

# Field Validation of An Improved Process-Based Model for Streambank Stability Analysis

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## Abstract

Streambank erosion is a major contributor to total erosion in disturbed landscapes. Identification of the dominant streambank erosion processes and their conceptualization in mathematical width adjustment models remain very difficult tasks. The U.S. Department of Agriculture, Agricultural Research Service, National Sedimentation Laboratory (NSL) conducts field studies to characterize the impact of pore-water pressures on failure dimensions and shearing resistance, and the role of riparian vegetation on matric suction, streambank permeability, and shearing resistance. Results from these studies are used to enhance the NSL's long-term channel evolution model CONCEPTS. This paper discusses the implementation of the above physical processes for bank failures along planar slip surfaces, and demonstrates the ability of CONCEPTS to simulate streambank failure processes. The proposed streambank-stability scheme accurately predicts timing and dimensions of bank failures along a bendway in the Goodwin Creek, Mississippi between March 1, 1996 and March 31, 1997. Differences between predicted and observed top-bank retreat (up to 40%) are caused by the observed cantilever failure of the streambank, yielding a large failure plane angle.

## Introduction

Channels adjust over time to variations in loadings of water and sediments. Streams can adjust in two directions: vertically (aggradation and degradation) and horizontally (shoreline erosion and lateral migration). These morphological adjustments may occur separately or in unison. The adjustment of channel width by mass-wasting and related processes can represent an important mechanism of channel response and energy dissipation in incised alluvial streams; bank material in the loess area of the Midwest United States contributes as much as 80 percent of the total sediment eroded from incised channels (Simon et al. 1996).

Streams draining the Bluff-line hills of north-central Mississippi have exhibited serious instability in response to catchment landuse changes and river management for drainage and flood control. The fluvial system initially responded through incision, which adversely affected the stability of the streambanks with respect to mass wasting by increasing bank heights and angles and triggered rapid widening (Decoursey 1981). Under the Demonstration Erosion Control Program, initiated by the federal government in 1984, the NSL and other federal agencies have been studying these streams to develop technology for channel stabilization and rehabilitation. Ongoing research aims at including improved bank-failure algorithms in channel

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evolution models to evaluate the effectiveness of stream corridor restoration designs, among others.

This paper presents continuing research on an improved, process-based bank failure algorithm and its implementation in the channel evolution model CONCEPTS (CONservational Channel Evolution and Pollutant Transport System). CONCEPTS simulates unsteady, one-dimensional flow, graded sediment transport, channel widening, and hydraulic and geomorphic effects of instream vegetative and structural remediation measures in watershed-scale channel networks or corridor-scale stream channels (Langendoen et al. 1996, 1998).

## **Streambank Failure Mechanics**

Erosion of cohesive streambanks is a combination of: (1) lateral erosion of the bank toe by fluvial entrainment of bank material particles, and (2) mass failure of the bank. Osman and Thorne (1988) present an analysis on the rate of lateral erosion. Here, we will address the second erosion process of streambank failure.

Streambank failure occurs when gravitational forces that tend to move soil downslope exceed the forces of friction and cohesion that resist movement. The risk of failure is usually expressed by a factor of safety defined as the ratio of resisting to driving forces or moments. Banks may fail by four distinct types of failure mechanisms (ASCE 1998): (1) planar failures, (2) rotational failures, (3) cantilever failures, and (4) piping and sapping failures.

Steep banks commonly fail along planar failure surfaces, with the failure block sliding downward and outward into the channel before toppling (Thorne 1982). High, mildly-sloped streambanks (bank angle less than  $60^\circ$ ) usually fail along curved surfaces; the block of failed material tends to rotate towards the bank as it slides. Cantilevered or overhanging banks are generated when erosion of an erodible layer in a stratified bank leads to undermining of overlying, erosion-resistant layers (Thorne and Tovey 1981). Streambanks may also fail by exfiltrating seepage known as piping or sapping (Hagerty 1991). Streambanks in the Midwestern and Midsouthern United States are generally very steep, failures occur mostly along planar failure surfaces (Osman and Thorne 1988). Hence, the stability analysis set forth in the following section assumes planar failure.

The bank's geometry, soil properties, pore-water pressures, confining pressure exerted by the water in the stream, and riparian vegetation determine the stability of the bank. The soil properties are bulk weight of the bank material and the shear strength parameters cohesion and internal angle of friction. The shear strength parameters represent factors such as density, void ratio, degree of saturation, mineral composition, stress history, and strain rate.

Positive pore-water pressures in saturated streambanks reduce the shear strength to resist mass failure. Bank failures have been most commonly reported during the recessional period of stormflow. This has been attributed to a 'rapid-drawdown' condition where positive pore-water pressures in the channel banks are not counteracted by confining pressures exerted by the streamflow. Conversely, Simon et al. (1999b) show that the loss of negative pore-water pressures, or matric suction, plays an important role in initiating bank instabilities following

periods of rainfall. Incised streams generally have high banks where the water table may be deep relative to the bank height during low and moderate channel stages. Above the water table the soil is unsaturated and negative pore-water pressures develop increasing the shear strength of the bank material. Therefore, slope stability analyses need to account for matric suction.

The effects of riparian vegetation on streambank erosion are varied and complex. The impact of vegetation on streambank stability may be both positive and negative. Roots mechanically reinforce soil by transferring shear stresses in the soil to tensile stresses in the roots (Gray and Leiser 1982). When bank height significantly exceeds rooting depth, and consequently the failure surface may pass beneath the vegetation rootballs, root reinforcement is negated. Vegetation decreases soil moisture levels by interception on the canopy and evapotranspiration. Conversely, roots may create pathways for water to enter the streambank, increasing pore-water pressures. The surcharge weight of vegetation may increase motivating forces, and wind loading of tall vegetation may exert an additional destabilizing moment on the bank. These vegetative effects are as yet poorly understood and, thus, not incorporated in the following analysis. Simon et al. (1999a) present an approach to take into account root reinforcement on bank material shear strength.

## Streambank Stability Analysis

Streambank stability can be analyzed using methods developed for engineered slopes and embankments, e.g., Bishop (1955), Morgenstern and Price (1965), and Fredlund and Krahn (1977). These limit equilibrium methods are based upon the static equilibrium of forces and/or moments. Simon et al. (1999b) have presented a comprehensive stability analysis for layered streambanks based on the limit equilibrium method. They included forces due to pore-water pressures and confining pressures. We have extended their stability analysis by accounting for the seepage force in the factor of safety and enhancing the implementation of confining pressures, though presently for homogeneous bank material.

Following Huang (1983) the surface water on the failure block is modeled by assuming it is a material with no strength (cohesion and angle of internal friction are set to zero). Hence, the slip surface is extended vertically through the water, and a horizontal hydrostatic force is applied on the vertical portion of the slip surface (see Figure 1). The forces acting on the failure block are (Figure 1): (1) the weight of the failure block,  $W_S$ ; (2) the weight of surface water on the failure block,  $W_w$ ; (3) the hydrostatic force exerted by surface water on the vertical slip surface,  $F_w$ ; (4) the hydrostatic force exerted by water in the tension crack,  $F_t$ ; (5) the seepage force,  $F_s$ ; (6) the shear force at the base of the failure block,  $S$ ; and (7) the total normal force at the base of the failure block,  $N$ .

The shear strength equation for either saturated or unsaturated soils (Fredlund and Rahardjo 1993) expresses the shear force acting at the base of the failure block as:

$$S = \frac{L}{F} \left( c' + (\sigma_n - u_a) \tan \phi' + (u_a - u_w) \tan \phi^b \right), \quad (1)$$

where  $\sigma_n$  is total stress normal to the base of the failure block,  $u_a$  is pore-air pressure,  $u_w$  is pore-water pressure,  $c'$  is the effective cohesion,  $\phi'$  is the effective angle of internal friction associated with the normal stress state variable  $(\sigma_n - u_a)$ ,  $\phi^b$  is an angle indicating the increase

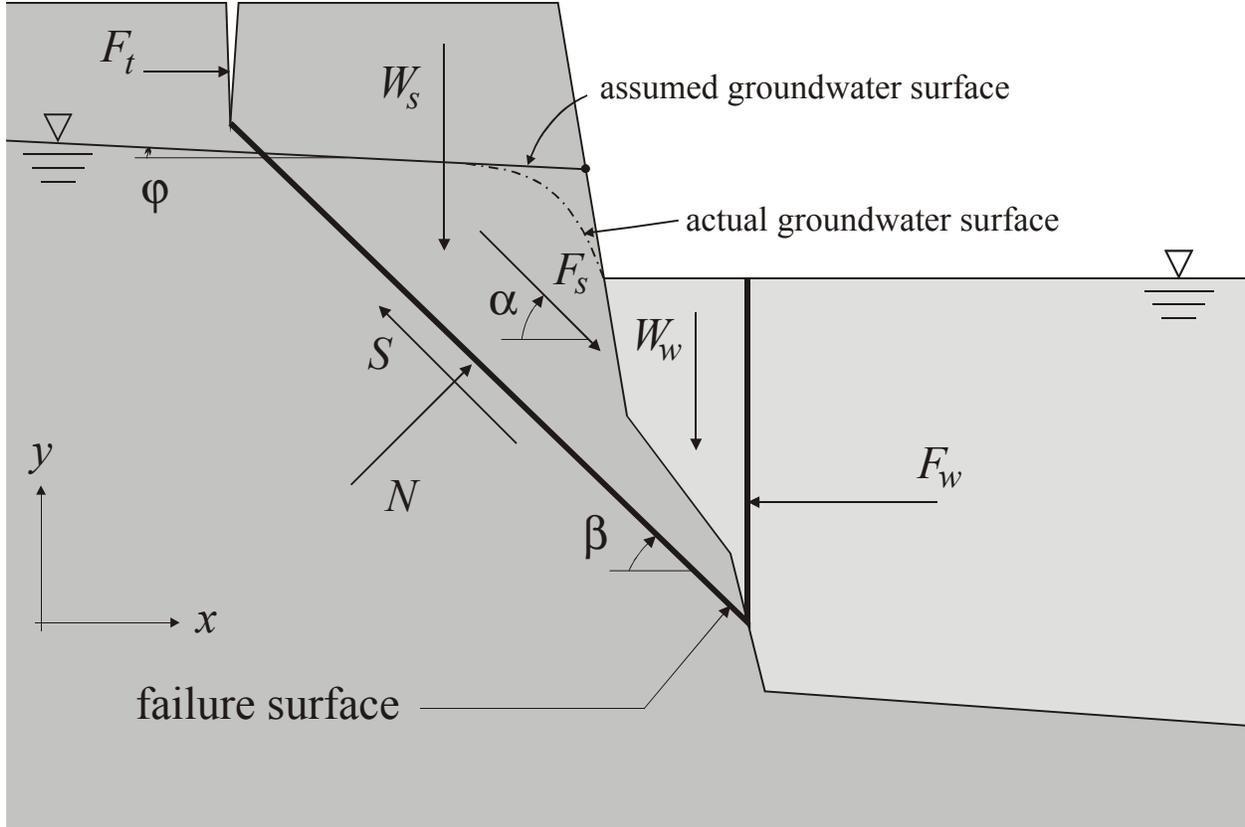


Figure 1 Definition sketch of forces acting on a streambank.

in shear strength for an increase in matric suction ( $u_a - u_w$ ),  $L$  is the length of the inclined slip surface, and  $F$  is the factor of safety which is defined as the factor by which the shear strength parameters must be reduced in order to bring the soil mass into a state of limiting equilibrium along the assumed slip surface.

Equilibrium of forces in horizontal and vertical directions yield:

$$\mathbf{F} = \frac{c_a L + ((W_s + W_w) \cos \beta + (F_w - F_t) \sin \beta) \tan \phi'}{(W_s + W_w) \sin \beta - (F_w - F_t) \cos \beta}, \quad (2)$$

where apparent or total cohesion is (Fredlund and Rahardjo 1993)

$$c_a = c' - \frac{1}{L} (r_u (W_s + W_w) \cos \beta \tan \phi' - \Psi \tan \phi^b), \quad (3)$$

and the pore-water pressure ratio is:

$$r_u = \frac{(\text{volume of sliding mass under water} + \text{volume of surface water on bank}) \times \gamma_w}{\text{volume of sliding mass} \times \gamma_b + \text{volume of surface water on bank} \times \gamma_w}. \quad (4)$$

We can also use (2) to determine the stability of layered streambanks by using a weighted, average cohesion and angle of internal friction for the heterogeneous bank material. However,

we cannot readily extend (2) to account for the strength of each soil in a layered streambank. In this case we have to revert to the method of slices (Fellenius 1936).

Darby and Thorne (1996) followed Taylor (1948), who stated that the most dangerous failure surface is that with the maximum stability number  $N_s = \gamma_b H / c_d$ , where  $H$  is height of failure block and  $c_d$  is developed cohesion, or equivalently the failure surface for which cohesion is fully mobilized, that is,  $\partial c_d / \partial \beta = 0$ . However, Darby and Thorne (1996) assumed  $c_d = c'$ . Effective cohesion is primarily determined by soil structure and soil moisture, and therefore may not vary much with failure plane angle. Hence, Darby and Thorne's approach of rewriting (2) to yield an expression for effective cohesion, and equating the derivative of this expression with respect to failure plane angle to zero, is unrealistic. Moreover, the factor of safety for the resulting failure plane angle is not a minimum.

Here, the inclination of the failure plane is that for which the factor of safety is minimum, that is,  $\partial FS / \partial \beta = 0$ . Taking the derivative of (2) with respect to the failure plane angle gives

$$\begin{aligned} \frac{\partial F}{\partial \beta} = & \left\{ L \frac{\partial c_a}{\partial \beta} [(W_s + W_w) \sin \beta - (F_w - F_t) \cos \beta] \right. \\ & - c_a L \left[ \frac{\partial (W_s + W_w)}{\partial \beta} \sin \beta + 2(W_s + W_w) \cos \beta - (F_w - F_t) \frac{\cos 2\beta}{\sin \beta} \right] \\ & \left. - \left[ (W_s + W_w)^2 + (F_w - F_t) \frac{\partial (W_s + W_w)}{\partial \beta} + (F_w - F_t)^2 \right] \tan \phi' \right\} \\ & / ((W_s + W_w) \sin \beta - (F_w - F_t) \cos \beta)^2. \end{aligned} \quad (5)$$

Equating (5) to zero yields the failure plane angle.

On the other hand, following Taylor's (1948) assumption, one obtains the following failure plane angle:

$$\beta = \frac{1}{2}(\theta + \phi'), \quad (6)$$

where  $\theta$  is average bank angle. This is the form used by Osman and Thorne (1988) and Simon et al. (1991, 1999b) in their analyses. The failure plane angles obtained from (5) and (6) are different. The calculated factor of safety using the failure plane angle (6) is not a minimum.

## Goodwin Creek Bendway

Since February 1996 NSL conducts extensive research on streambank failure mechanics along a bendway of the Goodwin Creek, northern Mississippi (Simon and Darby 1997). The following data are being collected: cross section geometry, water surface elevations, bank material properties, bank material shear strength parameters, pore-water pressures in the bank, root mapping and tensile strength, and plant stem flow. Two flow measuring flumes (flumes #3 and #4, Alonso 1997) in upstream tributaries provide continuous discharge and fine sediment data. Figure 2 shows a plan view of the bendway with the locations of ten surveyed cross sections. The flow is from top to bottom.

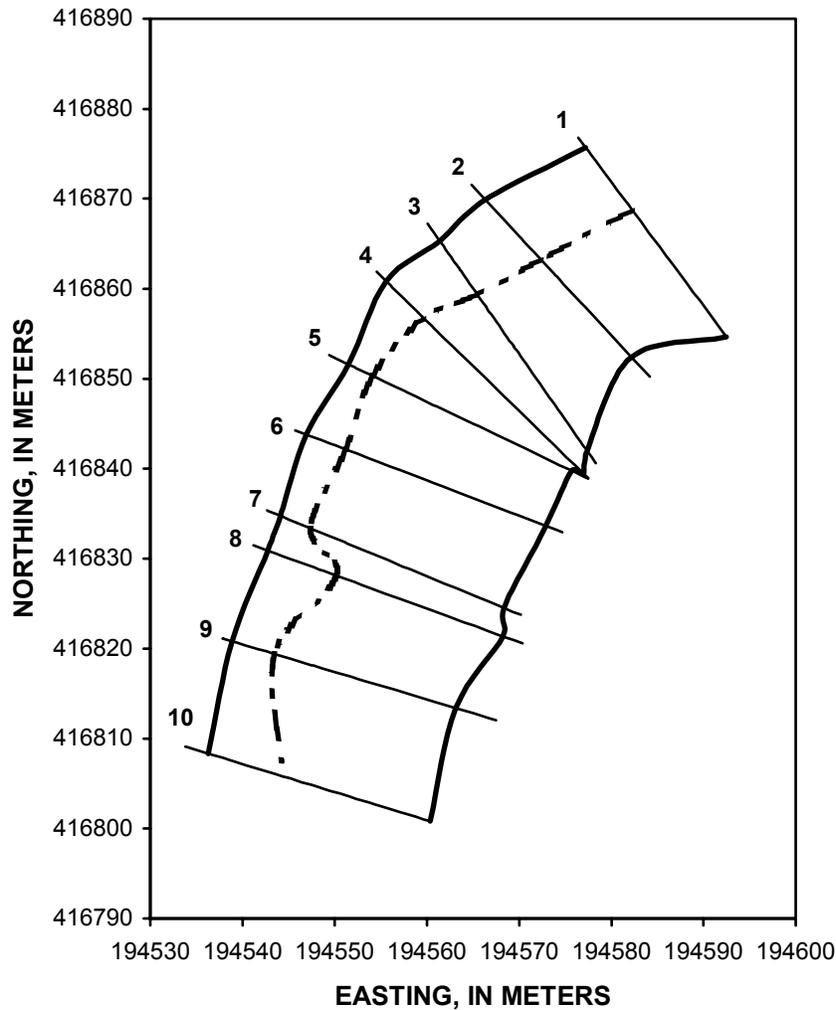


Figure 2 Plan view of the Goodwin Creek Bendway site showing the locations of the ten cross sections. Flow is from top to bottom. The dashed line is the channel thalweg.

Bank material consists of about 2 m of brown, clayey-silt of late Holocene age (LH unit) overlying 1.5 m of early Holocene gray, blocky silt of lower permeability (EH unit). These units are separated by a thin (0.1 to 0.2 m) layer containing manganese nodules and characterized by very low permeability, which perches water. These materials overlie 1 m of sand and 1.5 m of packed sandy gravel.

Apparent cohesion and effective friction angle were measured *in situ* using an Iowa Borehole Shear Tester (Luttenegger and Hallberg 1981). For the LH unit results revealed an apparent cohesion varying between 0.0 and 8.4 kPa with an average effective friction angle of 28.1°. The underlying EH unit has apparent cohesion values varying between 0.0 and 37.9 kPa with an average friction angle of 28.5° for the period between June 1996 and July 1998 (Simon et al. 1999b). Simon et al. (1999b) provide the following values for effective cohesion and angle  $\phi^b$ : LH unit,  $c' = 2.67$  kPa and  $\phi^b = 10.4^\circ$ ; and EH unit,  $c' = 13.0$  kPa and  $\phi^b = 17.5^\circ$ .

Four major failure episodes occurred at the research site between February and December 1996, resulting in up to 2 m of top-bank retreat. This rate is greater than the 30-year average of about 0.5 m/yr, and is attributed to persistent precipitation, manifest in 10 discharge peaks with a 1-year recurrence interval or greater. Planar and cantilever failures were relatively common along the steepest section of the 4.7 m high banks. Cantilevers were formed by (1) preferential erosion of sands and silts by fluvial undercutting about 3.0 to 3.5 m below the top bank, and (2) by sapping and small pop-out failures in the region of contrasting permeabilities in the Holocene units about 1.6 to 2 m below the top bank. Both processes resulted in oversteepening at the base of the EH unit and subsequent collapse during wet periods.

We employed CONCEPTS to simulate the above failure processes between March 1, 1996 and March 31, 1997. Because CONCEPTS assumes that bank material properties are homogeneous, we used the following average shear strength parameters:  $c' = 4.5$  kPa,  $\phi' = 28.3^\circ$ , and  $\phi^b = 13.5^\circ$ . The phreatic surface was located 3.5 m below the top bank. CONCEPTS follows the 'excess shear-stress' method of Osman and Thorne (1988) to calculate the lateral erosion at the bank toe. Critical shear stresses varied from 8 Pa at cross section 1 to 1.5 Pa at the apex of the bend (cross section 6) to 4 Pa at cross section 10. Figure 3 compares modeled and surveyed cross sections 6 and 8, and shows the time series of factor of safety of the right bank at the respective cross sections.

The right bank of cross section 6 underwent two successive failures on December 1, 1996 and midwinter 1997. The retreat of the top bank was 0.6 m and 1 m respectively. CONCEPTS simulated failure of the bank on February 4, 1997, the top bank retreating 2.7 m (Figure 3a). CONCEPTS underpredicted the rate of basal scour. Also, the observed failure plane angle is greater than that computed. Cantilever failures will generally occur along steeper slip surfaces than planar failures. The cantilever failure mechanism is not implemented in CONCEPTS.

Figure 3b shows that the right bank of cross section 8 fails in late April, 1996 (retreat of top bank is 1.0 m) and again on February 3, 1997 (retreat of top bank is 1.6 m). CONCEPTS simulated failure of the bank on March 6, 1997. The slip surface intersects the bank 3.1 m below the top bank. CONCEPTS was unable to predict the 'February 1997' failure because the flow did not remove all of the slumped material from the 'March 1996' failure. On March 31, 1997, 0.05 m<sup>3</sup>/m of the modeled, slumped material remained. The next flow event after March 31, 1997 should be able to erode the remaining slumped material and continue to steepen the bank, which may then fail in the spring of 1997.

Figure 3c shows how the factor of safety of the right bank of cross section 6 reduces after each runoff event. Scour of bank material near the bank toe steepens the bank and reduces the factor of safety. The spikes in the factor of safety represent the increase of factor of safety due to rising flow stage and thus increase in confining pressure.

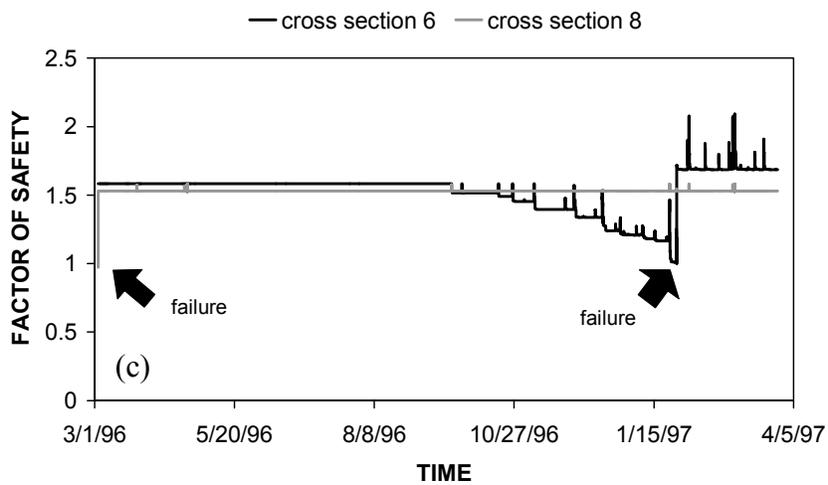
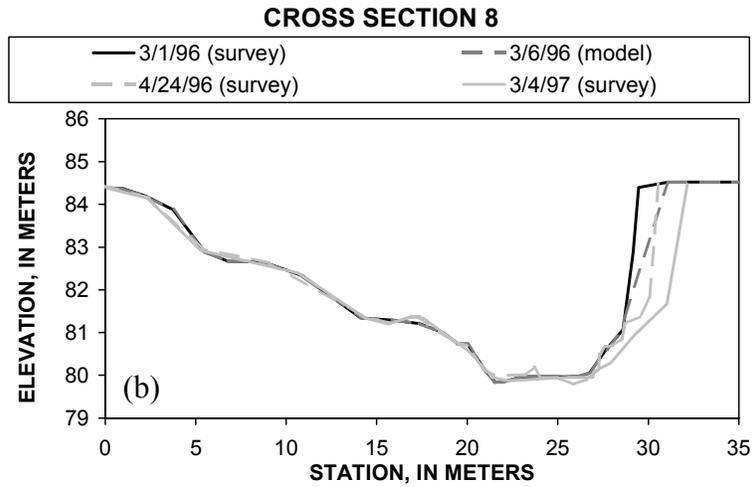
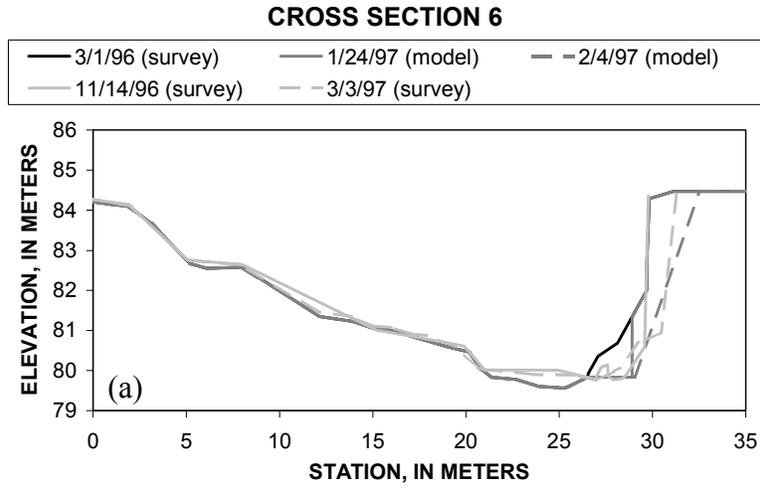


Figure 3 Streambank failure simulations: (a) surveyed and computed cross section 6, (b) surveyed and computed cross section 8, and (c) calculated factor of safety.

## Concluding Remarks

Channel width adjustment is an important response of incising stream systems found in the Midsouthern and Midwestern United States. Channel width adjustment is a combination of scour of bed and bank material near the bank toe by flow, and ensuing mass wasting of bank material because of steepening and increasing height of the streambank. Quantitative channel evolution models are needed to evaluate the success of stream stabilization and rehabilitation measures in protecting infrastructure and farmland adjacent to streams. The channel evolution model CONCEPTS simulates not only scour and fill of the channel bed but also streambank-erosion processes. The bank stability algorithm is able to evaluate the effect of surface water (confining pressures) and pore-water pressures on the factor of safety, and automatically searches for the slip surface that produces the smallest factor of safety. Field studies characterize the impact of pore-water pressures (matric suction, seepage, and confining pressures) on failure dimensions and shearing resistance. Results from these studies have been used to enhance and test CONCEPTS. Application of CONCEPTS to a bendway in Goodwin Creek, Mississippi shows that the bank-stability algorithm accurately predicts the timing and dimensions of the failure. Differences in failure-block dimensions are caused by the fact that CONCEPTS does not model the cantilever failure mechanism.

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