A MODEL OF RIVER BANK STABILITY ANALYSIS

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ABSTRACT

To promote the effective management of rivers subject to bank erosion, models are needed that can provide reliable predictions of the effects of changes in river morphology or bank material characteristics, so that the undesirable impacts of channel changes can be avoided. Although there are some models of stability analysis of river banks, most of them have a number of technical and conceptual shortcoming. The important deficiency in most existing models is that they consider only one type of river bank failure; i.e. planar failure. In this research a comprehensive model of river bank stability analysis has been developed which considers three most common types of river bank failures. By using several examples, the application of the new model has also been described here.

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ABSTRACT
To promote the effective management of rivers subject to bank erosion, models are needed that can provide reliable predictions of the effects of changes in river morphology or bank material characteristics, so that the undesirable impacts of channel changes can be avoided. Although there are some models of stability analysis of river banks, most of them have a number of technical and conceptual shortcoming. The important deficiency in most existing models is that they consider only one type of river bank failure; i.e. planar failure. In this research a comprehensive model of river bank stability analysis has been developed which considers three most common types of river bank failures. By using several examples, the application of the new model has also been described here.

Keywords: Cantilever failure, Comprehensive model, Planar failure, Rotational failure, Stability of river bank.

1 INTRODUCTION
River bank erosion and associated sedimentation and riparian land destruction are river engineering and water resources management problems of global significant. To promote the effective management of rivers subject to bank erosion, models are needed that can provide reliable predictions of the effects of changes in river morphology or bank material characteristics, so that the undesirable impacts of channel changes can be avoided. The stability of river banks has extensively been investigated by researches during the last three decades and some different models of bank stability analysis have been developed (e.g. Thorne and Tovey, 1981; Grissinger, 1982; Simons and Li, 1982; Osman and Thorne, 1988; Darby and Thorne, 1996; Rinaldi and Casagli, 1999; Simon et al., 1999; and Amiri-Tokaldany, 2002 among others). The major limitation of the existing models is that while some the available methods consider rotational type of bank failure (e.g. Taylor, 1948; Bishop, 1955; Osman, 1985; Darby and Thorne, 1996), most of the existing models of river bank stability consider only the planar failure mechanism of bank erosion (e.g. Osman and Thorne, 1988; Darby and Thorne, 1996; Simon et al., 1999; Rinaldi and Casagli, (1999); and Amiri-Tokaldany, 2002 among others) and they do not consider other types of bank erosion; e.g. progressive erosion of the bed and bank materials, and different types of cantilever failure mechanism.

To address the above major limitation, a new model of stability of river banks, called hereinafter Extensive Model of Stability Analysis of Riverbanks (EMSAR), has been developed in this research which has ability to analysis the bank stability against most frequent failures, i.e. planar, rotational, and cantilever. In next section, the criteria for development of each element of EMSAR have been described.

2 MODEL DEVELOPMENT
The mechanics of bank failures which result from the operation of the processes of erosion; i.e. fluvial entrainment, weakening and weathering, and basal endpoint control, are closely related to the size, geometry and structure or stratigraphy of the bank and to the

engineering properties of the bank material. Considering this point, the criteria for three failure mechanisms addressed above are demonstrated in this section.

2.1 PLANAR FAILURE

Planar failure can be expected on high banks with thin cohesive layers and on low banks in general (Thorne, 1999). The stability analysis of these banks can be carried out by computing the ratio of resisting and motivating forces apply upon most critical failure surface. This surface can be determined by performing the stability analysis for different surfaces considering the variation of forces affecting the stability. In Figure 1, the framework for analysis of the stability of a natural river bank against planar failure, together with the forces acting on the incipient failure block, are illustrated.

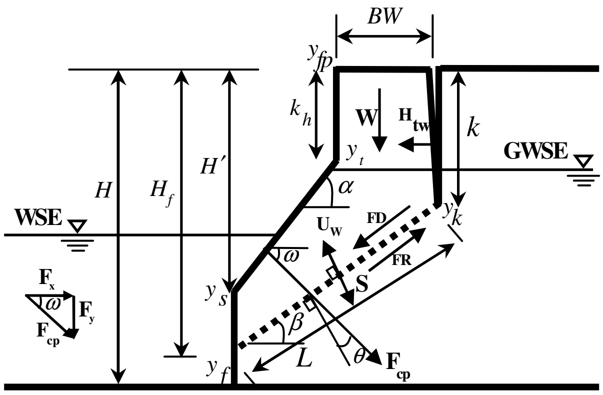

Fig 1: The bank geometry and forces exerted on the incipient failure block

Bank stability may be modelled using the factor of safety concept (Amiri-Tokaldany, 2002):

$$FS_p = \frac{FR_p}{FD_p} \tag{1}$$

where FS_p is the factor of safety against block sliding, and FR_p and FD_p are the resultant resisting and motivating forces acting on a unit width of the failure block, respectively. Hence, bank failure is predicted to occur once the ratio of resisting and motivating forces falls below unity. The resultant motivating and resisting forces acting on a unit width of the failure block is given by (Amiri-Tokaldany, 2002):

$$FD_p = W\sin\beta - F_{cp}\sin\theta + H_{tw}\cos\beta \tag{2}$$

$$FR_p = C'L + S\tan\phi^b + (W\cos\beta + F_{cp}\cos\theta - U_w - H_{tw}\sin\beta)\times\tan\phi' \tag{3}$$

where β= failure plane angle, θ= angle between the direction of the resultant of the hydrostatic confining pressure and a normal to the failure plane, W = weight of a unit width of the failure block, F_{cp} = hydrostatic confining pressure acting on a unit width of the failure block, H_{tw} = hydrostatic force exerted by any water present in the tension crack on a unit width of the failure block, C' = cohesion force acting along the surface of failure plane, L = length of the failure plane, S = negative pore water pressure, ϕ^b = angle expressing the strength increase rate relating to the negative pore water pressure, U_w = uplift force due to any positive pore water pressure acting on a unit width of the failure block, and $\phi^{'}$ = friction angle of bank material.

In the new model we modified the Amiri-Tokaldany's 2002 method of bank stability analysis so that it has now ability to estimate the depth of tension crack and consequently the geometry of the failure block. Based on these estimations, the stability analysis has been performed in the new model which can be accounted as a further step regarding models dealing with planar type of river bank failure.

2.2 ROTATIONAL FAILURE

In high banks composed of cohesive materials with small slope angles, deep failures from bank toe are usually curved so it can be assumed that the failure surface is a circular arc or a logarithmic spiral. The rotational failure in cohesive soils, may be a base, toe or slope failure depending on where the failure arc intersects the ground surface (Thorne, 1982). Analysis for slope failure, introduced first by Swedish engineers, so the method of analysis is called as "Swedish sliced method" and was further developed by Taylor (1948) and by Bishop (1955). In the method of slices the soil body within the failure arc is divided into vertical slices (Figure 2).

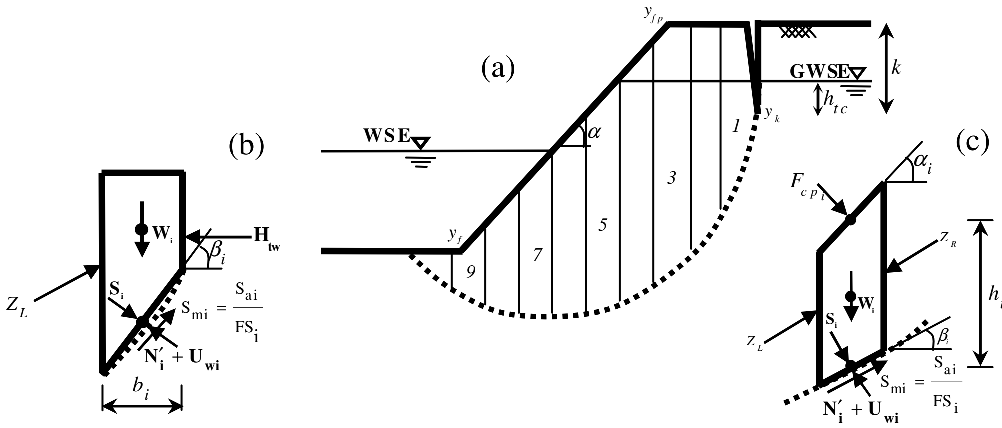

Fig 2: a) The bank geometry with regard to the rotational failure; b) The forces exerted on the incipient failure of first block; c) The forces exerted on the incipient failure of other blocks.

Using the Bishop's simplified assumption in which it is assumed that interslice forces act horizontally, by considering tension crack and the effect of hydrostatic pressure force produced by the water inside tension crack, and also taking into account the negative pore water pressure, a new method of analysis for this type of failure has been introduced in this research and used inside EMSAR which can be accounted as a further step in river bank stability analysis. In figure 2 the principles of the new analysis has been demonstrated in which FS_i = safety factor of i *th* slice, S_{ai} = available shear strength of i *th* slice ($S_a = C^{'} + \varphi \tan \phi^b + N^{'} \tan \phi^{'}$) where $N^{'}$ = effective vertical force, subtitle i denotes the number of the slice; e.g. U_{wi} = positive pore water pressure normal to the failure surface of i *th* slice and F_{cpi} = the hydrostatic force of river flow acting upon i *th* slice, S_{mi} = mobilized shear stress of i *th* slice, WSE and GWSE = level of water surface in the river and in the ground, respectively, Z_L and Z_R = left and right interslice forces, respectively, h_w = height of water inside tension crack, α_i = river bank angle corresponded to i *th* slice before failure, b_i = width of i *th* slice, and H_{tw} = hydrostatic force of water inside tension crack. In the new analysis, by neglecting the interslice shear forces, the equilibrium of the forces over each slice in vertical direction and the equilibrium of the momentum with respect to the center of considered failure arc have been computed. With respect to Figure 2, if safety factor be the same for all of the slices, it can be shown that:

$$FS_r = \frac{FR_r}{FD_r} \tag{4}$$

in which:

$$FR_r = \sum_{i=1}^{n}\left[C'L_i + S_i \tan\phi_i^b + N_i' \tan\phi'\right] \tag{5}$$

$$FD_r = \sum_{i=1}^{n}\left[\left(W_i + F_{cpi}\cos\alpha_i\right)\sin\beta_i + H_{tw}\left(\cos\beta_1 - \frac{h_{tc}}{3R}\right) - F_{cpi}\sin\alpha_i\left(\cos\beta_i - \frac{h_i}{R}\right)\right] \tag{6}$$

By computing the total forces in vertical direction of each slice, the effective vertical force acting on each slice can be calculated using:

$$N_i = \frac{1}{M_{\beta_i}}\left[W_i - S_i \sin\phi_i^b + C'L_i\left(\frac{\sin\beta_i}{FS_r}\right) - U_{wi}\cos\beta_i + F_{cpi}\cos\alpha_i\right] \tag{7}$$

in which:

$$M_{\beta_i} = \cos\beta_i\left[1 + \frac{\tan\beta_i \tan\phi'}{FS_r}\right] \tag{8}$$

Regarding the above equations, since the term FS_r appears in both sides of equation (4), the method of Newton-Raphson is deployed to solve this equation. So, with respect to equation (4), the value of FS_r is determined so that:

$$f\left(FS_r\right) = FS_r \times FD_r - FR_r = 0.0 \tag{9}$$

In the method of Newton-Raphson, the consequent prediction of the FS_r values is given by:

$$FS_{r(i+1)} = FS_{r(i)} - \frac{f\left(FS_r\right)}{f'\left(FS_r\right)} \tag{10}$$

where:

$$f'\left(FS_r\right) = FD_r - \frac{\partial\left(FR_r\right)}{\partial FS_r} \tag{11}$$

$$\frac{\partial\left(FR_r\right)}{\partial FS_r} = \sum_{i=1}^{n} tan\,\phi' \left[\frac{sin\,\beta_i \left(S_i\, tan\,\phi_i^b + C'L_i + N_i'\, tan\,\phi'\right)}{M_{\beta_i} \times FS_r^{\ 2}} \right] \tag{12}$$

The initial value of safety factor is determined by using either the ordinary or Fellenius method (e.g. Abramson et al., 2001). Convergence is obtained if the error tolerance between two successive solutions is less than 0.01.

2.3 CANTILEVER FAILURE

In the river banks consist of a composite structure of non-cohesive sand and gravel overlain by cohesive clay, the rate of fluvial entertainment of material from the lower, non-cohesive layer is much higher than material from the upper bank, which in turn, results in undermining and consequently produces cantilevers of cohesive material (Thorne and Tovey, 1981). Three types of cantilever failure mechanisms; i.e. shear, beam and tensile failure (Figure 3), have been identified by Thorne and Tovey (1981) from which the beam failure is the most common type among the others. Also, in Figure 3 the relative forces and other parameters considered in the bank stability analysis have been demonstrated. In EMSAR we used the Thorne and Tovey (1981)'s equations proposed to analyze the stability of a cantilever.

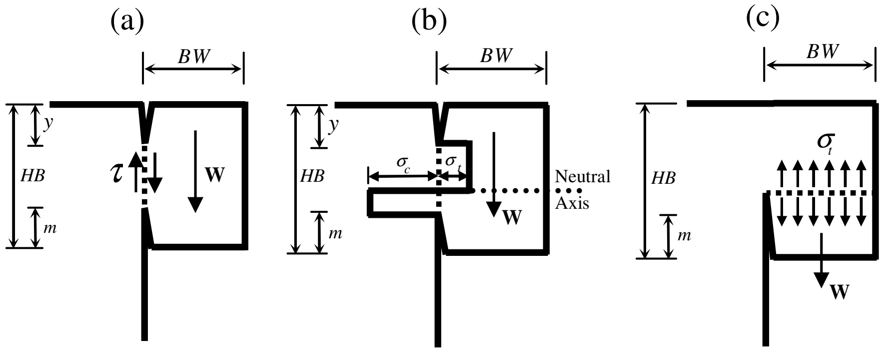

Fig 3: Forces of weight, shear, compression and tension acting on a cantilever with regard to the three modes of failure: a) Shear failure; b) Beam failure; c) Tensile failure. (From Thorne and Tovey, 1981)

2.4 ADVANTAGES AND DISADVANTAGES OF THE MODEL

The most advantage of EMSAR is its ability to analyze the stability of the river banks against most common types of failure mechanisms, so this is a development in this area of research. In Figure 4, the various steps for analysis of the stability of different types of river banks, including single layer or multi-layered composed of different types of soil materials, against mentioned mechanisms of bank failures have been addressed. As shown in Figure 4, on the basis of the first group of the input data describing the type of river bank materials, the most probable mechanism of bank failure for the bank has been recognized by the model. Having specified the type of bank failure, the relevant data, as the second group of data, should have been introduced to the model to start the processes of bank stability analysis and the results have been recorded in relevant output file.

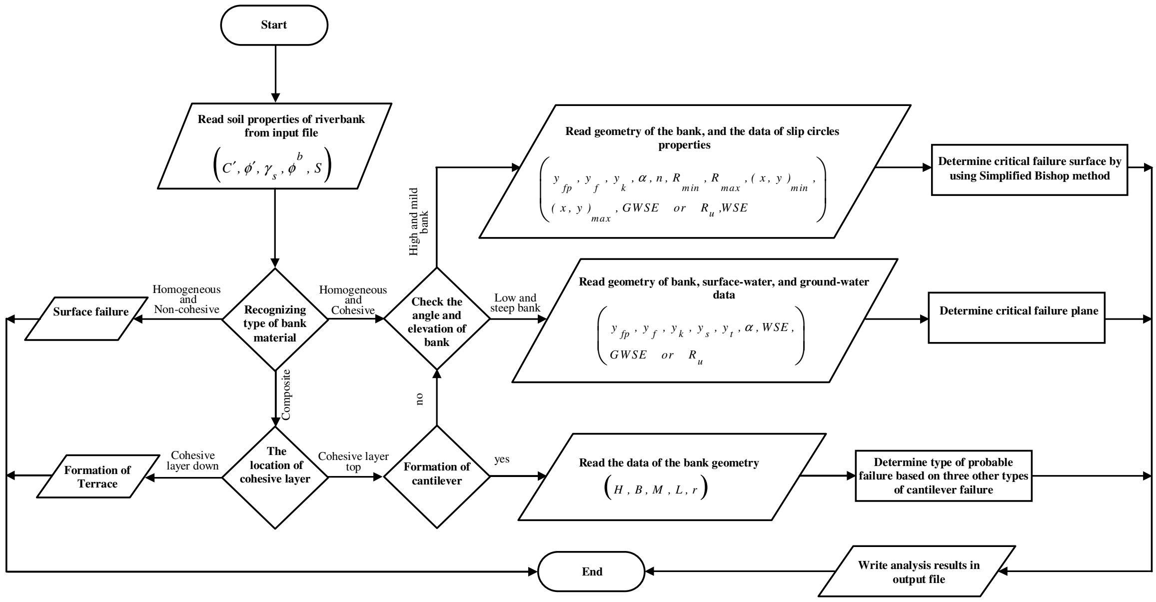

Fig 4: Diagram illustrating the computational procedure employed in EMSAR.

Comparing with the previous models, the new model also has the following advantages:

-Regarding rotational failure, the new model has ability to take into account the tension crack in upper part of the bank or on the bank slope and also considering the hydrostatic forces duo to water within both tension crack and the river channel. Besides, by using a pressure coefficient or water level, it considers the effects of pore water pressure in computations. Moreover, in the case of the availability of data regarding the equivalent height of soil suction, the new model has ability to consider the negative pore water force at each slice to analyze the stability of slices using simplified Bishap's method.

- With respect to the planar failure, in contrast to the previous models, the new model does not need either the angle of failure plane or the depth of the tension crack to analyze the stability of river bank.

- Since the stabilising potential of vegetation is evident from numerous studies, e.g. Smith (1976), Pizzuto and Melkelnberg (1989), Gray and MacDonald (1989), Thorne (1990), Millar and Quick (1998), Simon and Collison (2002) among others, the effects of vegetation on bank stability has been taken into account in terms of additional soil cohesion in the model.

There are also some limitations remain. The new model does consider the bank as homogenous with a single layer. In calculating the pore water pressure, it is also assumed that the phreatic surface is parallel to the floodplain surface. Moreover, the distribution of water pressure in the channel adjacent to the bank is assumed to be hydrostatic. In case that there is no measured data of negative pore water pressure, due to the lack of information regarding the relationship between soil moisture and the amount of matric suction for most of soil types, there is also the possibility of inaccurately estimating the effects of negative pore water pressure in the model.

3 APPLICATION OF THE MODEL

3.1 PLANAR FAILURE

In this example, the stability of river bank in a cross section of Hotophia Creek, Mississippi, has been investigated. Hotophia Creek is a sand-bedded alluvial channel located in the loess hills region of the Yazoo Basin in northern Mississippi (Amiri-Tokaldany et al., 2003). For the selected cross section, the stability analysis of river bank has been carried out by using the new model and the results have been compared to the observed related values. As it has been addressed earlier, as one of the advantages of the new model, there is no need for estimating the depth of tension crack to analysis the bank stability. In Table 1, the values of the failure block geometry; i.e. the width of bank retreat (*BW*), failure plane angle (β), the volume of the bank materials slip down through mass failure (*VB*), predicted by the new model and the observed values have been shown.

Table 1: Comparison between the predicted and observed geometry of failed block

Predicted			Observed		
β (degree)	*BW* (m)	*VB* (m^3/m)	β (degree)	*BW* (m)	*VB* (m^3/m)
45.06	1.7	10.86	57	1.4	10.29

Comparing the above results, it can be seen that the model under-predicts the value of failure block angle while there is a good agreement between predicted and observed values of width of bank retreat and volume of failed materials. The difference between the observed and predicted values of failure plane angle results from the lack of real precise data of river bank angle, and water surface in the river and in the ground.

3.2 ROTATIONAL FAILURE

Likewise, for this type of failure the stability analysis of a bank in Papillion Creek, Missouri River, (Soenksen et al., 2003), has been investigated by using the simplified Bishop's method which has been modified in this research. The stability of the bank with and without the presence of tension crack, based on aforementioned method for this type of failure, has been done by the new model and the results for the geometry of the critical sliding circle and other data have been shown in Table 2. It should be noted that the bank toe has been considered as the origin of coordinates in the example. The results show that the central point of the sliding circle is located 5.33 m far from the outside edge of the bank (directed to the river) and also there is a possibility of bank failure in this site.

Table 2: The geometry of failure circle and factor of safety for specified bank of Papillion Creek.

FS	Width of center of failure circle from toe	Height of center of failure circle from bed	Radius of failure circle	Volume of failed materials	Width of tension crack from top of bank
–	(m)	(m)	(m)	(m^3/m)	(m)
0.90	5.33	13.33	14.30	19.76	2.70

3.3 CANTILEVER FAILURE

In this example, the stability of a section of Severn River, in Wales, has been analysed against cantilever failure. In Table 3, the amount of safety factor against different types of cantilever failure along with the field observations have been shown. The results confirm the occurrence of a failure. This is because of the lack of humidity in the soil which results in the generation of tension crack in the lower part of the hanged block. Hence, by development of tension crack toward the upper part, the bank fails. As it is shown in Table 3, the factor of safety against beam failure (FSB) is less than 1 so that the bank failure is inevitable. Besides, it can be seen that the development of upper tension crack has no influence on the value of the safety factor against tensile failure. In Table 3, FSS, FSB, and FST are the safety factors against shear, beam, and tensile failures, respectively.

Table 3: The Stability of the specified bank of Severn River against cantilever failure.

Position	Time	*FSS*	*FSB*	*FST*
1	Time: 3:00 pm, Date: 15th of September 1977	10.21	2.02	∞
2	Time: 3:15 pm, Date: 17th of September 1977	8.17	1.29	6.13
3	Time: 3:30 pm, Date: 17th of September 1977	6.89	**0.92**	6.13

4 CONCLUSION

This paper presents a new model capable of analysing the stability of river banks against most common types of failure; i.e. planar, rotational, and cantilever. In the new model, the Amiri-Tokaldany's 2002 method of bank stability analysis has been modified so that it has now ability to estimate the depth of tension crack and consequently the geometry of the failure block. Also, the model has ability to consider the effects of riparian vegetation on the stability of river bank in terms of additional soil cohesion.

Regarding planar failure, by using the new model and comparing the predicted values with the observed data in a river site, e.g. Hotophia Creek, Mississippi, the results indicate that there is a good agreement between predicted and observed values of bank geometry.

With respect to the rotational type of failure, in the new model the Simplified Bishop's method has been modified so that it considers tension crack and the effect of hydrostatic pressure force produced by the water inside tension crack. Also, it takes into account the negative pore water pressure to analyze bank stability against rotational failure for river banks having different height and slopes. Having applied the new model for a river site experienced bank failure, the results indicated that the new model has ability to use for determination of the bank stability and predicting the bank geometry after possible failure.

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