HYDROLOGICAL PROCESSES
Hydrol. Process. **23**, 1537–1547 (2009)
Published online 31 March 2009 in Wiley InterScience
(www.interscience.wiley.com) DOI: 10.1002/hyp.7286

Streambank dewatering for increased stability[†]

F. Douglas Shields Jr,* Andrew Simon and Seth M. Dabney
USDA-Agricultural Research Service, National Sedimentation Laboratory Oxford, Mississippi 38655, USA

Abstract:

Streambank erosion is often the dominant source of sediment leaving modified watersheds. Mass failure of high, steep banks is one of the most serious forms of streambank erosion. The risk of a given bank experiencing mass failure is a function of bank height, angle, and soil strength, which is governed by soil moisture. Two methods for bank dewatering were tested in adjacent sections of streambank bordering a deeply incised channel in northern Mississippi: a low-cost pump system and subsurface horizontal drains. Pore water pressures (both positive and negative pressures, or matric suction) were continuously monitored for 2 years at the pumped site, at an adjacent untreated control section, and for 1 year at the site stabilized with horizontal drains. Resulting data were used to calculate a time series of the factor of safety using a computer model. Over the course of two wet seasons, average bank retreats for the control and pumped plots were 0·43 and 0·21 m, respectively. More limited monitoring revealed that the site with passive drains retreated about 0·23 m. At the pumped site pore water pressure was 3–4 kPa lower than at the control site during the most critical periods. Accordingly, computed factors of safety were above the failure threshold at the pumped site, but fell below unity at the control site on 11 occasions over the period of observation. Similarly, the drained site displayed generally lower pore water pressure and higher safety factors except for two events when drains were evidently overwhelmed with the volume of local surface and subsurface flows. These results suggest, but do not prove, that bank dewatering promoted lower rates of bank retreat and higher levels of stability since the three sites had slight differences in soils, geometry and boundary conditions. Initial cost of the dewatering treatments were significantly less than orthodox bank stabilization measures, but operation and maintenance requirements may be greater. Published in 2009 by John Wiley & Sons, Ltd.

KEY WORDS streambank erosion; bank retreat; soil moisture; slope stabilization; pore water pressure; matric suction; stream restoration; sediment

Received 16 July 2008; Accepted 21 January 2009

INTRODUCTION

A national assessment of the biological condition of wadeable streams rated 42% of streams (by length) as being in 'poor' condition; riparian disturbance and streambed instability were among the top four most commonly observed stressors responsible for poor condition (Paulsen *et al*., 2006). Riverbank failure and retreat is a significant threat to agricultural land and infrastructure, and is a particular problem in many parts of the USA where system-wide channel incision is occurring (Simon and Rinaldi, 2000). Sediment loads from such watersheds are dominated by material eroded from the channel boundary (Grissinger *et al*., 1991), particularly from streambanks (Simon and Rinaldi, 2006) (Table I). This is a critical issue to downstream water quality with sediment being listed as one of the primary pollutants of surface waters of the USA in national assessments (http://iaspub.epa.gov/waters10/attains_nation_cy.control#causes). Sediment yields from typically unstable channel systems of the mid-continent are shown in Table II and are generally one to two orders of magnitude higher than stable streams in the same ecoregion (Simon *et al*., 2004b).

Streambank erosion is often caused by the interaction of hydraulic forces operating at the bank toe which result in undercutting and steepening, and gravitational forces operating on the bank mass. When banks are high and steep, 'mass' or geotechnical failure can occur along a shear plane. Large blocks of material from the upper part of the bank are delivered to the bank toe where they can undergo subsequent weathering and fluvial erosion, or they may be delivered directly to the flow for dispersal and transport downstream as suspended load. Rates of bank failure are governed by bank height, angle and the shear strength of the soil. Shear strength is highly sensitive to pore water pressure within the bank material. When soils are saturated, pore water pressure reduces soil shear strength, as defined by the Mohr–Coulomb criterion:

$$\tau_f = c' + (\sigma - \mu_w)\tan\phi' \text{ for saturated conditions } (\mu_w > 0) \tag{1}$$

where τ_f = shear stress at failure (kPa); c' = effective cohesion (kPa); σ = normal stress (kPa); μ_w = pore water pressure (kPa); and ϕ' = effective angle of internal friction (degrees). In the zone above the water table where soils are not saturated, negative pore water pressure has

* Correspondence to: F. Douglas Shields Jr, USDA-Agricultural Research Service, National Sedimentation Laboratory Oxford, Mississippi 38655, USA. E-mail: doug.shields@ars.usda.gov

Table I. Contributions of streambank erosion to total sediment load in incised channels in the south-eastern United States

Stream	Ecoregion	Bed Material	Contribution from banks (%)	Reference
James Creek, MS	Southeastern Plains	Sand/Clay	78	Simon *et al.*, 2002
Shades Creek, AL	Ridge and Valley	Gravel	71–82	Simon *et al.*, 2004a
Goodwin Creek, MS	Mississippi Valley Loess Plains	Sand/Gravel	64	E. Langendoen, pers. comm., 2006
Yalobusha River, MS	Southeastern Plains	Clay/Sand	90*	Simon and Thomas, 2002
Obion-Forked Deer River, TN	Mississippi Valley Loess Plains	Sand	81*	Simon and Hupp, 1992
Beaver Creek, TN	Ridge and Valley	Sand/Gravel	80	Schwartz, 2006

* Indicate the contribution from banks relative to all channel sources

Table II. Reported sediment yields from unstable streams in the mid-continent of the United States

River	State	Sediment yield (T y^{-1} km^{-2})	Reference
Obion-Forked Deer River	Tennessee	770	Simon, 1989
Hotophia Creek	Mississippi	2300	Little and Murphey, 1981
Willow Creek	Iowa	400	Ruhe and Daniels, 1965
West Tarkio Creek	Iowa-Missouri	410	Piest *et al.*, 1976
Yalobusha River	Mississippi	989	Simon and Thomas, 2002

the effect of increasing shear strength through its effect on apparent cohesion. Fredlund *et al.* (1978) defined a functional relationship describing increasing soil strength ('apparent cohesion') with increasingly negative pore water pressure. The rate of increase is defined by the parameter ϕ^b such that:

$$\tau_f = c' - \mu_w \tan\phi^b \text{for unsaturated conditions} \quad (\mu_w \leq 0) \qquad (2)$$

The parameter ϕ^b, which is generally between 10° and 20°, reaches a maximum value of ϕ' under saturated conditions (Fredlund and Rahardjo, 1993). The apparent cohesion term incorporates both electro-chemical bonding within the soil matrix and cohesion due to surface tension on the air–water interface of the unsaturated soil. The term ϕ^b varies by soil type, and with moisture content (Fredlund and Rahardjo, 1993; Simon *et al.*, 2000). Data on ϕ^b are particularly lacking for alluvial materials, but our experience is that it generally lies between 8° and 10° (Simon *et al.*, 1999).

Bank failures along incised channels often follow periods of precipitation, when the additional strength provided by negative pore water pressure is lost as more of the bank mass becomes saturated, generating positive pore water pressure (Simon *et al.*, 1999). Positive pore water pressures not only reduce soil strength as described above, but also increase the weight of the bank mass and contribute to sapping and 'pop out' types of bank failure when water moving downward through permeable bank soils encounters restrictive layers (Wilson *et al.*, 2007). Thus, maintenance of negative pore water pressure by artificial or other means could provide greater bank stability and reduced frequency of mass failure.

Improving the stability of streambanks is a matter of either decreasing the driving (gravitational) forces operating on the bank mass and/or increasing the resistance of the bank material to gravitational failure. Maintaining lower levels of pore water pressures by dewatering may accomplish both. Bank dewatering occurs naturally through the transpiration of vegetation (Simon and Collison, 2002) and artificially by installing horizontal drains (Rahardjo *et al.*, 2003; Crenshaw and Santi, 2004) or by capillary siphon systems (D. Gray, written commun., 2006). Vegetation is an important contributor to bank stability because it increases the apparent cohesion of the soil through root reinforcement. In the case of high banks, like those along larger rivers or deeply incised channels, failure planes lie below the root zone, and vegetation has little effect on soil strength (Abernethy and Rutherfurd, 1998). Furthermore, vegetation can adversely impact bank stability by enhancing infiltration and thus elevating bank soil moisture (Simon and Collison, 2002). In cases where vegetation is not available or effective stability might be increased by pumping water out of the bank mass from within vertical wells or providing passive dewatering through horizontal drains. Dewatering has been used as a means of protecting bridge abutments (F. Schultz, Nebraska Dept. of Roads, oral commun.) but is generally considered too expensive to be used in less critical situations. However, low-cost solar powered pumps used for livestock watering are available and could potentially extend the application of this technique to more common situations. The objective of this study was to test and evaluate the applicability of horizontal drains and a low-cost pumping system for stabilizing a rapidly eroding bank along an incised, sand-bed stream.

SITE AND METHODS

Dewatering wells and horizontal drains were constructed along a rapidly eroding bank on the outside of a meander bend of Little Topashaw Creek, Chickasaw County, Mississippi (33°44′28·7″N, 89°10′28·0″W). The field adjacent to the creek is composed of an Arkabutla silt loam (fine-silty, mixed, active, acid, thermic Fluventic Endoaquepts) soil, with a surface slope of 0·002 towards the north, and was farmed to cotton (*Gossypium hirsutum* L.) or corn (*Zea mays* L.) during the period of study. There were no trees within 30 m of any of the study plots described below. Watershed history and geomorphology have been described previously (Wallerstein, 2000; Downs and Simon, 2001; Simon and Thomas, 2002; Watson *et al*., 2004). The drainage area upstream of the study site is 37 km^2, and the area receives approximately 1500 mm precipitation per year. Tensiometer measurements (described below) and piezometer data collected by Pezeshki *et al*. (2007) in a concurrent study conducted along this same stream reach showed that the permanent groundwater table during the course of this study was 2·5 to 5 m below the floodplain surface, with perched water occurring at higher elevations during wetter periods. Seepage was noted at several locations on the bank face, generally at the boundaries of soil layers. Limited measurements following storm events documented discharge rates from individual seeps ranging from 0·068 to 0·931 m^3/day (Wilson *et al*., 2007).

The channel incised following downstream channelization, and the resulting instability produced a channel about 35 m wide (top-bank width) with a thalweg about 6 m below the terrace (Figure 1, Shields *et al*., 2004). The channel bed and banks were unstable throughout this study, with upstream migration of a 0·6-m-high headcut of ∼60 m during a single storm event in April 1999. Bank materials are a mixture of coarse and fine sand, with smaller amounts of silt and clay. Shear strength parameters of the bank materials (c_a and ϕ') were measured *in situ* using an Iowa Borehole Shear Tester (Lutenegger and Hallberg, 1981) in a borehole located within the 'pumped plot' shown on Figure 2. Bulk unit weight of the bank materials was obtained from sample cores of known volume that were weighed, dried for 24 h at 105 °C, and weighed again in the laboratory.

To evaluate the effectiveness of dewatering, the site was divided into three plots spaced about 25 m apart to ensure that they were hydrologically isolated from one another, but close enough to be comparable in terms of inputs of groundwater and exposure to basal erosion. (Figure 2A). Normally, one would expect greater basal erosion and thus bank retreat downstream from the bend apex (at the pumped plot); however, during the period of observation, deposition actually occurred along the outside (concave) bank of the meander bend and erosion all along the inside as described by Shields *et al*. (2008). The plot treatments were as follows:

1. a plot dewatered using pumps,
2. a control plot with no dewatering, and
3. a plot passively dewatered using horizontal drains.

At the pumped plot, two pairs of 0·05 m diameter unscreened wells were installed 15 m apart to depths of 2 and 4 m (Figure 2B). Well spacing and depths were selected based on estimates of their influence (cones of depression) on shallow groundwater above bank failure planes. A 0·04-m-diameter, 12-volt-DC submersible pump was installed in each of the four boreholes, powered by a marine battery connected to solar panels. Float switches maintained water levels in the wells between 0·05 and 0·15 m above the pump intake. A check valve in the discharge hose prevented water from returning to the well once the pump had switched off.

At the drained plot, two perforated plastic tile lines, 0·10 m in diameter with woven sock material to exclude sands, were buried 1·5 m deep perpendicular to the bank and extending 10 m into the field (Figure 2A). Depth was dictated by limitations of available excavation equipment. The lines were placed 20 m apart which was consistent with criteria provided by van Schilfgaarde (1974) applied using best estimates of site soil hydraulic conductivity and stratigraphy. One of the lines had an additional 10 m of tile which turned 90° and ran an additional 10 m parallel to the top of bank. The drain tiles discharged into gutters that were routed to calibrated, tipping-bucket gauges. The site was also served by a logging, tipping-bucket rain gauge located 400 m south of the site. Pumped and drained water was discharged over the top bank into the channel after measurement.

At each plot there was a nest of four tensiometers installed 3 m from the bank edge to measure pore water pressure. The tensiometers were installed at depths of 0·30 m, 1·70 m, 3·00 m and 4·70 m (4·90 m for the drained site), and measured positive and negative

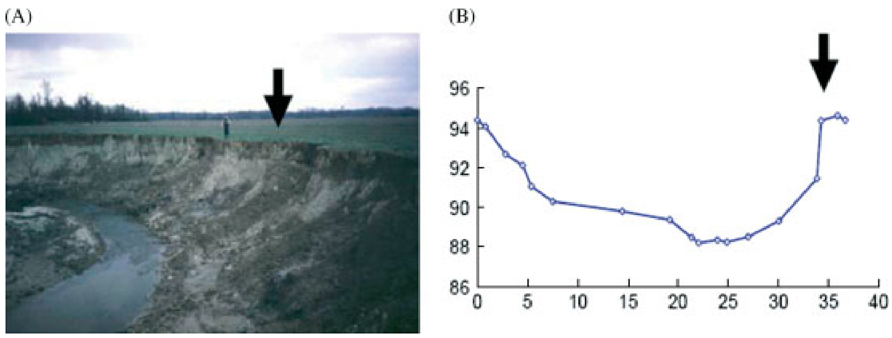

Figure 1. Study site on Little Topashaw Creek, Chickasaw County, MS prior to experiment. (A) Photograph taken facing downstream in early 1999 and (B) Channel cross section facing downstream surveyed in January 2000. Arrow shows approximate location of study site in both figures

Hydrol. Process. **23**, 1537–1547 (2009)
DOI: 10.1002/hyp

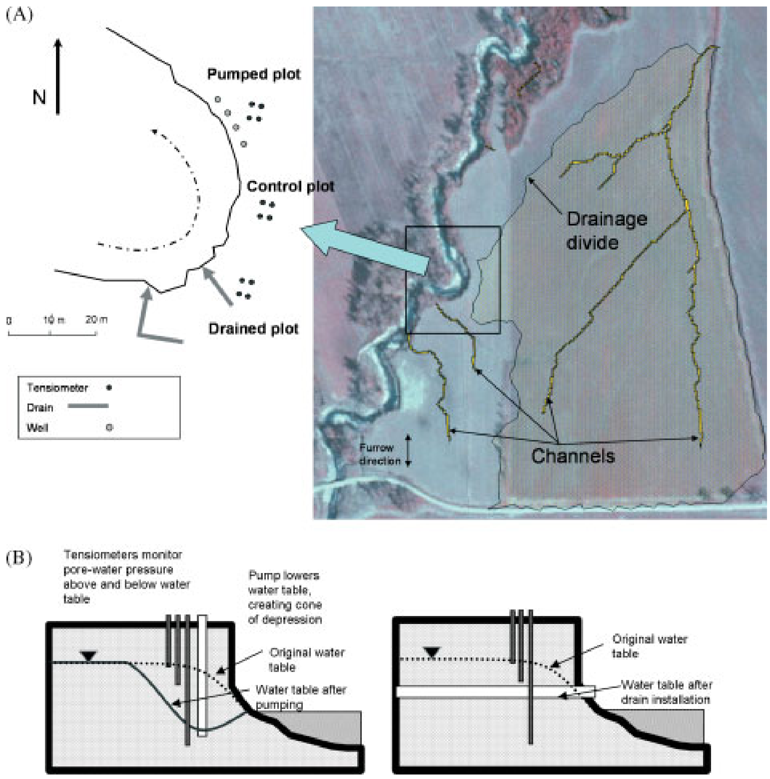

Figure 2. (A) Plan of the field site, Little Topashaw Creek, Chickasaw County, MS. Channels shown in yellow are shallow waterways and water furrows. (B) Schematic section of the pumped (left) and drained (right) plots. Drained plot shows only drain running parallel to top bank; similar effects on shallow groundwater would apply to drains running perpendicular to bank

pore water pressure within 0·1 kPa. Tensiometer depths were selected based on hydrologic and stratigraphic considerations following careful examination of two site borings. Pump discharge hoses and discharge lines from the horizontal drains were routed through tipping bucket rain gauges that measured water volume. Readings from the tensiometers and the pump discharge recorders were logged every 10 min. Once per week or after every large rainfall event, the distance from a baseline to the top bank along eight transects running perpendicular to the bank (four in each plot) was measured to the nearest 0·01 m to record bank retreat for the pumped and control plots (Figure 3). Bank retreat for the drained plot was computed using surveys obtained only at the beginning and end of the experiment. Following installation of pumps and drains, a detailed topographic map was created for the site using conventional and global-positioning system survey techniques.

To quantify the effect of dewatering on bank stability, a Bank Stability Model developed at the USDA-ARS, National Sedimentation Laboratory was employed. The Bank Stability Model is a further development of the wedge failure type developed by Simon and Curini (1998) and Simon *et al*. (1999), which in turn is a refinement of the models developed by Osman and Thorne (1988) and Simon *et al*. (1991). The model is a Limit Equilibrium analysis in which the Mohr–Coulomb failure criterion is used for the saturated part of the wedge, and the Fredlund *et al*. (1978) criterion is used for the unsaturated part. In addition to accounting for positive and negative pore

Figure 3. Determining bank retreat rate by measuring along established survey line on Little Topashaw Creek, Chickasaw County, MS

water pressure, the model incorporates complex bank geometries, unique layers, changes in bulk unit weight based on soil water content, and the external confining pressure provided by streamflow acting on the bank face. The model divides the bank profile into up to five user-definable layers with unique geotechnical properties.

The factor of safety (F_s) is the ratio between the resisting and driving forces, and is given by (Simon *et al*., 1999):

$$F_s = \frac{\sum c_i{}'L_i + [S_i \tan\phi_i{}^b] + [W_i\cos\beta - U_i + P_i\cos(\alpha-\beta)]\tan\phi_i{}'}{\sum W_i\sin\beta - P_i\sin[\alpha-\beta]} \quad (3)$$

where c_i' = effective cohesion of ith layer (kPa); L_i = length of the failure plane incorporated within the ith layer (m); S_i = force produced by negative pore water pressure on the unsaturated part of the failure surface (kN m^{-1}); W = weight of the ith layer (kN m^{-1}); U = the hydrostatic-uplift force on the saturated portion of the failure surface (kN m^{-1}); P = the hydrostatic-confining force due to external water level (kN/m); α = failure-plane angle (degrees from horizontal); and β = bank angle (degrees from horizontal). Units for W, U, and P are in dimensions of force per unit channel length, reflecting the two-dimensional character of the model.

Data from the tensiometer nests were used to run the model for the period May 2000 to April 2002 for control and pumped plots and for October 2001 to April 2002 for the drained plot. The pumps were activated on 24 November 2000, when the water table rose sufficiently to trigger the system. Initial bank profiles from surveys conducted at times similar to model start times and soil properties from *in situ* borehole shear tests were used in the model simulations. Stream stage was extrapolated from a gauge located about 600 m upstream by assuming that water surface slope was equal to channel slope, which was measured from a detailed thalweg survey profile (Shields *et al*., 2004).

Comparison of safety factors

The efficacy of bank dewatering treatments was examined by plotting the mean monthly computed safety factors for the pumped and drained plots against monthly mean values for the control plot. Since safety factor values were available from the control and pumped plots for periods before and during pump activation, a before and after paired site analysis similar to that described by Grabow *et al*. (1998) was used to quantify the effects of pumping. Monthly mean safety factors were computed for control, pumped and drained plots. Using a class variable separating the periods before (0) and after (1) pumping was initiated, monthly mean pumped plot safety factor was modelled as the dependent variable with the monthly mean control plot safety factor as the independent variable using PROC MIXED (SAS, 1996) from SAS 9·1·3. Monthly average air temperature was used as a covariate to capture seasonal variation, and an autoregressive covariance structure was used to account for serial correlation within the data. Similar statistical analyses were not possible for data from the drained plot since preconstruction data were not collected there.

RESULTS

Bank materials were similar for the pumped and control site, but higher sand content occurred in the surface layers at the drained site, producing slightly different shear strength. In general, the bank had a coarsening upward stratigraphy with coarse sand (SW in Universal Soil Classification System) at the top (c' = 1 to 3 kPa, ϕ' = 34) and fine sand (SM) at the bottom (c' = 1 kPa, ϕ' = 27° to 22°). Bulk unit weight was 17 kN/m^3 in surface layers, increasing to 19–20 kN m^{-3} below the upper 1 m of soil. The unsaturated strength parameter ϕ^b was assumed to be 10° based on limited field data collected in the region (Simon *et al*. 1999).

Wells and drains were installed in the Spring of 2000. Pumping was triggered concurrent with the onset of wetter weather in November 2000. Between November 2000 and May 2001 the pumped plot was pumped continuously when water levels in the bank exceeded the levels maintained by the float switches, although not all pumps were active at all times due to equipment problems until March 2001. Pumps were reliable from March 2001 onwards. The period of record thus included two dry- to wet-season cycles (Figure 4A).

Local topography and arrangement of a shallow waterway (‘water furrow’) by the farm operator directed more surface runoff toward the drained site than the other two sites (Figure 2). Drains required maintenance due to settlement and erosion of fill from concentrated runoff in the fill placed over the drains. During the spring of 2001, one of the drains was replaced because the perforated pipe became clogged with sand. The replacement drain functioned properly through the end of the experiment.

Pore water pressure

The effectiveness of artificial dewatering can be initially evaluated by comparing data from the tensiometer nests at various depths between the three plots. Lower pore water pressure translates to greater resistance to mass failure through its effect on apparent cohesion (Equation (2)). Tensiometer monitoring began during May 2000 and showed initially low values of pressure close to saturation that increased with time through the summer months. As expected, pore water pressure was lowest (most negative) near the surface and increased with depth throughout the period of record for all three plots. The shallow (0·30 m) tensiometers showed the greatest variability with time as they quickly responded to inputs of precipitation and subsequent drying by drainage and evapotranspiration. With the onset of wetter weather in November 2000, pore water pressure gradually increased at all depths and reached levels >0 (saturated conditions) at a depth of 1·7 m in the control plot during various times of the winter of 2000–2001 (Figures 4B and C).

Comparison of control and pumped plot prior to de-watering

During the relatively wet period from May through July 2000, data from the 0·30 and 1·70 m tensiometers (Figures 4B and C) showed magnitudes and trends that were very similar, with almost identical pore water pressures. At greater depths (3·00 and 4·70 m), however, the control plot was consistently wetter than the pumped plot during this period, with pore water pressures 4–5 kPa higher. During the very dry period from August through November 2000 the upper layers of the control plot dried

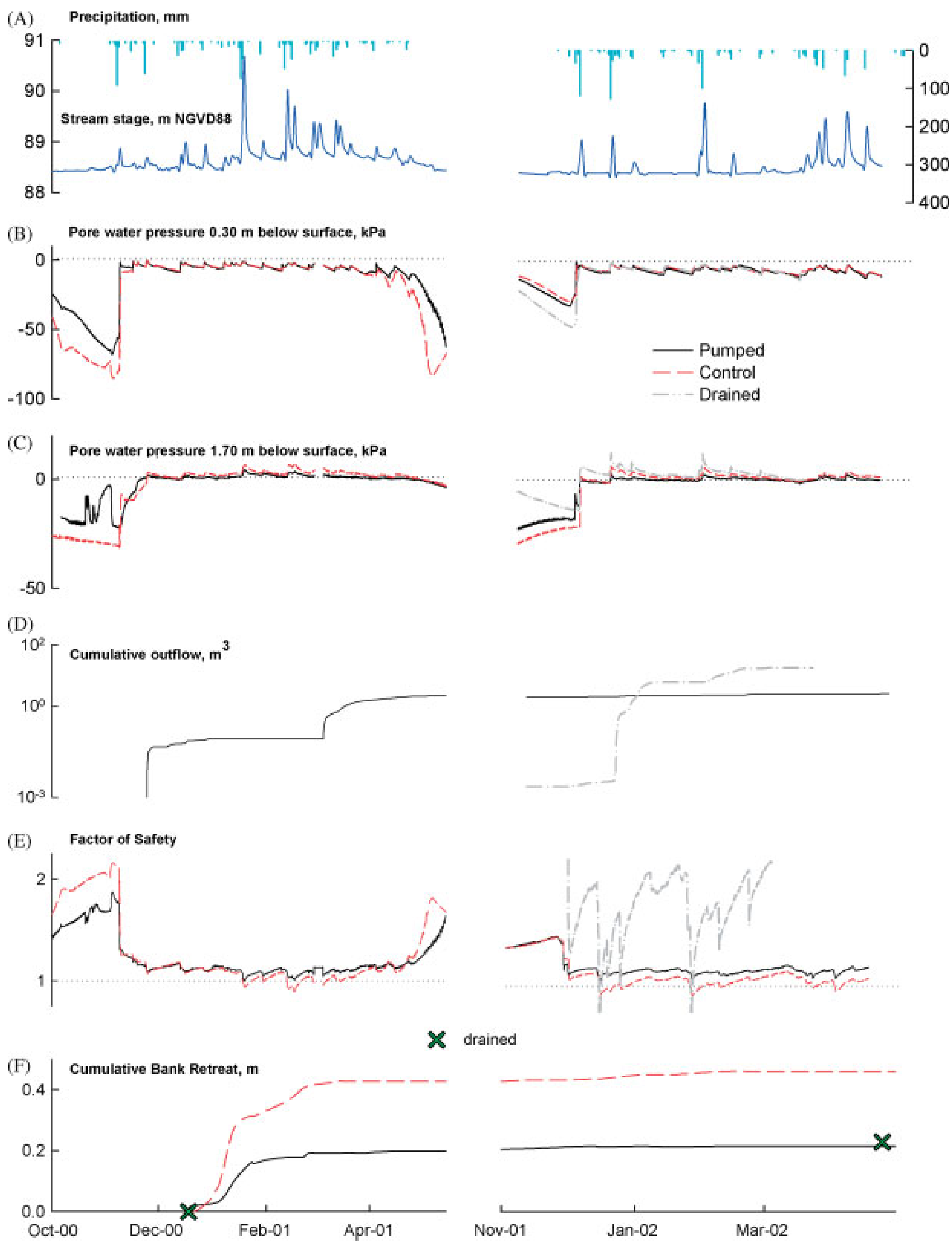

Figure 4. Conditions observed during critical periods at study site on Little Topashaw Creek, Chickasaw County, MS (A) Precipitation and stream stage. (B) Pore water pressure at 0.3 m depth. (C) Pore water pressure at 1.70 m depth. (D) Cumulative outflow from pumps and drains. (E) Computed safety factor. (F) Cumulative mean bank retreat. Green X shows cumulative mean bank retreat for drained plot

out more than the pumped plot, with pore water pressure 10–20 kPa lower. Pore water pressures at 3·00 m depth under both plots converged to within 1 kPa by the start of the winter rains in November 2000. At 4·70 m there was less convergence, and at the end of the dry period the control plot had pore water pressures 2–3 kPa higher than the pumped plot.

Differences between the control and de-watered plots before pumping were probably due to slight differences in soil permeability. The data indicated that the control plot may have been slightly better drained than the pumped plot, with associated drying in the upper layers and wetting in the lower layers. Differences largely cancelled out when averaged across the entire bank profile, and values were, therefore, assumed to be reasonably comparable. The greatest difference, at 4·70 m, had no effect on bank stability in the analysis described below because the potential shear surface was above this depth. Since the control plot was slightly drier than the pumped plot, results from the pumped plot were slightly conservative.

Comparison of plots during dewatering

During the two wet seasons following initiation of pumping, pore water pressure under the pumped plot averaged 0 to 7 kPa lower than for the control plot (Table III). In contrast, mean pore water pressures at depths >0·30 m under the drained plot were 2–7 kPa greater than the control plot and 3 to 14 kPa greater than

Table III. Mean and maximum pore water pressure values (in kPa) observed in the study streambank along Little Topashaw Creek during wet seasons (November–June)

Depth below surface, m	2000–2001						2001–2002					
	Mean (std dev)			Maximum			Mean (std dev)			Maximum		
	Control	Pumped	Drained	Control	Pumped	Drained	Control	Pumped	Drained	Control	Pumped	Drained
0·30	−19 (26)	−17 (22)	No data	−0·5	−0·2	No data	−7 (6)	−8 (7)	−11 (13)	−0·4	0·0	0·0
1·70	−2 (8)	−2 (6)		7·8	4·6		−2 (10)	−2 (7)	1 (6)	7·2	3·5	13·4
3·00	5 (4)	3 (2)		12·8	8·2		7 (3)	1 (1)	9 (5)	12·8	1·6	17·7
4·70	9 (6)	5 (5)		21·4	15·8		13 (5)	6 (4)	20 (4)	21·4	14·7	29·0

for the pumped plot. At the 0·30 m depth, conditions for all plots converged during wet seasons and diverged during drier periods (Figure 4B). These results were reasonable since the pumps were not expected to have much impact on soil moisture this far above the water table. At 1·70 m, pump activation produced a modest but noticeable difference between the control and pumped plots, with lower pore water pressure under the pumped plot, especially during high flow events For the most critical periods, which were associated with high stream stages during wet seasons, pore water pressure was 2–4 kPa lower for the pumped plot but up to 15 kPa greater for the drained plot relative to the control (Figure 4C). A similar trend occurred at 3·00 m, where the pumped plot maintained slightly lower pore water pressures (2–3 kPa) and a slower response to rainfall.

Results for the second wet season (winter and spring 2002) show an average difference between the control and pumped plots of 7 kPa in the upper 2 m of bank material, and 20 kPa in parts of the bank deeper than about 2 m. This corresponds to a lowering of the water table of 0·70 and 2·00 m in the perched and permanent water tables, respectively. The overall effect of dewatering was much more pronounced during this season than during the previous year, due largely to improved equipment performance during the early part of the season. The greatest differences in pore water pressure between the pumped and control plots occurred during the second wet season at a depth of 3·00 m (between the 2 m and 4 m pumps).

Since observed and simulated failure planes were above the 3·00 m depth, pore water pressure values for the 1·70 m level were the most critical determinants of stability. Pump activation caused a modest but noticeable shift in the relative pore water pressures of the control and pumped plots at this elevation, with higher pressure under the control plot, especially during the wet seasons (Figure 4C). Tensiometer readings from the drained site showed slightly drier conditions than the control site at the 0·30 m depth and generally wetter conditions at depths $\geq$1·70 m. This observation is consistent with the depth of the drains (1·5 m).

Dewatering volumes

During the first wet season, all four pumps produced a total of 2·20 m^3, or 0·009 m^3 day^{-1} (Figure 4D). Variations in pumped volumes with depth are probably a function of variations in stratigraphy and permeability as well as the presence of macropores (Wilson *et al.*, 2007). For example, during March and April, 2001, one pump set at 4 m generated 0·616 m^3 while another pump set at 2 m generated only 0·001 m^3. By comparison in another borehole, the 2 m pump generated 1·24 m^3 while the pump at 4 m generated only 0·130 m^3. Volumes from drains were not measured during the first wet season, but they exceeded the volumes from pumps by an order of magnitude during the second wet season (Figure 4D), with a total of 19·4 m^3 or 0·150 m^3 day^{-1} of water measured from the drains compared with 0·640 m^3 or 0·005 m^3 day^{-1} for pumps.

Rates of bank retreat

Bank edge monitoring for the control and pumped plot began in November 2000, and retreat began to occur in December 2000 by a mixture of small mass failures on the control plot and weathering on both plots (Figure 4F). Both plots initially experienced similar retreat rates, associated with wetting up after the first rainfalls of the winter, and freeze–thaw weathering and erosion. However, retreat slowed and stopped sooner on the pumped plot, and thus total retreat was much less. The pumped plot experienced much slower retreat, with only minor sloughing off the bank face. Little if any basal scour occurred during the study period along the concave bank where these plots were located (Shields *et al.*, 2004), effectively limiting rates of bank retreat for all plots. Average cumulative bank retreat for the first wet season was 0·43 m (std dev = 0·08 m) for the control plot and 0·21 m (std dev = 0·16 m) for the pumped plot. During the second wet season neither plot experienced much retreat. The pumped plot experienced minor sloughing in December and January, losing less than 0·01 m of material. The control plot lost approximately 0·05 m of material during the same period. Over the entire period of the experiment, the drained bank lost about 0·23 m. Overall, the results show that dewatered banks retreated about half as fast as the control.

Bank stability modelling

Simulations of bank stability (as defined by the factor of safety) were conducted using the monitored pore water pressure data to provide a continuous record of relative stability for the experimental plots over the study period. Figures 4E and F provide a detailed view of changes in stability and cumulative rates of bank retreat for the pumped and control plots. In general, results show that in the four cases where bank failure was predicted for the control plot, ($F_s < 1{\cdot}0$), stability was predicted for the pumped plot (albeit marginal stability $F_s > 1{\cdot}0$).

Prior to activation of the pump system, computed factors of safety rose for the control and pumped plots during the summer and fall of 2000 as the banks dried out from the previous winter and spring. From May through July 2000 both plots had almost identical F_s values, reflecting similar values of pore water pressure between the two plots. From July 2000, F_s on the control plot rose faster as pore water pressure decreased in the upper soil layers more rapidly than on the pumped plot. With the onset of the rainy season in November 2000 and the associated increase in pore water pressure, F_s declined rapidly for both plots (Figure 4E). After pump activation, the pumped plot maintained higher F_s values until the end of April 2001 when more rapid drying in the upper layer of the control plot again reduced pore water pressure and therefore increased stability. A similar pattern occurred during the second wet season, with the pumped plot again exhibiting slightly higher F_s with the onset of pumping. The drained plot exhibited higher F_s than the other two plots throughout the concurrent period of record except for three events when extremely wet conditions evidently overwhelmed the capacity of the drains to maintain low soil moisture (Figure 4E).

Since pore water pressure, both positive and negative, plays such a crucial role in controlling shear strength and therefore bank stability, the critical point for evaluation of dewatering is during the wettest periods. Table IV provides a comparison of the F_s during these periods for the control and dewatered plots. During the first wet season, six events were observed where F_s for the control site dipped below 1·00, indicating mass failure. Minimum F_s computed for the control plot during these events ranged from 0·90 to 0·99, while simultaneous values computed for the pumped plot were between 1·02 and 1·09. During the second wet season, there were five 'failure events' in the F_s time series computed for the control plot, with minimum F_s between 0·93 and 0·99, while simultaneous minima for the pumped plot were 1·08 and 1·09.

Given the uncertainty inherent in key model parameters, the question arises if the F_s differences (8–15%) in Table IV are significant. Sensitivity analyses of Equation (3) presented by Langendoen and Simon (2008) show that fairly large variations in soil parameters are required to produce shifts in F_s this large. They found that either a 30% change (1·3 kPa) in effective cohesion, c', a 20% change (6°) in effective friction angle, ϕ', or a 20% (3·5 kN m^{-3}) change in soil unit weight, γ were required to produce a 10% change is F_s. Only a 0·5 m change in water table elevation (11% of bank height) had a similar impact on F_s. Additional sensitivity analyses conducted using data sets from this study showed computed safety factors were relatively insensitive to variation of ϕ^b, with median safety factors increasing less than 2% when ϕ^b was set equal to 10° rather than 8°. Furthermore, variation in pore water pressure up to an order of magnitude greater than the uncertainty in our tensiometer data produced very small shifts in computed F_s and in the fraction of time that simulations indicated that a given bank would be unstable—0·0006%. Simulated time series of F_s showed that the pumped plot retains a small safety margin at the times of lowest stability.

Table IV. Minimum computed factors of safety for selected periods for the study streambank, Little Topashaw Creek, Mississippi

Wet season	Date of event	Minimum computed factor of safety		
		Control	Pumped	Drained
First	19-24 Jan 01	0·94	1·02	No data.
	30-31 Jan 01	0·99	1·05	
	13-21 Feb 01	0·90	1·02	
	28 Feb 01	0·99	1·07	
	6-7 Mar 01	0·97	1·06	
	15-16 Mar 01	0·99	1·09	
Second	14-20 Dec 01	0·93	1·08	0·71
	23-25 Dec 01	0·98	1·12	0·99
	24-29 Jan 02	0·93	1·08	0·69
	21-22 Mar 02	0·99	1·09	No data.
	1-3 Apr 02	0·96	1·10	No data.

The record of computed F_s for the drained plot (partial record for second wet season only) shows that it experienced three failure events, and that these events F_s had minima between 0·69 and 0·99. The generally higher safety factors for the drained site reflect a more gradual bank slope as well as drainage; on the other hand, the bank profiles and soil properties for the control and pumped plots were virtually identical.

Comparison of predicted stability with measured bank retreat shows a close, though not perfect, correspondence between times when the model predicted bank instability and the observed occurrences of retreat. The initial bank retreat in December and January 2000–2001 closely corresponded to the first predicted failure event (Table IV). Further retreat occurred during the latter part of January and February 2001. Very small amounts of retreat (∼0·01 m) occurred along the control and pumped banks for the remainder of the first wet season, despite prediction of control site failures in early March 2001. For the remainder of the first wet season, predicted F_s increased and observed bank retreat ceased. The second period during which simulated F_s values fell below the critical value of 1·0 (December 2001 to January 2002) coincided with only a small amount of observed bank retreat.

Comparisons of safety factor

Monthly mean safety factors for the pumped and drained plots were correlated with simultaneous mean values for the control plots (Figure 5). The pumped

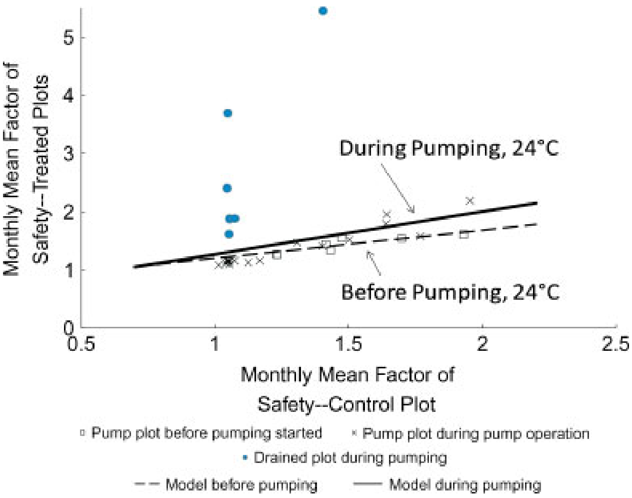

Figure 5. Monthly mean computed safety factor for the pumped plot versus monthly mean computed safety factor for the control plot before and after pumping

and control plots varied in the range of 1 to 2, while variation in the drained-plot safety factor was much greater. Monthly variation in the control plot factor of safety was caused by variation in bank water potential due to the balance of rainfall and evapotranspiration.

A statistical regression model was fitted to explore the variation in the pumped-plot safety factor in terms of variation in the control-plot safety factor and mean monthly air temperature. The significance of model terms was tested using partial sums of squares, which represent the contribution of each term to the model after all other terms in the model are considered. Results indicated the pumped-plot safety factor did not increase as rapidly as the control plot safety factor but that the relative response differed between the pumping and pre-pumping periods ($F = 33$, $P < 0{\cdot}01$), and that the pump-plot safety factor increased with increasing monthly temperature beyond any temperature effects reflected in the control-plot safety factor ($F = 8$, $P < 0{\cdot}02$). The regression equations were:

$$SF_p = 0{\cdot}39 + 0{\cdot}48 SF_c + 0{\cdot}013 T \qquad \text{(pre-pumping)}$$

$$SF_p = 0{\cdot}22 + 0{\cdot}73 SF_c + 0{\cdot}013 T \qquad \text{(during pumping)}$$

where SF_p is the pumped-plot safety factor, SF_c is the control-plot safety factor, and T is the mean monthly air temperature (°C). Thus, if the safety factor of the control plot was equal to 1·0 and if $T = 24\,°C$ (the average temperature for the pre-pumping period), the predicted SF_p would be 1·18 pre-pumping and 1·26 during pumping. In contrast, if $T = 15{\cdot}4\,°C$ (the mean for the pumping period), predicted SF_p would be 1·07 pre-pumping and 1·15 during pumping. In Figure 5, both equations are plotted with $T = 24\,°C$, so some of the deviation of observations from the 'during pumping' regression line is explained by temperature variation.

The much greater variation of the drained plot safety factor relative to that of the control plot (Figure 5) may reflect site variation since the drained plot also had a lower minimum safety factor (Table IV) and a larger volume of water removed (Figure 4D).

DISCUSSION AND CONCLUSIONS

Observed bank retreat was much greater during the first wet season than during the second, even though more rainfall occurred during the second wet season (Pezeshki *et al*., 2007). During the first wet season, which was colder as well as drier, freeze–thaw action may have primed the bank for subsequent mass failure by opening vertical cracks extending from the floodplain surface into the upper bank. Sloughing associated with freeze–thaw and mass failures during the first year produced material that accumulated at the bank toes, reducing the bank angles and perhaps increasing stability. For stability modelling, the initial bank profiles were used for the entire 2-year simulation, leading to an underestimation of stability in the second year.

Evidently freeze–thaw processes play a role in bank stability even in the relatively mild climate of northern Mississippi, and these processes are partially addressed by dewatering. The effectiveness of freeze–thaw cycling depends on an adequate moisture supply to form ice crystals and thus dewatering should retard freeze–thaw to some degree. Also, due to the high specific heat capacity of water, changes in soil moisture affect how rapidly soil temperature changes.

Differences in the volume of water discharged by the drains and pumps as well as the differences among the four pumps suggest that movement of groundwater through the site was heterogeneous, probably as a result of the lenses of coarser and finer bed material that compose the bank materials and the presence of macropores (Wilson *et al*., 2007). As noted above, the local topography and a shallow waterway directed more surface runoff toward the drained site than the other two sites Differences in surface soil were also observed, particularly in regard to dry season pore water pressure at a depth of 0·30 m. We ascribe these differences primarily to differences in soils and local hydrology instead of dewatering treatment because pumps and drains likely had little impact on soil moisture this far above the water table. In addition, at 0·30 m both plots responded to near-surface fluxes of rainfall, drainage and evapotranspiration rather than movements of water deeper in the bank profile. Clearly, precise comparison of the two methods of bank dewatering is not possible due to the site-specific conditions at each plot, but this is typical of field demonstrations as opposed to carefully controlled but less realistic laboratory experiments.

Since shallow soil moisture (0·30 m) was not responsive to pumping, and since pumping occurred during the wetter winter and early spring months when vegetation was dormant, impacts of bank dewatering on vegetation were likely slight, and none were visually observed. Bank vegetation is desirable at most sites for aesthetic, environmental and stability reasons and impacts of dewatering on vegetation should be considered when such a project is planned. Site soils, hydrology and geometry will interact with the local plant community in site-specific ways. However, the main effects of correctly designed bank dewatering will generally be beneath the root zone, as for our site.

The dewatered plots experienced bank retreat rates that were about half as great as the control plot rate. However, definite linkage between dewatering and bank retreat suppression would require multiple study sites and careful accounting for site differences in soils, groundwater movement, bank geometry and chemistry. Lack of replication was a fundamental limitation of this study. Bank retreat rates show great spatial variation when measured at smaller scales, and more sites would be required to address this source of variation. For example, Shields *et al.* (2004) reported results of surveys of 38 cross-sections along a 2 km reach of Little Topashaw Creek that encompassed the sites studied herein. The standard deviation of bank retreat for a given year was 1·5 to 3·0 times the annual mean. Retreat rates for the plots are comparable if site characteristics are carefully considered. Although the plots were located at different positions along the outside of the creek meander bend (Figure 2), removal of failed material from bank toes by fluvial action was not a significant erosion driver during the study period—in fact, there was some deposition in the toe areas adjacent to the plots as revealed in cross-section surveys summarized by Shields *et al.* (2004). Higher safety factors observed for the drained site were partially due to more gradual bank slope there, and the lack of pre-treatment data for that site made it impossible to conduct the kind of statistical tests that were done for the pumped site. Therefore use of passive drains was not shown to have an effect on bank retreat, although there were promising indications that drainage could improve bank stability under some conditions. It is notable that the 'failure events' detected by the time series of computed safety factor (Figure 4E, Table IV) show lower minima for the drained plot than for the other treatments. These minima suggest that during certain hydrologic conditions the drains were simply overwhelmed with surface and subsurface flow, greatly increasing pore water pressure at key depths.

Performance of the drains could have been improved by modifying drain design parameters (depth and spacing) using better information regarding soil permeability and hydrologic loading than we had available. Both dewatering treatments were relatively inexpensive compared with other methods of bank stabilization; total initial costs were \$1200 and \$370 for the pumped and drained plots, respectively (year 2000 dollars). Each plot stabilized about 30 m of bank at a cost of \$40 m^{-1} for the pumped plot and \$12 m^{-1} for the drained plot. This compares with costs of approximately \$300/m for stabilization using re-grading and riprap, though there are recurrent costs associated with maintenance for both dewatering systems. Bank dewatering may be practical to protect critical infrastructure and also provides a potential means of temporarily stabilizing a bank while vegetation is becoming established.

ACKNOWLEDGEMENTS

Andrew Collison made substantial contributions to analyses of the data described here. Wendy Dickerson, Brian Bell, Mark Griffith, John Massey and Calvin Vick assisted with installation of bank treatments and data collection and reduction. Cooperation of the riparian landowner, Mr Carter Edmondson, is also gratefully acknowledged. Don Gray and Natasha Pollen read an earlier version of this paper and made many helpful comments.

REFERENCES

Abernethy B, Rutherfurd ID. 1998. Where along a river's length will vegetation most effectively stabilise stream banks? *Geomorphology* **23**(1): 55–75.

Crenshaw BA, Santi PA. 2004. Water table profiles in the vicinity of horizontal drains. *Environmental & Engineering Geoscience* **10**: 191–201.

Downs PW, Simon A. 2001. Fluvial geomorphological analysis of the recruitment of large woody debris in the Yalobusha River network, Central Mississippi, USA. *Geomorphology* **37**: 65–91.

Fredlund DG, Rahardjo H. 1993. *Soil Mechanics of Unsaturated Soils*. John Wiley & Sons, Inc.: New York.

Fredlund DG, Morgenstern NR, Widger RA. 1978. The shear strength of unsaturated soils. *Canadian Geotechnical Journal* **15**: 313–321.

Grabow GL, Spooner J, Lombardo LA, Line DE. 1998. Detecting water quality changes before and after BMP implementation: use of a spreadsheet for statistical analysis. NWQEP Notes 92, North Carolina State University, Raleigh; 1–9.

Grissinger EH, Bowie AJ, Murphey JB. 1991. Goodwin Creek bank instability and sediment yield. In *Proceedings of the Fifth Federal Interagency Sedimentation Conference (FIASC)*. 1991. Federal Energy Regulatory Commission; PS-32-PS-39.

Langendoen EJ, Simon A. 2008. Modeling the evolution of incised streams II: streambank erosion. *Journal of Hydraulic Engineering* **134**(7): 905–915.

Little WC, Murphey JB. 1981. Evaluation of streambank erosion control demonstration projects in the bluffline streams of northwest Mississippi. Appendix A, Stream Stability Report to Vicksburg District, U.S. Army Corps of Engineers, AD-A101-386. National Technical Information Service, Washington, DC.

Luttenegger JA, Hallberg BR. 1981. *Borehole Shear Test in Geotechnical Investigations*. *American Society of Testing Materials Special Publication* 740, ASTM: West Conshohocken, PA.

Osman AM, Thorne CR. 1988. Riverbank stability analysis I: theory. *Journal of Hydraulic Engineering* **114**: 134–150.

Paulsen S, Stoddard J, Holdsworth S, Mayio A, Tarquinio E. 2006. Wadeable streams assessment: a collaborative survey of the nation's streams. Report No. EPA 841-B-06-002. Office of Water, United States Environmental Protection Agency, Washington DC.

Pezeshki SR, Li S, Shields FD Jr, Martin LT. 2007. Factors governing survival of black willow (Salix nigra) cuttings in a field restoration project. *Ecological Engineering* **29**(1): 56–65. DOI:10·1016/j.ecoleng.2006·07·014.

Piest RF, Beer CE, Spomer RG. 1976. Entrenchment of drainage systems in western Iowa and northwestern Missouri. In *Proceedings of the Third Federal Inter-agency Sedimentation Conference*; 5-48–5-60.

Rahardjo H, Hritzuk KJ, Leong EC, Rezaur RB. 2003. Effectiveness of horizontal drains for slope stability. *Engineering Geology* **69**: 295–308.

Ruhe RV, Daniels RB. 1965. Landscape erosion: geologic and historic. *Journal of Soil and Water Conservation* **20**: 52–57.

SAS. 1996. *SAS/STAT Software: changes and enhancements through release 6·11*. SAS Institute, Inc.: Cary, NC.

Schwartz JS. 2006. Beaver Creek watershed plan, Knox County, Tennessee. Draft Report for Causes and Sources of Sediment Pollution for the Tennessee Department of Environmental Quality, 604(b) Funds, Nashville, TN. University of Tennessee, Department of Civil & Environmental Engineering, Knoxville, TN.

Shields Jr FD, Morin N, Cooper CM. 2004. Large woody debris structures for sand bed channels. *Journal of Hydraulic Engineering* **130**(3): 208–217.

Shields Jr FD, Pezeshki SR, Wilson GV, Wu W, Dabney SM. 2008. Rehabilitation of an incised stream with plant materials: the dominance of geomorphic processes. *Ecology and Society* **13**(2): 54. [online] URL: http://www.ecologyandsociety.org/vol13/iss2/art54/.

Simon A. 1989. The discharge of sediment in channelized alluvial streams. *Water Resources Bulletin* **25**: 1177–1188.

Simon A, Collison AJC. 2002. Quantifying the mechanical and hydrologic effects of riparian vegetation on streambank stability. *Earth Surface Processes and Landforms* **27**(5): 527–546.

Simon A, Curini A. 1998. Pore pressure and bank stability: the influence of matric suction. In *The 1998 International Water Resources Engineering Conference*, Abt SR (ed). Memphis, TN; 358–363.

Simon A, Hupp CR. 1992. Geomorphic and vegetative recovery processes along modified stream channels of West Tennessee. US Geological Survey Open-File Report, vol. 91–502, USGS, Nashville, TN.

Simon A, Rinaldi M. 2000. Channel instability in the loess area of the Midwestern United States. *Journal of American Water Resources Association* **36**: 133–150.

Simon A, Rinaldi M. 2006. Disturbance, stream incision, and channel evolution: the roles of excess transport capacity and boundary materials in controlling channel response. *Geomorphology* **79**: 361–383.

Simon A, Thomas RE. 2002. Processes and forms of an unstable alluvial system with resistant, cohesive streambeds. *Earth Surface Processes and Landforms* **27**: 699–718.

Simon A, Wolfe WJ, Molinas A. 1991. Mass wasting algorithms in an alluvial channel model. In *Proceedings of the 5th Federal Interagency Sedimentation Conference*, August 1991, Las Vegas, NV; 8-22–8-29.

Simon A, Curini A, Darby SE, Langendoen EJ. 1999. Streambank mechanics and the role of bank and near-bank processes in incised channels. In *Incised River Channels: Processes, Forms, Engineering and Management*, Darby SE, Simon A (eds). John Wiley and Sons: Chichester; 123–152.

Simon A, Curini A, Darby SE, Langendoen EJ. 2000. Bank and near-bank processes in an incised channel. *Geomorphology* **35**: 193–217.

Simon A, Bingner RL, Langendoen EJ, Alonso CV. 2002. Actual and reference sediment yields for the James Creek Watershed, Mississippi. National Sedimentation Laboratory Research Report 31, US Dept of Agric. Agric. Res. Serv., Oxford, MS.

Simon A, Langendoen E, Bingner R, Wells R, Yuan Y, Alonso C. 2004a. Suspended-sediment transport and bed-material characteristics of Shades Creek, Alabama and ecoregion 67: Developing water-quality criteria for suspended and bed-material sediment. Report no. 43, USDA Agricultural Research Service National Sedimentation Laboratory, Oxford, MS.

Simon A, Dickerson W, Heins . 2004b. Suspended-sediment transport rates at the 1·5-year recurrence interval for ecoregions of the United States: transport conditions at the bankfull and effective discharge. *Geomorphology* **58**: 243–262.

Van Schilfgaarde J, ed. 1974. *Drainage for agriculture*. American Society of Agronomy, Inc.: Madison, WI.

Wallerstein NP. 2000. Geomorphic evaluation of Little Topashaw Creek: June 2000. Unpublished report, National Sedimentation Laboratory, Oxford MS.

Watson CC, Bledsoe BP, Biedenharn DS. 2004. System level analysis of watershed instability in the Yalobusha Basin, Mississippi. In *Watershed Management and Operations Management 2000*, Flug M, Frevert D, Watkins Jr DW (eds). American Society of Civil Engineers: Reston, VA, CD-ROM.

Wilson GV, Periketi R, Fox GA, Dabney S, Shields Jr FD, Cullum RF. 2007. Soil properties controlling seepage erosion contributions to streambank failure. *Earth Surface Processes and Landforms* **32**: 447–459.