

Parker et al. The effects of variability in bank material properties on riverbank stability: Goodwin Creek, Mississippi

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12

13 Abstract

14

15 Bank erosion is an important area of research within fluvial geomorphology and is a land management
16 problem of global significance. The Yazoo River Basin in Mississippi is one example of a system which is a
17 victim of excessive erosion and bank instability. The properties of bank materials are important in
18 controlling the stability of stream banks and past studies have found that these properties are often
19 variable spatially. Through an investigation of bank material properties on a stretch of Goodwin Creek in
20 the Yazoo Basin, Mississippi, this study focuses on: i) how and why effective bank material properties vary
21 through different scales; ii) how this variation impacts on the outputs from a bank stability model; and iii)
22 how best to appropriately represent this variability within a bank stability model.

23 The study demonstrates the importance that the variability of effective bank material properties has on
24 bank stability: at both the micro-scale within a site, and at the meso-scale between sites in a reach. This
25 variability was shown to have important implications for the usage of the Bank-Stability and Toe-Erosion
26 Model (BSTEM), a deterministic bank stability model that currently uses a single value to describe each
27 bank material property. As a result, a probabilistic representation of effective bank material strength
28 parameters is recommended as a potential solution for any bank stability model that wishes to account for
29 the important influence of the inherent variability of soil properties.

30

31 **Keywords:** bank erosion; bank stability model; model uncertainty; variability; probabilistic assessment of
32 stability.

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35 **1. Introduction**

36

37 Bank erosion is an important erosion process in alluvial streams and is a land management
38 problem of global significance (ASCE, 1998a; 1998b). For example Simon et al. (1996) describe how
39 in the loess area of the Midwest United States bank material can contribute as much as 80% of the
40 total sediment eroded from incised channels. The Yazoo River Basin in Mississippi is one such
41 example of a system which is a victim of excessive erosion and bank instability (DeCoursey, 1981).

42 The process of bank erosion is often associated with a channel response to incision through width
43 adjustment. Conceptual models of bank retreat attempt to explain this response, describing how bank
44 failure occurs when erosion of the bank toe and the channel bed adjacent to the bank have increased
45 the height and the angle of the bank to the point that the gravitational forces exceed the shear strength
46 of the bank material, resulting in mass failure (Osman and Thorne, 1988). Taking this conceptual
47 model, the stability of river banks can therefore be considered to be controlled by a balance between
48 the gravitational forces acting on the steepened bank, and the resisting forces controlled by the
49 geotechnical strength of the in situ bank material. Given this threshold condition that determines bank
50 stability, it is important to specifically quantify the driving and resisting forces in order to accurately
51 define bank-stability thresholds.

52 The key component of the resisting force within this balance is the geotechnical strength of the
53 bank material. Numerous studies have previously demonstrated the importance of soil strength in
54 slope and bank stability. Lohnes and Handy (1968) described the importance of physical properties of
55 the materials in their analysis of slope stability in loess and Thorne et al. (1981) used in situ tests of
56 geotechnical properties to determine bank-stability conditions of incised streams in northern
57 Mississippi. In more recent studies, Simon and Darby (1997), Simon et al. (2000), Rinaldi and Casgali
58 (1999) and Darby et al. (2000), using many of the techniques described in Thorne *et al.* (1981), have
59 all demonstrated how bank failures are triggered by changes in the geotechnical characteristics of the
60 bank materials. This study aims to expand on this previous research and explore the variability in
61 resisting forces that help to determine bank stability.

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63 **1.1 Bank stability analysis theory:**

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65 For the simple case of a planar failure of unit width and length, the driving (gravitational) force is given
66 by:

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$$S_d = W \sin \beta \quad (1)$$

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70 where S_d is the driving force; W is the weight of the failure block and β is the angle of the failure plane
 71 (degrees). For saturated soils, bank resistance is represented by the revised Coulomb equation (Simon et
 72 al., 2000):

73

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$$S_r = c' + (\sigma - \mu) \tan \phi' \quad (2)$$

75

76 where S_r is the shear strength of the bank material; c' is the effective cohesion (kPa); σ is the normal
 77 stress given by $\sigma = W \cos \beta$; μ is the pore-water pressure (kPa) and ϕ' is the effective friction angle
 78 (degrees). For un-saturated or partially saturated banks, due to the effect of negative pore-water pressures
 79 described by Simon et al. (2000), the equation derived by Fredlund et al., (1978) applies:

80

81

$$S_r = c' + (\sigma - \mu) \tan \phi' + (\mu_a - \mu_w) \tan \phi^b \quad (3)$$

82

83 where $(\mu_a - \mu_w)$ is the difference between the air pressure μ_a and the water pressure μ_w in the pores
 84 and represents the matric suction in the soil, which when summed with the inherent *effective* cohesion
 85 within the soil forms the total or *apparent* cohesion. ϕ^b describes the rate of increase in shear strength due
 86 to an increase in matric suction.

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The ratio between the resisting (S_r) and driving (S_d) forces is expressed as a Factor of Safety (F_s),
 where a value greater than 1.0 indicates stability and where a value of 1.0 or less indicates imminent
 failure.

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Building on previous work, Simon et al. (2000) addressed in detail the specific forces and processes
 controlling bank failures in incised channels and developed a bank stability algorithm for layered cohesive
 stream banks. This algorithm is for layered banks and is based on combining the Coulomb equation for
 saturated banks with the Fredlund et al. (1978) equation for unsaturated banks. The algorithm thus
 encompasses the influence of negative pore-water pressures on increasing bank strength, the influence of
 positive pore-water pressures in reducing bank strength and the supporting hydrostatic forces provided by
 in-channel flow, as well as accounting for the way that soil properties vary both vertically between layers
 and over time as moisture content changes. This algorithm became the initial version of the Bank Stability
 and Toe Erosion Model (BSTEM) developed at the USDA-ARS, National Sedimentation Laboratory

100 (Simon and Curini, 1998), an Excel based model that calculates the F_s for layered cohesive streambanks.
 101 The reader is referred to Simon et al. (2000) for further details on the derivation of the algorithm; the
 102 equation for the factor of safety is:

103

$$104 \quad F_s = \frac{\sum c'_i L_i + (S_i \tan \phi_i^b) + [W_i \cos \beta - U_i + P_i \cos(\alpha - \beta)] \tan \phi'_i}{\sum W_i \sin \beta - P_i \sin(\alpha - \beta)} \quad (3)$$

105

106 where F_s is the Factor of Safety; c'_i is the *effective* cohesion of the material of the i th layer (kPa); L_i is the
 107 length of the failure plane incorporated within the i th layer (m); S_i is the force produced by matric suction
 108 on the unsaturated part of the failure surface (kN/m); ϕ_i^b is the rate of increase in shear strength due to
 109 matric suction in the material of the i th layer; W_i is the weight that the i th layer contributes to the failure
 110 block; β is the angle of the failure plane ($^\circ$); U_i is the hydrostatic uplift force on the saturated portion of
 111 the failure surface (kN/m); P_i is the hydrostatic confining force due to external water level (kN/m); α is the
 112 original bank angle ($^\circ$); and ϕ'_i is the friction angle of the material comprising the i th layer.

113

114 **1.2 Uncertainty in bank stability analyses:**

115

116 Young (1999) highlights that since the inherent uncertainty associated with modelling most
 117 environmental systems is often acknowledged, it is surprising that many models are completely
 118 deterministic in nature. In a similar application to riverbank stability, research surrounding slope stability
 119 has revealed that the heterogeneity of soils provides a major source of uncertainty in estimations of
 120 operational shear strengths within all slope design applications (El-Ramly et al., 2005), and is a well
 121 recognised issue within geotechnical research (Vanmarcke, 1977; Huang, 1983; El-Ramly et al., 2002;
 122 Duncan et al., 2003).

123 The case of bank erosion is no exception with the large number of influencing factors involved, and the
 124 variability within each of these factors, forming a significant level of uncertainty in the prediction of bank
 125 erosion rates (Bull, 1997). In particular the primary soil mechanics variables that control the resisting
 126 strength of river banks, including cohesion, friction angle and soil unit weight, have been found to be
 127 significantly inconsistent in several studies (Lohnes and Handy, 1968; Thorne et al., 1981; Simon, 1989;
 128 Simon and Darby, 1997). The uncertainty caused by this variability is currently recognised in the BSTEM in
 129 the form of a safety margin between Factor of Safety values of 1 and 1.3 within which banks should only
 130 be considered to be 'conditionally stable'.

131 Thorne et al. (1981) originally represented this variability in geotechnical strength by calculating bank
132 factor of safety for both the average and worst case ambient conditions during measurement. However,
133 due to limited awareness of the important role of pore-water pressures on bank shear strength at the time
134 there was no separation of *effective* cohesion and matric suction within their analysis. Therefore Thorne et
135 al. (1981) were actually representing the variability in the measured *apparent* geotechnical parameters,
136 largely controlled by pore-water pressure conditions at the time of measurement.

137 Darby and Thorne (1996a) also recognised the importance of variable bank material properties and
138 attempted to provide a river bank probability of failure based on the range of soil properties present in the
139 bank rather than the more traditional factor of safety based on a single value soil property. However,
140 despite the useful nature of this probabilistic approach, as with the analysis performed by Thorne et al.
141 (1981), this work was limited by its inability to distinguish measured *apparent* geotechnical parameters,
142 caused by ambient moisture conditions, from actual *effective* geotechnical parameters.

143 Following a large body of research into the impact that matric suction has on the *apparent* shear
144 strength of soils (Casagli et al., 1997, 1999; Simon and Curini, 1998; Simon et al., 2000), it is now possible
145 to explore the true variability of *effective* soil strength parameters rather than that variation driven by soil
146 moisture conditions. This study hopes to take advantage of this, and through an investigation of bank
147 material properties on a stream in the Yazoo Basin, Mississippi, we focus on two issues; firstly, on how
148 and why *effective* bank material properties vary spatially and secondly, on what impact this variability has
149 on the output of a bank stability model.

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151

152 **2. Study Area, Instrumentation and Data Collection**

153

154 The study area for this research is an intensively studied bendway section in the Goodwin Creek
155 Experimental Watershed (Simon and Collison, 2002), north-central Mississippi [Figure 1]. Bank materials
156 along Goodwin Creek consist of 1 to 2 meters of moderately cohesive brown clayey-silt of late Holocene
157 age (LH) overlying approximately 1.50m of early Holocene grey, blocky silt of low cohesion lower
158 permeability. These two units are separated by a thin layer (~10cm) containing manganese nodules and
159 characterised by very low permeability. These materials overlie 1.00m of sand and 1.50m of packed sand
160 gravel.

161

162 ****Figure 1****

163

164 All of the data required to complete this study's aims was collected from the Goodwin Creek
165 experimental bendway, during July and August 2005 and is based the around seven cross-sections
166 spaced approximately 30m apart. Continuous measurements of pore-water pressures at five depths (0.30,

167 1.48, 2.00, 2.70 and 4.30m), surface-water stage and rainfall have been recorded at the site from
168 November 1996, along with regular cross-section surveys following every major flow event (Simon and
169 Darby, 1997; Simon and Curini, 1998; Simon et al., 2000).

170 A series of in situ shear strength measurements were taken using an Iowa Bore-hole Shear Tester
171 (BST; Luttenegger and Hallberg, 1981). Samples for particle size, soil moisture and bulk unit weight
172 were also taken on the outer banks of cross-sections A through G, with one cross-section (B) chosen
173 to receive more intense measurement. In total 10 sets of measurements were taken at Cross-Section
174 B: 5 in the upper, Late Holocene layer (LH - approximately 1.00m depth) and 5 in the lower, Early
175 Holocene layer (EH - approximately 2.00m depth). For the remaining cross-sections the number of in
176 situ shear strength measurements and associated particle size, soil moisture and unit weight
177 measurements was reduced to just 2 at both the 1.00m and 2.00m depths.

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179 **2.1 Evaluation of effective geotechnical properties:**

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181 Friction angle and apparent cohesion values were obtained from the direct-shear measurements.
182 However, since apparent cohesion is the sum of the effective cohesion (due to the soil skeleton) and
183 cohesion caused by matric suction (ψ ; negative pore-water pressures), as described above, it was
184 necessary to account for the impact that moisture content has on generating cohesion. This was done
185 by converting the apparent cohesion (c_a) values given by the direct shear measurements to effective
186 cohesion (c') values using (Fredlund et al., 1978):

187

$$188 \quad c_a = c' + (\psi) \tan \phi^b \quad (4)$$

189

190 where ϕ^b is the rate of increase in shear strength due to matric suction.

191

192 The values for the parameter ϕ^b used within this study (9.98 in the LH layer and 19.8 in the EH
193 layer) were derived in a similar manner to the value that Simon et al. (2000) derived for the LH unit at
194 the same site. A series of BST tests were conducted in both the LH and EH units at the same depths
195 as tensiometers around which the measurements were taken. The BST tests were performed over a
196 wide range of soil-moisture conditions reached through artificially wetting the soil from a dry state. By
197 plotting the measured apparent cohesion values against matric suction for each soil unit it was
198 possible to evaluate the ϕ^b value for both the LH and EH layers. The ϕ^b value of 9.98 found for the LH
199 layer correlates well with the value of 10.4 the Simon et al. (2000) found within the same layer while
200 the value of 19.8 in the EH layer demonstrates that the value of 17.5 assumed by Simon et al. (2000)

201 for that layer was not unreasonable.

203 **2.2 Generation of bank stability model test conditions:**

204
205 To evaluate the effects of variability in measured effective geotechnical parameters on predicted
206 bank stability, a synthetic test-bank condition was created based on a single surveyed bank profile and
207 set of pore-water pressure measurements at various depths for an instance when the bank profile was
208 at a near critical state ($F_s \approx 1$). The bank and instance chosen was Cross Section D on the 31st
209 November 2004 at 0:00am. It is important to note at this point that the reason for choosing to isolate
210 just one instant is that, as explained above, this study is not concerned with the impact of soil pore-
211 water pressures, channel hydrostatic supporting forces and the effects of moisture content changes on
212 geotechnical properties. These issues have been explored in earlier work by Simon et al. (2000),
213 which the user is referred to. Instead, this study attempts to isolate the impact that the variation in
214 *effective* cohesion, friction angle and unit weight properties have on predicted bank stability. In short,
215 herein we attempt to quantify the uncertainty in bank stability estimation caused by variability of the
216 *effective* geotechnical parameters rather than the variability of those *apparent* geotechnical
217 parameters that are influenced by hydrological changes through time.

218 In order to assess how the variation in material properties influences the accuracy with which the
219 BSTEM predicts bank failures, the necessary model input parameters for a series of past 'near critical
220 state' instances were obtained from a combination of regularly updated cross-section surveys, historic
221 pore-water pressure values from permanently installed tensiometers, and the *effective* bank material
222 property values that were gathered for the intensively measured cross-section (B).

223 For further details on the data-collection procedure readers are referred to Parker (2005) a copy of
224 which can be accessed through contact with the lead author.

227 **3. Investigating the spatial variation of the geotechnical properties of a stream bank.**

228
229 Table 1 and Figure 2 give details of the distributions of each of the geotechnical parameters in both the
230 layers tested for the entire Goodwin Creek bendway. A visual examination of this data shows a significant
231 level of variation within all of the parameters measured. This level of variability is strongly supported
232 throughout the bank stability literature. For example, data published by Thorne et al. (1981) demonstrate
233 similar variation throughout their measurements also taken within Yazoo Basin, Mississippi. This is due to
234 variation in soil composition and properties from one location to another, even within homogenous layers.

235 El-Ramy et al. (2002) attribute this variability to factors such as variations in mineralogical composition,
236 conditions during deposition, stress history, and physical and mechanical decomposition processes.

237 Also of note is the different frequency distribution shapes between the various geotechnical
238 parameters. The friction angle values measured in both layers approximate a normal distribution, the
239 effective cohesion values have a strong positive skew with a high concentration of values around zero,
240 and the saturated unit weight distributions are erratic in both layers. These characteristic distribution
241 shapes for effective cohesion and friction angle have been observed previously by other studies,
242 notably the analysis of soil properties of slopes in Hong Kong by El-Ramly et al. (2005).

243

244 ****Table 1****

245

246 ****Figure 2****

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248 ****Figure 3****

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250 ****Table 2****

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252 ****Figure 4****

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254 ***3.1 Micro- versus meso-scale variability:***

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256 Whilst it has been demonstrated that there is a significant amount of variation within the bank material
257 properties of the Goodwin Creek bendway, it is not known at what scale this variation is present. Soil
258 properties may vary at the micro-scale (within a single block of soil found at one cross-section bank) or at
259 the meso-scale (between different blocks of soil found at separate locations). In order to compare the
260 distribution of the geotechnical parameter values measured at a single cross-section (Cross-Section B) to
261 the distribution of the average geotechnical parameter values from each of the cross-sections, exploratory
262 data analysis in the form of boxplots has been performed [Figure 3].

263 Although by no means entirely consistent, a visual examination of these plots highlights a trend of wider
264 distributions in the data obtained from all of the measured cross-sections than in the data measured just at
265 cross-section B. However, it must be taken into account that the sample size for the data obtained from all
266 of the measured cross-sections is slightly larger the data measured just at Cross-Section B, and this can
267 impact on this particular method of viewing the spread of data (Coakes and Steed, 2001).

268 As well as comparing the variation of bank material properties within Cross-Section B to the variation of
269 average bank material properties between cross-sections it is also possible to statistically examine the
270 proportion of the total variation that is explained by the results being grouped into cross-sections using a

271 one way analysis of variance (ANOVA) test [Table 2]. The results of this analysis is show that although
272 neither layer's friction angle measurements are statistically significantly related to cross-section, there is a
273 statistically significant relationship between cross-section and both the effective cohesion and saturated
274 unit weight values for both layers. Also for all of the variables the eta-squared values are well above
275 Cohen's (1988) guideline for independent variables having a large effect, suggesting that cross-section
276 significantly influences bank soil properties.

277 The means plots of each parameter across the cross-section in Figure 4 support the results of the
278 ANOVA test, demonstrating significant differences in some of the parameter values when compared
279 across the cross-sections. Ideally any patterns in these geotechnical parameters across the cross-sections
280 would be compared against geological survey data to find a cause for this variation. However,
281 unfortunately the best available surveys for the Goodwin Creek catchment are crude in terms of both
282 spatial resolution and accuracy making this impossible.

283 It would initially seem that the results of the above analysis are inconclusive, that the neither the
284 variation within or between cross-sections was significantly greater than the other. However, this is not
285 the case since these results are important in demonstrating that there is *both* a significant level of
286 variation between sites, and also within them.

287
288 The micro-scale variability observed in this study is frequently observed in the literature. For example,
289 Thorne et al. (1981) also found a large amount of within site variability in the Old Paleosol layer at the
290 'Tommy Florence' site on Johnson Creek, Mississippi, where apparent cohesion values ranged between
291 15.2 and 118.3 kPa. However, it is important to recognise that the variability observed in past studies
292 like these may be partly due to variability in matric suction rather than the material properties
293 themselves. Nevertheless, Mitchell and Soga (2005) support the idea of inherent variability of soil
294 properties, describing how these variations in composition and texture can occur within distances as small
295 as a few centimeters, whilst Bull (1997) goes further, describing how each of the primary soil properties
296 described by Grissinger (1982): particle size, clay content, bulk density and ionic strength; vary over small
297 spatial scales, impacting on interparticle strength. Further explanation of this small scale variability can be
298 gained from Wood's (2001) description of the loess materials making up the Goodwin Creek bendway
299 banks where she explains that the soils properties may be complex as a result of structurally controlled
300 weathering and erosion processes such as desiccation cracking, tensile stresses and biological and
301 chemical processes.

302 When considering the meso-scale, between cross-section variation, demonstrated by both the
303 results of this study and also the variation present in other studies, such as the between site variation
304 in Thorne et al.'s 1981 study and the between bend differences found by Simon and Darby (1997) it
305 appears that there is a spatial control over bank material properties. DeCoursey (1981: 50) refers to
306 this kind of variation in the banks from place to place as being a result of the:

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...deposition pattern of ancient sediments and the re-working of bank and bed materials as the channel migrates back and forth through the valley.

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Grissinger et al. (1982) concur, describing that the nature of the valley fill deposits in North Central Mississippi significantly influence the properties of streambank material.

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4. Investigating how the observed variations in bank material properties impact the results of a bank stability model.

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Before exploring the impact that the above observed variations in geotechnical parameter values have on the factor of safety values predicted by the BSTEM it is first necessary to observe the correlations between the input bank material properties [Figure 5]. This is crucial since, for example, maximum friction angle values are unlikely to occur in conjunction with maximum cohesion values and therefore it is necessary to restrict the values of input parameters to within reasonable boundaries of the general correlation found between the properties. Prediction intervals describing where any soil property measurement will fall 95% of the time have been imposed on the correlation plots, defining the boundaries that input parameter values can be drawn from. This ensures that the correlations between the geotechnical parameters are maintained during bank stability simulations.

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****Figure 5****

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****Figure 6****

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A sensitivity analysis of the BSTEM to each of the geotechnical input parameters within the ranges observed at the Goodwin Creek bendway and allowed by the correlation relationships demonstrates that effective cohesion has the strongest control over bank factor of safety when all other factors are kept constant [Figure 6]. Within this sensitivity analysis each of the 3 geotechnical parameters of interest was varied from 0% (the minimum observed value that fitted within the parameter correlations) to 100% (the maximum observed value that fitted within parameter correlations). These increases were carried out in parallel in both modelled layers, maintaining the importance of stratigraphy in bank stability.

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Increases in both effective cohesion and friction angle were found to increase the stability of the modelled bank while increases in saturated unit weight did the opposite, decreasing the factor of safety. These patterns are supported strongly by the literature which describes how the stability of

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342 streambanks increases with an increase in both soil shear strength parameters, since they increase
343 the resisting forces to failure (Osman and Thorne, 1988a). The importance of cohesion in particular is
344 re-iterated by Istanbuluoglu et al. (2005) who found that as soil cohesion of gully banks in Colorado
345 increased, erosion slowed down. Past studies have also reported how an increase in the unit weight of
346 the bank material increases the driving forces causing bank failure (Rinaldi and Casagli, 1999),
347 although this increase in unit weight does also increase the frictional resistance resisting failure
348 (Simon et al., 2000).

349

350 Based on the relationships observed in the above sensitivity analysis it is possible to explore the
351 maximum and minimum factor of safety values that could be predicted by the BSTEM based on
352 combinations of the effective geotechnical parameter measurements from just one bank profile cross-
353 section. By exploring the potential range of factor of safety values, this study attempts to demonstrate how
354 the results of deterministic models such as the BSTEM can be affected by the variability inherent to natural
355 systems.

356 This type of analysis is alike to that carried out by Thorne et al. (1981) in which they found the average
357 and 'worst case' bank conditions based on a range of measurements. Similarly, Simon and Hupp (1987)
358 looked at 'ambient' and 'worst case' conditions in a consideration of critical bank heights on the North Fork
359 Obion River, Tennessee. A key difference between these studies and the analysis in this study is that in
360 their instance 'worst case' conditions are those where the bank is under saturated conditions, as might
361 occur after prolonged rainfall. Since our study differentiates between the inherent, effective soil properties
362 and those properties controlled by soil moisture conditions it is possible to examine the extreme conditions
363 of stability generated by the range in effective bank material properties alone. To avoid confusion with the
364 afore mentioned earlier 'ambient – worst case' work, within this study we shall refer to our extreme cases
365 as those under most- and least- resistive effective geotechnical conditions.

366 Table 3 contains the input data used and the resultant most resistive, average (mean) and least
367 resistive factor of safety values returned by the model. For this part of the analysis each of the runs was
368 based upon the same bank profile and hydrological conditions, and used the range of values for each of
369 the geotechnical values collected at cross-section B. Note that for all cases the range of values is restricted
370 in order to preserve the natural correlation between the variables described above. As would be expected
371 based on the results of the sensitivity analysis, these tables show that the model produces a significant
372 range of possible factor of safety values in response to the range of bank material properties found at
373 Cross-Section B. In reality though, it is important to recognise that whilst the extreme F_s values are
374 theoretically possible, their chance of occurrence is extremely low, requiring specific unlikely combinations
375 of geotechnical parameter values. The majority of combinations of geotechnical parameter values are in

376 general likely to result in more conservative F_s values, making the most- and least- resistive cases unlikely
377 but nevertheless still possible.

378

379 **Table 3**

380

381 **Figure 7**

382

383 One point to note in Table 3 is that the scope of F_s values caused by the variability of geotechnical
384 parameter values crosses either side of the critical value of 1. In theory this implies that depending on
385 which measurement is chosen to represent each parameter the bank may be predicted as both
386 (conditionally) stable and unstable. In order to explore this effect further the F_s values generated as a
387 result of the most and least resistant possible and mean effective geotechnical conditions were found for a
388 further 9 events. The resultant ranges in predicted F_s values, along with whether a bank failure was
389 observed, are displayed in Figure 7. As with the event in Table 3, for all 9 cases the scope of F_s values
390 predicted crosses the point of unity between driving and resistive forces (F_s of 1). This means that
391 depending on which geotechnical parameter values were taken, different conclusions could have been
392 drawn on the stability of the river bank in question.

393 In terms of validating the success of the BSTEM in predicting failure events no definite conclusions can
394 be drawn as it could be said that in each case the model both predicted the stability both correctly and
395 incorrectly dependent on the input parameters chosen. Interestingly, this even applies to the events where
396 the BSTEM predicted F_s values above the safety margin of 1.3, below which banks are considered
397 'conditionally stable'. As mentioned above, this is the means by which the BSTEM currently accounts for
398 uncertainty in stability predictions caused by the variability of bank material properties. This is common
399 with many conventional deterministic slope analyses which, rather than accounting for quantified
400 uncertainty in an explicit manner, rely instead on conservative parameters to deal with uncertain
401 conditions. El-Ramly et al. (2002) describe how past experience has shown that designs based on these
402 conservative parameters are not always safe against failure. Figure 7 shows this to be true in the case of
403 the BSTEM also, with a failure being observed during the event on the 11th April 2005 when the F_s value
404 given under mean effective geotechnical parameter values is well above that conservative 'conditionally
405 stable' level. Yet when considering the full range of geotechnical parameter values it is clear that failure
406 could have been predicted by the BSTEM. It is clear that whilst the BSTEM may be effectual in
407 determining bank F_s given the correct input parameters, its current approach for dealing with the
408 uncertainty caused by variability in bank material properties is limited.

409

410

411 **5. Probabilistic assessment of riverbank stability**

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413 El-Ramly et al. (2002) identify that in order to deal with uncertainty appropriately in slope analyses it is
414 necessary to implement probability concepts. Probabilistic slope stability analysis was first developed in
415 the 1970s and has now become well established in slope engineering literature (Huang, 1983), although
416 El-Ramly et al. (2002) criticise its slow adoption into engineering practice. As described above, the
417 riverbank stability model developed by Darby and Thorne (1996a) includes the option of providing a
418 probability of failure rather than the deterministic factor of safety approach. The model works by
419 substituting measured bank material probability distributions instead of the single valued soil property
420 values used in factor of safety equations. Then by dividing each continuous bank material property
421 distribution into discrete classes, it is possible to define a finite number of combinations of soil property
422 values. Each of these discrete combinations is directly applied in the bank stability equations to
423 determine the factor of safety for that combination. Then the probability of failure is obtained by
424 calculating the proportion of all possible combinations of cohesion, friction angle and unit weight
425 values that result in a factor of safety of less than 1 (Darby and Thorne, 1996b).

426 However, despite the attractiveness of the probabilistic approach taken by Darby and Thorne, the
427 Darby and Thorne model algorithm itself is not recommended above that of the BSTEM since it is
428 limited in its ability to account for the effects of pore-water pressure, which is a fundamental factor in
429 determining conditions of instability (Rinaldi et al., 2004). Instead it is recommended that a means of
430 representing bank stability in a probabilistic manner is developed for the BSTEM so that the variability
431 of bank material properties demonstrated within this study can be appropriately accounted for.

432 Following the example set by those involved with slope engineering (Huang, 1983; El-Ramly et al.,
433 2002; 2005) and by Darby and Thorne (1996a; 1996b; 1996c) it is suggested that each of the
434 geotechnical parameters is assigned a probability distribution function based upon shear strength
435 tests in comparable soils, as in Figure 2, and that the correlation relationships between the variables
436 are defined in a manner similar to those in Figure 5. Then a Monte Carlo simulation could draw at
437 random a value for each input variable from within its defined probability distribution, maintaining the
438 correlation relationships between variables. Each set of randomly sampled input geotechnical
439 parameters would be used to solve the BSTEM algorithm and calculate the corresponding factor of
440 safety for that particular selection of values. After a sufficient number of iterations, the statistical
441 distribution of the factor of safety would be generated. Whilst it is beyond the scope of this study to
442 incorporate this within the BSTEM an example of a potential output is displayed in Figure 8.

443 Unlike the approach suggested by Darby and Thorne which simply results in the probability of
444 failure occurring, this Monte Carlo based technique gives not only the probability of bank failure and

445 the probability of bank F_s falling below or exceeding any other given value but also the most likely
446 F_s value. In the hypothetical example in Figure 8 the probability of bank failure is the cumulative
447 probability of all F_s values below 1, which is approximately equal to 10% or a probability of 0.1. The
448 most likely F_s is given by the modal value, which is approximately equal to 1.125. Of particular value is
449 that the output of this methodology, as exemplified by Figure 8, whilst providing the potential maximum
450 range of factor of safety values possible for a given bank case, also identifies that those extreme
451 cases are generally likely to be extremely improbable. For instance in Figure 8, whilst a F_s of below
452 0.9 is recognised as possible, it is also shown that its probability of occurrence is 0.01. This demonstrates
453 how this particular method for dealing with uncertainty both provides the user with all possible outcomes,
454 as well as realistically recognising the most likely outcome. This depth of information regarding bank
455 stability has the potential to be extremely useful to channel design practitioners requiring stable
456 riverbanks, giving them the ability to choose an appropriate probability of failure when set against the
457 risk tolerance of a specific design specification.

458 A further benefit of the probabilistic approach to bank stability modelling was recognised by Darby
459 and Thorne (1996b). They identified that deterministic bank stability models, when used in conjunction
460 with downstream channel evolution analyses, over predict the longitudinal extent of mass failures
461 since an unstable bank is assumed to fail along the entire reach of the model when in reality mass
462 failures over bank lengths of more than a few meters is rare. Darby and Thorne hypothesised that
463 more realistic predictions of reach-scale bank stability can be obtained using a probabilistic riverbank
464 stability analysis such as that described above. This would be achieved through the assumption that
465 the fraction of the reach that is unstable with respect to mass failure is equal to the probability of
466 failure. Whilst this form of analysis is still essentially a two-dimensional solution to the three-
467 dimensional problem of longitudinal channel adjustment it does present a more realistic means of
468 representation than deterministic based two-dimensional approaches.

469
470 Whilst it is apparent that a probabilistic approach is useful in practical applications, El-Ramly et al.
471 (2002) identify several factors that limit its employment by practitioners. The most relevant of these is the
472 level of data acquisition required to generate the requisite probability distributions representing the material
473 properties (Darby et al., 2000). In reality it is unlikely that a practitioner will undertake an extensive series
474 of shear strength measurements for each study and therefore will not have the statistical distribution data
475 available to perform the probability based analysis. A potential solution is the use of databases of
476 generalised geotechnical parameter distributions based on measurements performed in similar materials.
477 A small number of measurements within the materials for the study in question would enable a set of
478 appropriate general distributions to be selected, upon which the probabilistic analysis could be based. In a

479 similar approach, El-Ramly et al. (2005) use regional probability distributions of cohesion and friction angle
480 to apply a probabilistic slope stability analysis of the Shek Kip Mei cut in Hong Kong. However, it is
481 recommended that for best practice site-specific measurements of the parameters are taken for each
482 study to ensure that the statistical distributions chosen are a good approximation of the values observed in
483 the field.

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485

486 **6. Conclusions**

487

488 It is important to consider the results of any study in the context they were obtained (Bauer, 1996) and
489 the specific results from this study are delimited spatially to the seven surveyed cross-sections on the
490 Goodwin Creek Bendway, Mississippi and temporally to the 8 weeks over which they were surveyed
491 during the summer of 2005. Yet the results of this study have importance reaching far beyond these
492 constricted boundaries and having implications for all issues involving bank stability, and any study within
493 which variability and uncertainty is hidden behind deterministic model outcomes.

494 This study did not find any significant difference between the importance of within site (micro-scale) and
495 between site (meso-scale) variation in bank material properties, but instead showed that they are both
496 present, and both significant in influencing bank stability. The micro-scale variation is thought to be a result
497 of the inherent variability of soil properties, with the meso-scale variation considered to be a relic of historic
498 deposition patterns, although a lack of contextual information restricts any firm conclusions on this.

499 When the range of observed effective geotechnical parameter values was applied to bank stability
500 analyses using the BSTEM it was found that the variability present produced a significant scope of
501 uncertainty in bank factor of safety prediction. The current implicit means by which the BSTEM addresses
502 this uncertainty is thought to be unsuitable, leading the authors to consider a probabilistic based method
503 for dealing with the uncertainty caused by bank material property variability.

504 The core message from the above results is that bank material properties do vary spatially and
505 therefore this variation should be considered during all bank stability analyses, with probabilistic based
506 methods currently offering the most appropriate means of doing this. Further work following on from this
507 study will aim to incorporate a probabilistic representation of bank strength parameters within the BSTEM
508 and test its suitability. In addition, future studies aimed at the determination of the statistical distributions of
509 geotechnical parameters in a range of material types would greatly assist in the widespread acceptance of
510 probabilistic approaches.

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646 **Tables**

647

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Table 1: Summary of all data collected from cross-sections A through G on the Goodwin Creek Experimental Bendway

Depth / Layer of Measurement	~ 1.00m (LH)	~ 2.00m (EH)
Number of tests carried out	17	17
Mean effective cohesion values (c' in kPa)	4.37	0.410
Range of effective cohesion values (c' in kPa)	0 - 13.2	0 - 3.10
Standard Deviation of effective cohesion values (c' in kPa)	4.10	0.935
Mean effective friction angle values (ϕ' in degrees)	31.7	35.1
Range of effective friction angle values (ϕ' in degrees)	22.4 – 40.6	30.5 – 41.1
Standard Deviation of effective friction angle values (ϕ' in degrees)	5.69	3.02
Mean saturated unit weight of sediment values (γ in kN/m ³)	18.6	19.3
Range of saturated unit weight of sediment values (γ in kN/m ³)	18.0 – 19.3	17.8 – 21.1
Standard Deviation of saturated unit weight of sediment values (γ in kN/m ³)	0.428	0.908

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Table 2: One way ANOVA test testing the impact of grouping geotechnical parameter values by cross-section. Where the necessary ANOVA assumptions have not been met a non-parametric alternative (the Kruskal-Willis test) is used instead. The 'eta-squared' value describes the amount of the total variance in the dependent variable that is predictable from knowledge of the levels of the independent variable. Cohen (1988) recommends the following guidelines to interpret the strength of eta squared values: 0.01 = small effect; 0.06 = moderate effect; 0.14 = large effect.

Parameter	Friction Angle (LH)	Friction Angle (EH)	Effective Cohesion (LH)	Effective Cohesion (EH)	Unit Weight (LH)	Unit Weight (EH)
One-way between groups ANOVA test	- (Assumptions not met)	No significant differences (Sig value = 0.243)	- (Assumptions not met)	- (Assumptions not met)	- (Assumptions not met)	Significant difference at the 95% significance level (Sig value = 0.002)
Kruskal-Wallis test	No significant difference (Sig value = 0.336)	- (Parametric alternative preferred)	Significant difference at the 90% significance level only (Sig value = 0.099)	Significant difference at the 90% significance level only (Sig value = 0.063)	No significant difference (Sig value = 0.128)	- (Parametric alternative preferred)
Eta-squared	0.376	0.490	0.556	0.842	0.635	0.833

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Table 3: Input data used and resultant factor of safety values returned by the BSTEM model when predicting the range of possible bank stability conditions using the single bank profile and hydrologic condition from Cross-Section D at 0.00am on 31st November 2004 and the range of bank material properties measured at a single cross-section.

Parameter	Most resistant effective geotechnical conditions	Mean effective geotechnical conditions	Least effective geotechnical conditions
Bank Profile	Cross Section D (31 st November 2004) surveyed profile		
Pore Water Pressures	31 st November 2004 @ 0:00am tensiometer data		
Surface Water Elevation	80.5m		
Friction Angle in Late Holocene Layer	34.0	33.3	22.8
Friction Angle in Early Holocene Layer	41.1	39.3	30.5
Effective Cohesion in Late Holocene Layer	10.9	7.80	3.13
Effective Cohesion in Early Holocene Layer	0	0	0
Saturated Unit Weight in Late Holocene Layer	18.0	18.2	18.5
Saturated Unit Weight in Early Holocene Layer	17.8	18.5	19.3
Factor of Safety	1.02 (Conditionally Stable)	0.940 (Unstable)	0.590 (Unstable)

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656 **Figure Captions**

657

658 *Figure 1: Goodwin Creek experimental watershed, Mississippi.*

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660 *Figure 2: Frequency distribution plots for the measured geotechnical input parameters in each layer.*

661

662 *Figure 3: Boxplot diagrams comparing the distribution of each soil parameter within Cross-Section B to the average values from*
663 *each of the Cross-Sections from A to G for each layer.*

664

665 *Figure 4: Means plots across all seven cross-sections for each soil property parameter in each layer.*

666

667 *Figure 5: Regression plots with 95% prediction intervals displaying the correlations among geotechnical variables and the*
668 *boundaries within which 95% of measurements should fall.*

669

670 *Figure 6: Sensitivity analysis of BSTEM predicted factor of safety to the ranges of each of the measured geotechnical parameters*
671 *when the remaining parameters are set to mean values and one cross-section profile and hydrological condition is used (based on*
672 *Cross-Section D, 31st November 2004 at 0.00am).*

673

674 *Figure 7: Range of F_s values predicted by the BSTEM for 9 separate hydrological events on Goodwin Creek based on the most*
675 *resistant, least resistant and mean effective geotechnical parameter values measured within the layers of a single bank profile.*
676 *Events plotted in red indicate that a bank failure was observed, those plotted in green indicate no observed failure.*

677

678 *Figure 8: A hypothetical example of an output from a probabilistic analysis performed within the BSTEM. The graphs show the % of*
679 *the total frequency for each factor of safety bin class (left) and the cumulative frequency across the range of factor of safety values*
680 *(right).*

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