packed granular materials range from 20° to 35°. Consequently, the stable slope angle is also much lower and the wedge of accumulated material produces a basal concavity in the profile (Figures 1 and 2). The proportions of the lower bank profile formed in closely and loosely packed material depend on the relative effectiveness of processes of loosening and of fluvial entrainment. Where fluvial activity is low, a large wedge of poorly packed gravel can accumulate so that most of the lower bank profile is at a low angle. Where fluvial activity is high, any loosened material is removed from the bank base soon after failure and particles are entrained directly from the imbricated deposits. Only small accumulations of poorly packed gravel then occur, principally between flood events. (Carson (1971); Brunsden and Kesel (1973); Thorne and Lewin (1979)). This system of slope profile control has been named 'basal endpoint control' by Carson and Kirkby (1972).

Cohesive upper bank

In contrast to the non-cohesive material of the lower bank, the cohesive soil of the upper bank is quite resistant to erosion by the fluvial entrainment of individual particles at the bank surface (Graf (1971); Yalin (1977); Thorne and Lewin (1979)). Field observations show that unless the surface of a cohesive bank is loosened or weakened by such processes as frost heave or thorough wetting, fluvial entrainment alone is not particularly effective in causing erosion (Wolman (1959); Thorne (1978); Hooke (1979)). Also, the position of the cohesive layer at the top of the bank results in a much lower frequency of attack by the flow. Even when the channel is near bankfull stage and the upper bank is being attacked, velocity measurements in bends (where most bank erosion occurs) suggest that the lower bank may experience the higher velocities and boundary shear stresses (Bathurst, Thorne and Hey (1979)).

Although, for these reasons, the cohesive upper bank is less affected by fluvial erosion than the non-cohesive lower bank, its engineering properties favour retreat by failure along discrete failure surfaces

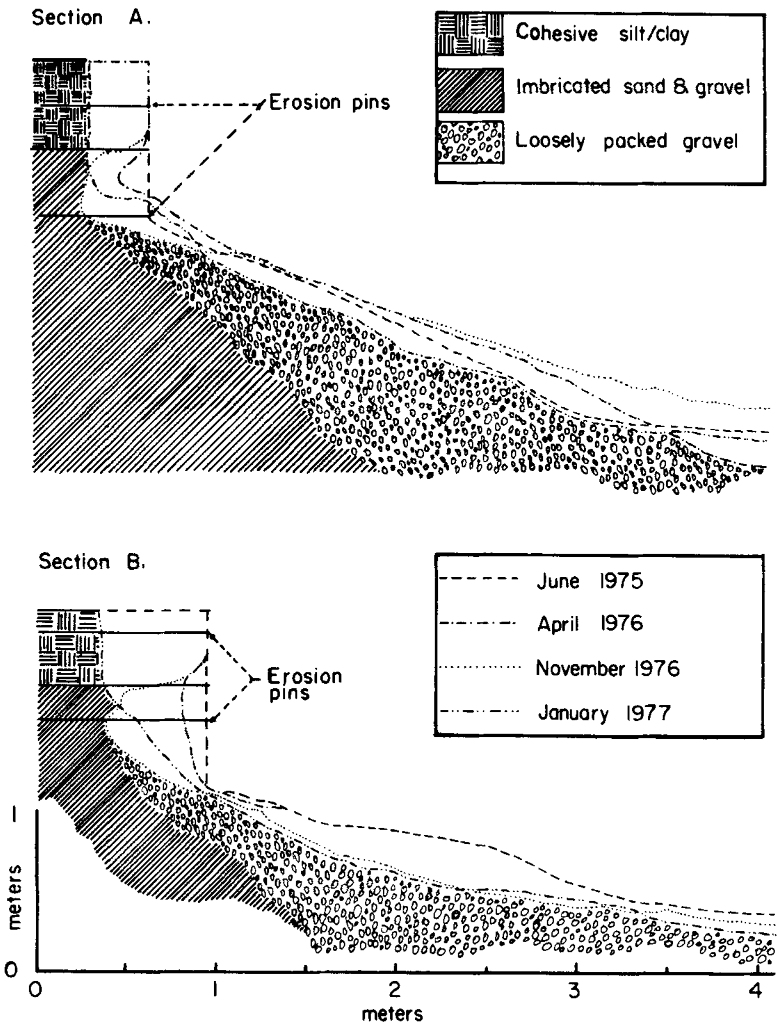

Figure 2. Generation of a cantilever in a composite bank. River Severn at Morfodian, Powys, Wales

deep inside the bank. This is the case because in a cohesive material shear stress usually increases more quickly with depth than does shear strength (Terzaghi and Peck (1948; Carson and Kirkby (1972)). Stability analyses based on static equilibrium and soil mechanics have been developed for planar, circular and log spiral failure surfaces. Several versions of the fundamental methods of analysis are available. They have different degrees of accuracy depending on the different assumptions and simplifications adopted regarding the forces acting on elements of the bank (Taylor (1948); Bishop (1955); Craig (1974)). Cohesive materials are seldom well drained and consequently pore water pressure can be an important factor in bank stability. Positive pore pressures can develop in a poorly drained river bank during rapid drawdown in the channel. This may be taken into account in the more sophisticated stability analyses which are expressed in terms of effective stress (Terzaghi and Peck (1948); Bishop (1955); Craig (1974)).

Dimensionless stability charts for circular slip failures in banks under drained and undrained conditions have been developed from the static equilibrium analyses (Bishop and Morgenstern (1960); Morgenstern (1963); Ponce (1978)). However, the applicability of these charts to natural river banks is limited because failure surfaces in alluvial banks are seldom circular (Thorne and Tovey (1979)).

In analysing the stability of cohesive banks, it is important to take into account the weakening effect of tension cracks. Tension cracks can develop downwards from the ground surface some distance behind the edge of the bank because of tensile stress in this region. The maximum depth to which they may develop can be predicted from the engineering properties of the soil (Terzaghi and Peck (1948)).

Tension cracks reduce the effective length of the potential failure surface and decrease bank stability, but they do not invalidate the stability analyses provided that the depth of tension cracking is small compared with the bank height. Although this is the case for the great majority of slopes dealt with in civil engineering, it is not the case for many alluvial river banks. In a low river bank a tension crack may occupy a significant proportion of the bank height and then failure takes place by the shearing and toppling of a block forward into the channel, rather than by circular slip with back tilting of the upper surface (Figure 3). Field observations of the erosion of cohesive bank material support this assertion (Turnbull, Krinitzsky and Weaver (1966); Brunsden and Kesel (1973); Imeson and Jungeris (1977); Bradford and Piest (1977)). Under these circumstances, slope stability analyses for rotational slip failures cannot be used to predict bank stability. Furthermore, the stability of such banks will depend more on the tensile strength rather than on the shear strength of the soil.

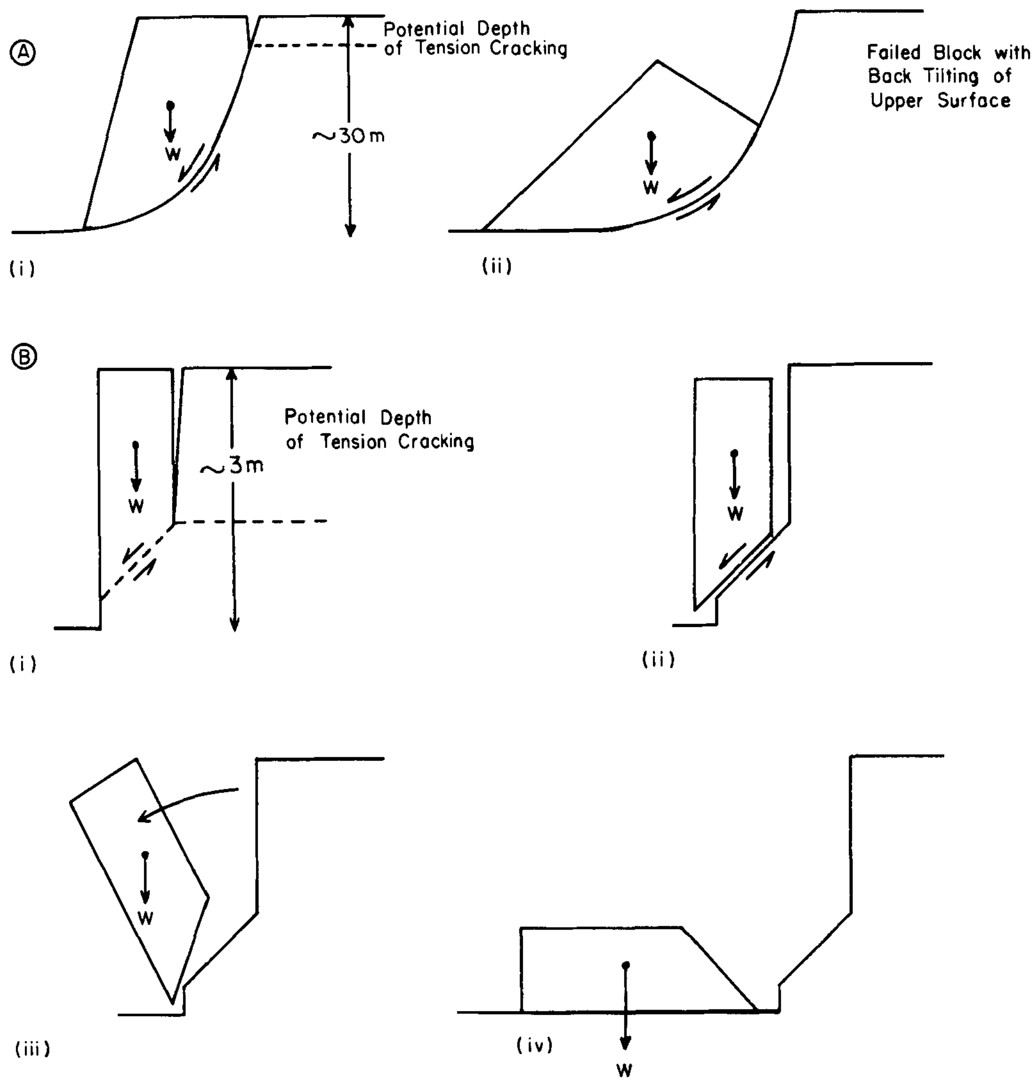

Figure 3. Failure mechanisms in cohesive bank materials. (a) Circular slip on a high bank. (b) Toppling failure on a low bank

Field observations

In a qualitative study of bank erosion processes on the lower Mississippi, Turnbull, Krinitzsky and Weaver (1966) encountered banks with a composite structure. These banks were formed in a coarse non-cohesive substratum with a cohesive top stratum. Scour of the bed and lower bank caused subaqueous failures of the lower bank due to oversteepening. Several of these lower bank failures caused failure of the upper bank by their cumulative effect.

The mechanism of failure of the upper bank was related to the thickness of the cohesive layer. Deep rotational slips were observed where this layer was thick, but where it was thin failure took place by blocks of top stratum shear off and toppling over into the river. These descriptions seem consistent with the rotational slip and toppling mechanisms of failure outlined in the previous section.

Thorne (1978) and Hooke (1979) observed composite bank erosion on gravel-bed rivers in Powys, Wales and Devon, England. The banks of these rivers are much lower than those of the Mississippi and are perhaps typical of the banks of most gravel-bed rivers. In these studies erosion pins of the type described by Wolman (1959) were used to monitor erosion processes and rates of retreat.

Hooke (1979) reported two principal processes of retreat: fluvial entrainment (corrasion) and bank collapse, or slumping. The gravel of the lower bank was more easily eroded than the silty upper bank leading to some undercutting. Overhangs, often root bound, collapsed later and were removed by subsequent high flows. Slumping occurred by the toppling forward of blocks of soil with characteristic dimensions of $1{\cdot}5 \times 1 \times 0{\cdot}75$ m (length, height, width).

Thorne (1978) installed erosion pins in the upper and lower bank layers at fourteen sites on the River Severn. The erosion pin data show different rates of fluvial erosion for the cohesive and non-cohesive layers. For example, data from two bank sections at the Morfodian site are shown in Figure 2. In the period from June 1975 to November 1976, the non-cohesive lower bank retreated by 385 and 550 mm, respectively at sections A and B. In the same period only about 20 mm of erosion of the cohesive upper bank occurred. These rates are typical of those observed throughout the reach which were generally in the range of 200 to 350 mm y^{-1} for the lower bank and 15 to 30 mm y^{-1} for the upper bank. At the outer bank at bends rates of undercutting were, exceptionally, as high as 600 mm y^{-1} (Thorne (1978); Thorne and Lewin (1979)).

The difference in the rates of retreat of the upper and lower bank produced cantilevers at both sections A and B, but at some time between November 1976 and January 1977 these cantilevers failed, producing nearly vertical upper bank sections and accumulations of debris that partially buried the lower pins.

In a survey of many similar cantilever failures at Morfodian and six other sites in the study reach, three principal mechanisms of cantilever failure were identified. These were shear, beam and tensile failure.

The three principal modes of cantilever failure are shown in Figure 4. Shear failure occurs by downward displacement of an overhanging block along a vertical plane AB, which is most probably an inter-ped fissure (Figure 4a). Failure comes about because the shear stress due to the weight of the block overcomes the shear strength of the soil.

In a beam failure a block rotates forward about a horizontal axis somewhere in the block (Figure 4b). Above the axis failure is in tension and below it, in compression. At the axis, the forces are neutral. Failure occurs because the moment of the weight of the block about the neutral axis overcomes the resistive moments of the soil's strength in tension and compression.

A tensile failure across a horizontal plane at some height above the base causes the lower part of a block to fall away (Figure 4c). This occurs when the tensile stress due to the weight of the lower part of the block overcomes the tensile strength of the soil.

Generally beam failure was the most common mechanism of cantilever collapse. Shear failures were restricted to sandy soils of low cohesion and to areas where the bank vegetation was weak or had been removed. Tensile failures were observed where the thickness of the cohesive layer was greater than about 0·8 m. Tensile failures left root bound remnant blocks, which failed subsequently by the beam mechanism.

In a tensile failure, the lower part of the block is already detached from the bank. This is very frequently the case because of the development of a vertical crack upwards from the base of the overhang, along the

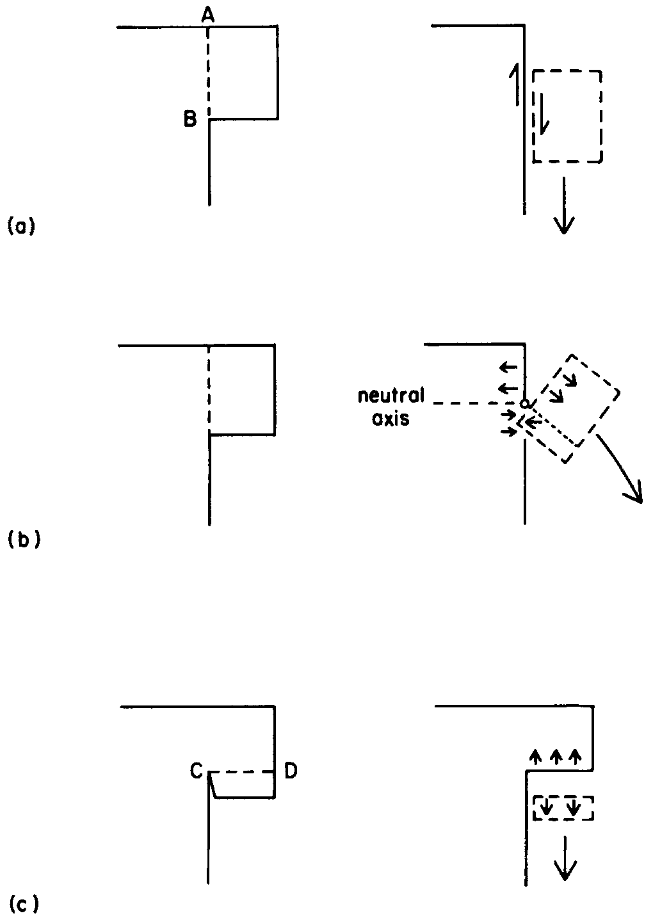

Figure 4. Mechanisms of cantilever failure. (a) Shear failure along AB. (b) Beam failure about the neutral axis. (c) Tensile failure across CD

ped boundary which is closest to the block to bank interface. This crack develops for two reasons. Primarily, it is caused by the tendency of the overhanging ped block to spring away from the bank when its exposure at the bank surface removes the lateral earth pressure from the outermost side. The opening up of fissures in clayey soils which are subject to the removal of either overburden, or lateral earth pressures is a phenomenon widely recognized in civil engineering (McGowan, Saldivar-Sali, and Radwan (1974)). A contributory effect is that of desiccation. Air drying of the overhanging block causes it to shrink and crack away from the bank. Desiccation will be most effective at the lower end of the block since any crumbs of soil that are loosened here fall way to leave an ever widening crack which does not close even when the soil is saturated.

The crack develops upwards from the base because the binding effect of vegetation roots is smaller here than just below the surface. In the surface zone roots form a mat which is very effective in reinforcing the tensile strength of the soil. The strength of the soil generally decreases with depth and reaches a minimum at the base of the root matting (about 250 mm for grassy banks). Thereafter the strength remains nearly constant or increases slightly towards the base of the cohesive layer. Consequently, many tensile failures occur at a plane corresponding with the minimum strength (i.e. a depth of about 250 mm).

A crack may also develop downwards from the top of the bank due to tensile stress. Usually this crack opens up an inter-ped fissure since this is a plane of weakness. If a dense root mat is present the binding effect in the uppermost layers of the soil inhibits the rapid development of a tension crack.

In assessing the stability of a cantilever bank the effects of tension and desiccation cracks must be taken into account. Low composite banks are usually well drained and positive pore water pressures are small. Field measurements of positive pore water pressures in low composite river banks show that they may be neglected from the analysis of cantilever stability without introducing significant error (Thorne (1978)).

CANTILEVER STABILITY

Cantilever stability may be assessed from basic principles using a static equilibrium analysis of forces and the elementary theory for the bending of beams. The full analysis is presented in a paper in preparation (Thorne and Tovey (in preparation)) and it is not proposed to replicate it here. Instead an outline of the method is included, together with some examples of the use of the analysis.

The forces of weight, shear strength, compressive strength and tensile strength which act on a cantilever block are depicted in Figure 5. In the analysis, the critical case is examined, that is the forces (motive and resistive) are exactly balanced and the block is in a state of static equilibrium. To produce the stability

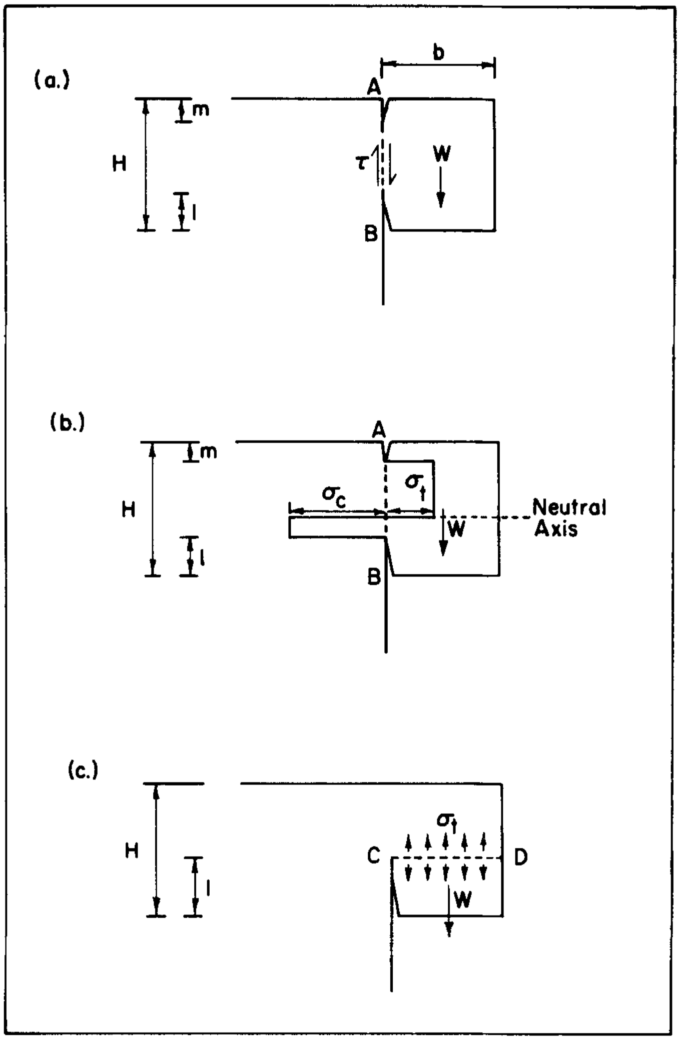

Figure 5. Forces of weight, shear, compression and tension acting on a cantilever with regard to the three modes of failure: (a) Shear failure. (b) Beam failure. (c) Tensile failure

equations (1)–(3) for the block with regard to shear, beam, and tensile failure, forces are resolved horizontally and vertically, and moments about the neutral axis are taken. The terms in the equations are then arranged into dimensionless groups so that each equation describes the factor of safety of the block with regard to one particular mechanism of failure. Some of the non-dimensional terms are common to all three equations and so they may be used to plot dimensionless charts for cantilever stability. These charts are similar in essence to those mentioned earlier for circular arc failures in high banks (Taylor (1948)) (Figure 6). The ordinate of each chart is the non-dimensional group F/A where F is the appropriate factor of safety and A is a parameter composed of σ_{tc}, the tensile strength of the soil, γ, the unit weight of the soil and b, the overhang width.

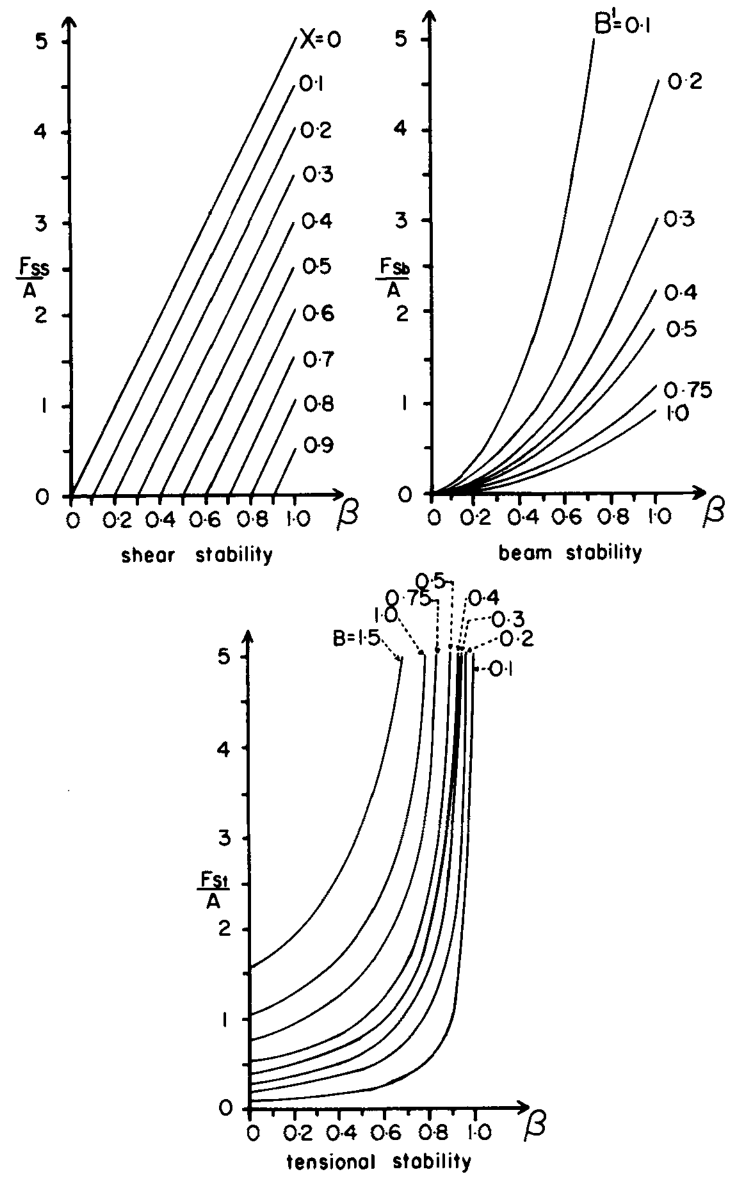

Figure 6. Dimensionless charts for cantilever stability F_{ss}, F_{sb}, and F_{st} are the factors of safety for shear, beam and tensile failure. A is a measure of stability. B and B' are measures of cantilever geometry. χ and β are measures of crack development (see equations (1)–(8))

The stability equations used to produce the charts are: for shear failure (factor of safety F_{ss})

$$\frac{F_{ss}}{A}=\frac{(\beta-\chi)}{\beta}\frac{\beta}{2r} \tag{1}$$

for beam failure (factor of safety F_{sb})

$$\frac{F_{sb}}{A}=\frac{\beta^2}{(1+r)B'} \tag{2}$$

for tensile failure (failure of safety F_{st})

$$\frac{F_{st}}{A}=\frac{B}{(1-\beta)} \tag{3}$$

where

$$A=\frac{\sigma_t}{\gamma b} \tag{4}$$

The quantities β, χ, B, and B' are dimensionless numbers depending only on the geometry of the cantilever overhang. r is the ratio of the tensile strength to the compressive strength of the soil.

$$B=\frac{b}{H} \tag{5}$$

$$B'=B\left[\frac{\beta}{\beta-\chi}\right]^2 \tag{6}$$

$$\beta=\left(\frac{H-l}{H}\right) \tag{7}$$

$$\chi=\left(\frac{m}{H}\right) \tag{8}$$

where H is the overhang height, m is the length of the upper crack and l is the length of the lower desiccation crack.

These equations, derived in Thorne (1978), could be simplified further, however, they are retained in their present form so that they can be used to produce charts that all have the same axes.

The charts shown in Figure 6 apply to a bank material which has an r value of 0·1. Any value of r between 0 and 1 could be used, but measurements of the ratio of tensile to compressive strength for soil suggest a value of about 0·1 (Ajaz (1973)). The stability of a block is in any case rather insensitive to changes in the r value. Indeed, an increase in r by a factor of 2 changes the estimate of the factor of safety by less than 10 per cent. The charts may be used to identify the most critical mechanism of failure and the limiting factor of safety by calculating values for the dimensionless parameters A, B, β and χ. This involves simple measurements of the geometry of a cantilever block and its soil properties. For example, suppose an overhanging block has the following dimensions:

$$b=0{\cdot}2\text{ m} \qquad l=0{\cdot}16\text{ m}$$

$$H=0{\cdot}4\text{ m} \qquad m=0{\cdot}04\text{ m}$$

and the soil has the following properties

$$\sigma_{tc}=8\text{ kPa} \qquad \gamma=16\text{ kN m}^{-3} \qquad r=0{\cdot}1$$

To identify the most critical mechanism, equations (5)–(8) are used to calculate values for B, B', β and χ. This yields: $B=0{\cdot}5$, $B'=0{\cdot}72$, $\beta=0{\cdot}6$, and $\chi=0{\cdot}1$.

To determine F_{ss}/A, enter the chart by locating $\beta = 0{\cdot}6$ on the abscissa. Move vertically upwards until the line $\chi = 0{\cdot}1$ is reached. This yields:

$$F_{ss}/A = 2{\cdot}5$$

Similarly, for F_{sb}/A with $\beta = 0{\cdot}6$ and $B' = 0{\cdot}72$

$$F_{sb}/A = 0{\cdot}46$$

Also

$$F_{st}/A = 1{\cdot}25 \text{ (for } \beta = 0{\cdot}6 \text{ and } B = 0{\cdot}5)$$

Since A is a common factor, F_{sb} has the lowest value; hence it is defined as the most critical factor of safety and a beam failure is the most likely mechanism of failure. The actual factor of safety is then calculated by determining a value for A from equation (4). Here,

$$A = 2{\cdot}5 \text{ and } F_{sb} = 0{\cdot}46A = 1{\cdot}15$$

This factor of safety indicates that the overhang would just be stable even against the most likely mechanism of failure.

A cantilever can be brought to failure in a number of ways. Continued erosion and retreat of the lower bank increases the overhang width, b. This decreases the factor of safety with respect to both shear and beam failures, but leaves unchanged the factor of safety for tension failure. Whenever the factor of safety for a particular mechanism approaches unity, failure is imminent in that mode.

Once the basic geometry (i.e. width and depth) of the overhanging block has been established, failure can also arise by increasing the length of the upper crack (increasing χ), increasing the length of the lower crack (decreasing β), or increasing the unit weight of the soil through saturation. Conversely block stability can be increased by a reduction of the unit weight by drying or submergence. For instance, in the example above, a value of 16 kN m^{-3} was used for the unit weight. After precipitation, or immersion of the bank during a flood, the saturated unit weight could increase to 19·8 kN m^{-3}. Actually during immersion, the operative unit weight is the submerged unit weight (γ') defined by:

$$\gamma' = \gamma - \gamma_w$$

where γ_w is the unit weight of water (=9·81 kN m^{-3}). Thus $\gamma' = 19{\cdot}8 - 9{\cdot}8 = 10$ kN m^{-3} and the factor of safety with respect to beam failure increases from 1·15 to 1·84 during immersion, representing a considerable increase in stability. When the river level recedes, the block changes from the submerged to the saturated state, the factor of safety is reduced by about half to 0·93, and beam failure occurs. Clearly, the increase in bulk unit weight due to the change from submerged to saturated conditions on drawdown can often be critical to cantilever stability. The additional motive forces of fluid drag and shear which act on a cantilever during submergence seldom exceed 5 per cent of the weight and are insufficient in themselves to seriously affect the block stability (Thorne (1978)). Increasing moisture content usually reduces the strength of a soil. The combined effect of increasing the unit weight whilst decreasing the strength of the soil can cause failure due to wetting. Conversely drying the bank can also lead to failure owing to an increase in the length of the desiccation crack. Field observations suggest that failure often results from the development of this crack up from the base (Thorne (1978); Thorne and Tovey (in preparation)). Finally the development of a tension crack downwards from the top of the bank will lead rapidly to failure. As in the case in circular arc slips, tension cracks in cantilevers seem to develop only when the overhang is already close to failure.

The accuracy of the stability charts was tested using data from cantilevers cut out of composite banks and brought to failure under controlled conditions. It had been intended to use field measurements for this purpose, but it became apparent during the study that the erosion pins installed at the test sites were very effective in holding up the cohesive layer and producing unnaturally wide cantilevers. Although this did not seriously affect the data for rates of surface erosion it did prevent the use of the measurements for testing the stability charts.

Table I. Cantilever stability calculations: River Severn near Maes Mawr. 15–17 September 1977

Width (b) metres	Height (H) metres	χ	β	F_{ss}	F_{sb}	F_{st}	Field notes
0·3	0·4	0	1	10·70	2·02	∞	Time: 3:00 pm, 15th block stable
0·3	0·4	0	0·8	8·56	1·29	6·10	Time: 3:15 pm, 17th desiccation/stress release crack 80 mm extent. Block stable.
0·3	0·4	0·125	0·8	7·22	0·92	6·10	Time: 3:30 pm, 17th Tension crack noted. Extent 5 mm. Failure by beam failure shortly after 3:30 pm

$\gamma = 15{\cdot}3\ \mathrm{kNm^{-3}}$ $\sigma_c = 98$ kPa $\sigma_t = 7{\cdot}5$ kPa ($r = 0{\cdot}08$)

In Table I, data for a cantilever cut out of the bank near Maes Mawr on the River Severn in Wales are presented. Over a period of 48 hours after cutting a desiccation/stress release crack developed upwards from the base, between the overhanging block and the bank. Then a tension crack began to develop vertically downwards from the ground surface at the top of the cantilever. A few minutes later failure occurred by the beam mechanism. At the time of failure, the vertical extent of the desiccation/stress release crack was 80 mm and that of the tension crack 5 mm. Both extended along the whole length of the block, which was 0·8 mm.

Immediately after failure, soil cores were taken and the unit weight, compressive strength and tensile strength determined (Thorne, Tovey and Bryant (1980)). These data were used with the stability equations and charts to calculate the factor of safety of the cantilever with respect to shear, beam and tensile failure. Consideration of the results, in Table I shows a good agreement with the field observations, a beam failure being predicted by the F_{sb} value of 0·92.

Many similar tests were carried o' t and the results are presented in detail in a paper in preparation (Thorne and Tovey (in preparation)). The results of these tests show that the stability charts may be used to identify the critical failure mechanism with a high degree of confidence and that factors of safety may be predicted to within ten or fifteen per cent of the true value.

REMOVAL OF FAILED BLOCKS

Following cantilever failure, blocks of upper bank soil fall on to the lower gravel bank. The blocks may break up on impact or may remain intact. If the fall height is great, or the lower bank steep, blocks roll directly to the foot of the bank. On gentle slopes, or where midslope benches are present, they may come to rest at intermediate points on the lower bank (Brunsden and Kesel (1973)).

The block is now no longer strongly bonded to the bank by cohesion or vegetation, although it may become so in time. It rests in place under its own weight and any cohesion between its underside and the gravel of the lower bank. For there to be further movement one of these materials, or the interface between them, must fail. Movement may take place by sliding or by rolling, depending on the geometry of the block and its orientation relative to the flow in the channel. Field observations indicate that in the case of sliding, movement usually occurs by the development of a shear failure at the block to bank interface. Shear failure in the block itself is unlikely. Rolling failure occurs by pivoting of the block about its long axis. The probability of rolling depends largely on the ratio of height to breadth, but the orientation of the block relative to the flow is also important as this controls whether fluid drag and downslope weight moments act together or in opposition. A high ratio of height to breadth favours rolling but as soon as rotation through an angle $\pi/2$ has occurred, the dimensions are transposed. In the case of an initially high height to breadth ratio the block in its new position will be very stable with regard to rolling. Only blocks with height to breadth ratios close to unity will experience sustained rolling, while blocks with dissimilar height and breadth dimensions will move by sliding with their largest face downwards.

Stability equations for blocks on slopes may be developed using static equilibrium (Thorne (1978)). Such equations reveal that bank angle is the most important single parameter in determining block stability. High up on the bank where the bank angle is steep blocks are rather unstable and so their residence times are short. As a block moves downslope the bank angle decreases until the channel bed is reached. The stability of a block increases as a result and without fluid forces it would remain in place indefinitely near the toe of the bank.

While in position on the lower slope, failed ped blocks are subject to subaerial, subaqueous and fluvial processes of weathering and erosion. These processes tend to increase block stability significantly by improving coplanarity, and consequently cohesion between the base of the block and the bank and by reducing the drag coefficient by rounding and streamlining the block. Also, especially for blocks that are lodged above the mean daily flow level, stability will increase as vegetation becomes established. Conversely, weathering and erosion also act to decrease stability by reducing block size through cracking and soil removal, but usually the stability of a block on the lower bank increases rather than decreases with time. It therefore becomes increasingly difficult for the river to remove failed ped blocks from the bank toe. In time, if the river is unable to remove them, blocks cease to have an entity of their own and become part of the lower bank. This has important implications for bank erosion processes and bank retreat.

For fluvial erosion of the gravel of the lower bank to continue, failed cantilever blocks must be removed from the lower bank and toe. If these blocks are not removed, they break down to produce a cohesive piedmont that protects the underlying gravel from further erosion. This reduces the rate of undercutting and cantilever generation, leading to stabilization of the upper bank. In time, the whole profile will grade back to a stable state. Clearly, the form of the bank profile and its retreat rate are fluvially controlled, even though the dominant erosion process operating on the upper layer (cantilever failure) is not directly fluvial in nature.

Once a block is entrained by the flow its size is reduced rapidly by abrasion, and its tendency to break up under its own weight. The soil disintegrates quickly to wash load. Blocks that fail during high discharge and that fall directly into rapidly flowing water are unlikely to come to rest on the lower bank. At about one-third bankfull stage (about 25 m^3s^{-1}) on the River Severn at Caersws, Powys, Wales, blocks that failed were carried distances of 5 to 15 m downstream before coming to rest. While in motion soil blocks tended to become orientated with their long axis across the flow facilitating motion by rolling along the bed. During transit blocks were observed to begin to break up and disintegrate.

IMPLICATIONS OF COMPOSITE BANK EROSION PROCESSES

The cycle of erosion of low composite banks (undercutting, cantilever formation, cantilever failure and the removal by the channel flow of failed blocks of soil from the basal area) has some important implications for bank protection schemes, channel pattern changes and sediment movement in gravel bed rivers.

The stability of a composite bank depends on internal forces of weight, friction, and cohesion, and on the external forces of fluid drag and shear. The rate of bank retreat depends on the sediment balance at the bank toe. It is important to recognize these facts when protecting and stabilizing composite banks. For example, Turnbull, Krinitzsky, and Weaver (1966) noted that the building of bank revetments on the Mississippi led to deep scouring of the bed near the bank toe. This scouring eventually caused deep seated rotational bank failures due to oversteepening.

Scour, or deposition, at the toe of a bank results from an imbalance between the quantity of sediment supplied to the basal area by bank erosion and removed from it by fluvial entrainment. If the sediment supply is reduced drastically by bank protection, as it was on the banks of the Mississippi there is a tendency for the bed to scour to maintain the sediment balance. This tendency for basal scour will be reinforced if the design or alignment of a bank protection structure also increases the velocity gradient and boundary shear stress at the toe, increasing the local sediment transport capacity of the flow. These facts explain why close attention must be paid to alignment and toe protection if a bank protection scheme is to be successful.

The cycle or erosion, failure and basal scour described here helps to explain the complex relationships observed between the magnitude of a flood event and its effectiveness in eroding the banks and altering channel plan shape (Hooke (1979); Thorne and Lewin (1979)).

The high susceptibility of lower bank gravel to fluvial erosion, can result in appreciable undercutting at flows well below bankfull. During trail releases from the Clywedog Reservoir, Powys, Wales, in June, 1975 at the most active bends in the experimental reach of the River Severn (between Llanidloes and Caersws, Powys) the rate of undercutting was about 50 mm per day. The discharge (26 $m^3 s^{-1}$) was about one-third bankfull capacity. At flow stages of this scale secondary currents have their greatest effect on the boundary shear stress distribution at a bend and this makes the flow particularly effective in attacking the outer bank (Bathurst, Thorne and Hey (1979)).

The limiting overhand width that can develop in the upper bank depends on the engineering properties of the soil and the thickness of the cohesive layer. Hooke (1979) noted that overhangs on Devon streams were 100–200 mm in width, whereas Thorne (1978) observed overhangs of 400–500 mm on the River Severn.

This suggests that several high stages, or a prolonged period of high flow, are required to undercut a composite bank sufficiently to cause cantilever failure due to increased overhang width actually during a flood. However, field observations suggest that most cantilevers fail sometime after the peak flow, having been brought to failure in other ways than by increased width (Thorne (1978); Hooke (1979)). Some of these failures are related to the flow stage (for example, failure due to the change from submerged to saturated conditions on drawdown), others, such as failure due to crack development, are related to climatic and soil mechanics parameters. Blocks which fail some time after undercutting by a high stage must be removed from the toe area by subsequent high flows. This shows that the cycle of erosion, failure, and removal operates over several flow events, so that no simple relationship between flood magnitude and bank retreat should be expected. The main effect of high flows is probably in scouring basal accumulations of bank debris and undercutting the bank by eroding the gravel. The stability of the cohesive upper bank is more closely related to climate and soil mechanics.

An important exception may be events of great magnitude but low frequency. They tend to drown out features of the bed and to alter the distribution of basal scour to produce bank failures in normally stable areas. In this way, radical changes in planform and unusual fluvial landforms, such as unsedimented scour holes and abandoned sections of cut bank, can be produced (Thorne and Lewin (1979)).

There is evidence that at least in the case of gravel-bed rivers with armoured beds, in well vegetated catchments, the channel banks may be the major source of sediment load (Simons and Chen (1977)). Consequently, the cycle of erosion, failure, and basal removal might significantly affect the relationship between discharge and sediment load.

Coarse bed load comes from the gravel deposits of the lower bank while suspended load is derived from the silt/clay upper bank and from the breakdown of soil blocks at the toe. The availability of material of any particular type for entrainment by a given flow depends in a rather complex way on such factors as the amount of basal accumulation of bank debris since the last flood, the nature of the debris (soil crumbs or gravel particles from surface loosening, or soil blocks from cantilever failures), and the degree of armouring of the surface of the non-cohesive lower bank by gravel lag or soil blocks. All this in turn depends on parameters of climate and soil mechanics and on the time elapsed since the bank toe was last scoured. This might explain why antecedent conditions provide some explanation of the variation in sediment load between flows of similar magnitude and hydraulic properties.

CONCLUSIONS

Composite banks with non-cohesive sand and gravel overlain by cohesive sandy silt/clay are widely observed on rivers flowing through alluvial deposits. Rapid erosion of the lower bank by fluvial entrainment generates cantilevers in the cohesive upper layer of eroding banks. Three principal mechanisms of cantilever failure have been identified. These are shear, beam and tensile failure. The stability of such cantilevers may be analysed using static equilibrium and elementary beam theory. The stability equations for each mechanism of failure may be used to construct dimensionless stability charts like those of Taylor

(1948) for slopes. These charts yield factors of safety which are within 10 or 15 per cent of the true values. The most critical failure mechanism can be predicted from cantilever geometry alone.

Cracks due to stress release, desiccation and tension can develop rapidly between the block and bank. They must be taken into account in any assessment of bank stability. Failed blocks of soil are removed from the bank toe by fluvial entrainment by rolling or sliding. This removal completes the cycle of erosion of the composite bank. The balance between rates of sediment supply and removal from the bank toe controls the bank's profile and rate of retreat. As this balance depends on fluvial processes, the profile and retreat rate are fluvially controlled even though the dominant failure mechanism of the upper bank is not directly fluvial in nature. The cycle of erosion, failure, and basal removal described here has some important implications for bank protection, channel changes and sediment movement in gravel-bed rivers.

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