An improved analysis model for cantilever failure of composite riverbanks and collapse simulation

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Abstract. The riverbanks in the Lower Jingjiang Reach are mostly composite ones with fine sand overlain by cohesive clay, thus when the sandy layer is undermined due to rapid erosion, the cohesive upper layer generates a cantilever and is prone to beam failure. In this study, the formula of critical width of cantilever failure was improved considering the hydraulic effects. The formula of accumulated width of failed blocks at bank foot was deduced. A collapse analysis model of composite banks was subsequently proposed, and the collapse process of a typical bank slope was simulated using the model. The conclusions include: The cantilever failure of composite riverbanks is more prone to occur during the period of water level decline than that of water level rise. The critical width of cantilever decreases with the dropping of river level, the weakening of the supporting role of river water and the enhancement of the overturning effect of groundwater. The thicker the cohesive layer is, the greater the decrement of its critical overhang width is as river level decline. The protective effect of failed blocks accumulated at bank foot reduces its erosion distance, the more times the bank collapse takes place, the greater the impact is.

1. Introduction
As a typical alluvial plain river, bank collapse occurs frequently in the Lower Jingjiang Reach of the Yangtze River (The LJR). The riverbanks in the reach are mostly composite ones with fine sand overlain by cohesive clay, thus when the sandy layer is gradually undermined due to current scour, the cohesive upper layer loses support and generates a cantilever, which is prone to failure under the actions of driving forces such as soil weight. The research on the analysis model for the cantilever failure of composite riverbanks can provide technical support for the prediction and prevention of bank collapse in the LJR, and is of great significance for the further enhancement of passage capacity of the Yangtze River.

Some progresses have been made in the collapse mechanism study of composite riverbanks so far. Thorne and Tovey[1] identified three types of cantilever failure and put forward corresponding calculation methods, namely shear, beam and tensile failure, among which beam failure is the most common one. Fukuoka[2] observed by his field experiment that bank retreat process involves three repetitive stages, that is the erosion of sandy lower layer, the cantilever failure of upper clay, and the removal of failed blocks accumulated at bank toe. Meanwhile, the critical overhang width of cohesive cantilever was proposed for collapse prediction. Xia[3] improved the existing formulas of the critical width of cantilever[1,2], in light of the characteristics of beam failure in the LJR. Besides self-factors (e.g. the weight and strength of soil), hydrological environment (e.g. the variation of river level and
groundwater) will affect the stability of cantilevered banks as well. The critical condition of tensile-type failure has been amended accordingly\cite{4}, yet that of beam-type cantilever failure calls for further improvement, which is the most universal type of bank collapse in the LJR.

In addition, the widely used BSTEM model mainly calculates the tensile-type failure of cantilever, and is not able to automatically simulate the piling up of failed blocks at the foot of bank\cite{5}, thus an improved analysis model needs to be proposed. In this paper, considering the impact of hydraulic actions, the formula of the critical width of beam-type cantilever failure was improved. A collapse analysis model of composite riverbanks was subsequently established to simulate three repetitive stages of bank retreat process, namely the erosion of sandy layer, the cantilever failure of cohesive upper layer, and the accumulation and removal of failed blocks at bank foot. On this basis, the collapse process of a typical bank slope in the LJR was analyzed using the model, as well as the influences of different factors such as the river level variation and the thickness of cohesive layer on bank collapse.

2. Analysis model for cantilever failure of composite riverbanks

2.1. Erosion of sandy lower layer

Based on the theory of residual shear stress, the erosion rate of riverbanks depends largely on the fluid shear stress and the soil's shear resistance to erosion. Once the fluid shear stress exceeds the soil's critical shear stress, the soil of bank slope is scoured and its lateral erosion distance can be determined as shown in equation (1):

\[
\Delta B = K \cdot \Delta t \cdot (\tau_0 - \tau_c) 
\]

\[K = 2 \times 10^{-13} \tau_c^{0.5}\]

\[
\tau_c = \gamma_w R J 
\]

Where, \(\Delta t\) is erosion duration (s), \(K\) is erosion coefficient of soil (m$^3$/N\cdot s), an empirical formula that is applicable to both cohesive and non-cohesive soils has been put forward\cite{6}, as shown in equation (2), \(\tau_c\) is the critical shear stress of soil (N/m$^2$), it can be estimated by soil erosion tests or empirical formulas, \(\tau_0\) is fluid shear stress (N/m$^2$), its calculation formula in straight channels is shown in equation (3), in which \(\gamma_w\) is unit weight of water (N/m$^3$), \(R\) is hydraulic radius (m), \(J\) is water surface slope. While the current velocity and fluid shear stress in curved channels are usually larger and need to be corrected. As for composite riverbanks, the erosion of sandy lower layer typically occurs at a much higher rate than the erosion of cohesive layer above, and a cantilever will be generated consequently.

2.2. Cantilever failure of cohesive upper layer

The width of cohesive cantilever keeps growing with the continual undermining of sandy lower layer due to current scour. Once the overhang width reaches its critical value, which means the driving moments that act on the cantilever overcomes the resistive moments, the cohesive block is prone to rotates forward about a horizontal axis somewhere in the block, i.e., a beam failure takes place.

The existing formulas of critical overhang width of beam-type cantilever failure, which mainly consider the driving moment of soil weight and the resistive moment of soil's strength in tension and compression, were further improved in this paper. Assuming that both tension stress and compression stress on the potential failure surface present triangular distribution. Considering the impact of tension crack, which often come into being at the top of bank slope before the cantilever failure occurs in the LJR. Moreover, taking the hydraulic actions (i.e., the hydrostatic confining pressures exerted by river water and that by filled water in the tension crack at bank top, the pore water pressure due to groundwater) into account, the improved stress analysis of cohesive cantilever is shown in figure 1.
2.2.1. Driving moments $M_D$

The driving moments that act on the cantilever are mainly caused by the weight of overhang block ($W$), the pore water pressure due to groundwater ($U_w$), and the hydrostatic confining pressure resulting from filled water in the tension crack ($F_{tw}$), as shown in figure 1, it is described as follows:

$$M_D = M_W + M_U + M_{tw}$$  \hspace{1cm} (4)

Where, $M_W$, $M_U$, $M_{tw}$ are the driving moments that caused by $W$, $U_w$ and $F_{tw}$ respectively (kN·m).

The moment of the weight of overhang block ($M_W$) is determined by:

$$M_W = \frac{WB}{2} = \frac{\gamma_1 H^2}{2}$$  \hspace{1cm} (5)

Where, $\gamma_1$ is the unit weight of clay (kN/m$^3$), $H$ and $B$ are the height and the width of cantilever (m).

If the groundwater level (GWSE) is above the base of cohesive layer ($y_u$), the moment of the pore pressure due to groundwater ($M_U$) is calculated by:

$$M_U = \begin{cases} \gamma_w H_0^2 \left( \frac{GWSE - y_u}{2} \left( \frac{1}{1+a} \right) + \frac{H_0}{2} \left( \frac{1}{3} - \frac{a}{1+a} \right) \right) & (GWSE > y_u) \\ \frac{1}{2} \gamma_w h_{gw}^3 \left( \frac{h_{gw}}{3} - \frac{aH_0}{1+a} \right) & (y_u < GWSE \leq y_{ki}) \end{cases}$$  \hspace{1cm} (6)

Where, $\gamma_w$ is the unit weight of water (kN/m$^3$), $y_{ki}$ is the bottom elevation of tension crack (m), $H_0$ is the height of uncracked cohesive layer (m), $a$ is the ratio of tensile strength to compressive strength of soil, $h_{gw}$ is the height of cohesive layer that submerged by groundwater (m), which satisfies the relationship that $h_{gw} = GWSE - y_u$, if $GWSE < y_u$, $h_{gw} = 0$.

If the tensile crack at bank top is filled with water (e.g. when the river level or groundwater level is high, or during heavy rainfall), the moment of extra hydrostatic pressure ($M_{tw}$) is calculated by:

$$M_{tw} = F_{tw} \left( \frac{GWSE - y_u}{3} + \frac{H_0}{1+a} \right)$$  \hspace{1cm} (7)

For cases where the river level (WSE) is higher than the groundwater level (GWSE), the equation (7) is used by replacing GWSE with WSE, while where both GWSE and WSE are below $y_{ki}$, $M_{tw} = 0$.

2.2.2. Resistive moments $M_R$

The resistive moments that act on cohesive cantilever are mainly caused by the soil’s strength in tension ($\sigma_t$) and compression ($\sigma_c$), as well as the hydrostatic confining pressure exerted by river water ($F_{rw}$), as shown in figure 1, it is described as follows:

$$M_R = \frac{1}{3} \sigma_t \frac{H_0^2}{(1+a)^2} + \frac{1}{3} \sigma_c \frac{a^2 H_0^2}{(1+a)^3} + M_{tw}$$  \hspace{1cm} (8)

Where, $\sigma_t$, $\sigma_c$ are the tension strength and the compression strength of clay respectively (kPa), $a$ is the ratio of tensile to compressive strength of soil, $M_{tw}$ is the resistive moment that caused by $F_{rw}$ (kN·m).
The hydrostatic confining pressure of river water \(F_{rw}\) can be decomposed into horizontal component \(F_x\) and vertical component \(F_y\), as shown in the hydrostatic pressure sketch of figure 1, the corresponding moments of two components \(M_x, M_y\) are calculated by:

\[
M_x = F_x \left( \frac{h_u}{3} - \frac{aH_u}{1+a} \right) = \frac{1}{2} \gamma_u h_u^3 \left( \frac{h_u}{3} - \frac{aH_u}{1+a} \right) \\
M_y = F_y B / 2 = \gamma_s h_u B^2 / 2
\]  \hspace{1cm} (9)

Where, \(h_u\) is the height of cohesive layer that submerged by river level (m), which satisfies the relationship that \(h_u = WSE - y_u\), if \(WSE < y_u\), \(h_u = 0\).

2.2.3. Critical overhang width of cohesive layer \(B_c\)

According to the equilibrium principle, when the cohesive cantilever is in its critical equilibrium state, the driving moments \(M_D\) that act on the cantilever is equal to the resistance moments \(M_R\) (i.e., \(M_D = M_R\)). Combining the equations from (4) to (10), the relationship can be described as:

\[
\frac{1}{2} (\gamma' h_u + \gamma h_u) B^2 = \frac{\sigma H_0^2}{3(1+a)} + \frac{1}{6} \gamma (h_u^3 - h_{gw}^3) - \frac{aH_u}{2(1+a)} \gamma_w (h_u^2 - h_{gw}^2)
\]

\hspace{1cm} (11)

Where, \(B_c\) is the critical width of cantilever (m); \(\gamma'\) is the weight of soil that below river level, consider the effect of vertical hydrostatic pressure \(F_y\) and take it to the submerged unit weight of soil \((kN/m^3)\); \(\gamma\) is the unit weight of soil that above river level \((kN/m^3)\); \(h_u\) is the height of uncracked cohesive cantilever \((m)\), i.e., \(h_u = H - h_u\), \(H\) is the total height of cohesive cantilever \((m)\); \(H_0\) is the height of uncracked cohesive soil layer \((m)\), which satisfies the relationship that \(H_0 = H - H_t\); \(H_t\) is the depth of tension crack at bank top \((m)\), which can be estimated by the empirical formula that \(H_t = 1.34c\tan(45^\circ + \varphi/2)/\gamma_t\) [7], \(c\) and \(\varphi\) are the cohesion \((kPa)\) and the friction angle \((^\circ)\) of clay respectively.

Take \(a = 0.1[3]\), the formula of critical overhang width of cohesive upper layer can be simplified as:

\[
B_c = \left[ \frac{0.606 \sigma H_0^2 + 0.333 \gamma (h_u^3 - h_{gw}^3) - 0.0909 \gamma_u H_0 (h_u^2 - h_{gw}^2)}{\gamma h_u + \gamma h_u} \right]^{1/2}
\]

\hspace{1cm} (12)

According to the relative size between the actual width \(B\) of a cantilever and its critical value \(B_c\), whether it is stable or not can be judged, for the cases where \(B > B_c\), a cantilever failure takes place.

In order to verify the applicability of above formulas, equation (12) was applied to Fukuoka’s field experiment[2] and the calculated critical widths of cantilever were compared with his measured values, as shown in figure 2. It can be seen that the calculated results are basically consistent with the measured values, especially when the thickness of cohesive layer is larger than 100cm.

2.3. Accumulation and removal of failed blocks

After the occurrence of a cantilever failure, the failed cohesive block temporarily piles up at the foot of bank and forms a triangular covering, the slope of which \((\alpha)\) is approximately equal to the repose
angle of sediment\cite{8}, as shown in figure 3. The formula of accumulated width of failed blocks at bank foot was deduced based on these assumptions.

The accumulated volume of failed blocks is a dynamic value, which increases with the occurrences of bank collapse and decreases as the flow scouring. The accumulated volumes that before the collapse ($V_c$), during the collapse ($V$) and after the collapse ($V_t$) can be described as:

$$ V_c = V_t + V $$

(13)

$$ V = K_b B_c H_1 $$

(14)

Where, $K_b$ is the volume ratio of accumulated soil to collapsed block, the stronger the current scour is, the smaller the value of $K_b$ is, $B_c$ and $H_1$ are the width and the height of collapsed block respectively.

As for two-dimensional calculation section, the volume, the height and the width of accumulated soil at the foot of bank satisfy the geometric relationship as follows:

$$ V_e = 0.5h^2_{ce} \left( \cot \alpha - \cot \beta \right) $$

(15)

$$ L_{ce} = \left( \cot \alpha - \cot \beta \right) h_{ce} $$

(16)

$$ V_t = 0.5h^2_{c} \left( \cot \alpha - \cot \beta \right) $$

(17)

$$ L_{c} = \left( \cot \alpha - \cot \beta \right) h_{c} $$

(18)

Where, $V_e$, $h_{ce}$, $L_{ce}$ are the volume (m$^3$), the height and the width (m) of accumulated soil that before the collapse; $V_t$, $h_c$, $L_c$ are the volume (m$^3$), the height and the width (m) of accumulated soil that after the collapse; $\alpha$ and $\beta$ are the slops of accumulated soil and of bank slop respectively (°).

Combining the equations from (13) to (18), the formula of accumulated width of failed blocks at bank foot can be simplified as:

$$ L_c = \sqrt{2(V + V_t) \left( \cot \alpha - \cot \beta \right)} = \sqrt{2K_b B_c H_1 \left( \cot \alpha - \cot \beta \right) + L^2_{ce}} $$

(19)

Since the critical shear stress of cohesive soil is greater than that of sandy soil, the protective effect of accumulated failed blocks enhances the shear resistance of bank foot to erosion and reduces its erosion rate\cite{8}, until the cohesive covering is completely removed due to current scour.

2.4. Establishment and verification of analysis model

An analysis model for cantilever failure of composite riverbanks is established to simulate three main stages of bank collapse process, as shown in figure 4. Firstly, equation (1) is used to calculate the erosion of sandy lower layer and determine its retreat distance. Next, equation (12) is applied to acquire the critical overhang width of cohesive upper layer and judge its stable status accordingly. If a cantilever failure takes place, equation (19) is used to simulate the piling up of failed blocks at bank foot, as well as its later removal due to current scour.
In order to verify the reliability of above analysis model, the model was applied to simulate the retreat process of a typical riverbank in the LJR\cite{5} by programming. The result was compared with that calculated by BSTEM model and with the field data\cite{5}, as shown in figure 5. It is apparent that the calculated retreat distance of bank top (70m) is in good agreement with the actual value (72m), and the difference between the simulated section and the measured one is relatively small.

3. Simulation of cantilever failure of composite riverbanks

\textbf{3.1. Typical section of composite riverbanks in the LJR}

A typical section of composite riverbanks was selected to conduct the research, which is seated at 200m upstream of slope protection engineering of Zhongzhouzi Reach in the LJR, as shown in figure 6. The elevations of the top and the base of bank slope are 34.5m and 13.0m respectively (Yellow Sea Datum), as shown in figure 7. The slope section consists of three layers of soil, while the collapse analysis for cantilever failure mainly involves the upper two: the first layer made up of silty clay, with a thickness of 3.5m, and the second layer lays incompact fine sand, with a thickness of 17-23m. According to the results of laboratory soil tests, the soil parameters of each layer are shown in Table 1.

In accordance with the monitored data of hydrological station nearby, the river level variation in the LJR can be generalized into two stages, i.e., the rising stage and the falling stage. The water level rises from 24m to 34m at a speed of 0.2 m/s constantly in the rising stage, while it decreases from 34m to 24m at a constant speed of 0.2m/s during the period of water level decline, and both periods lasts for 50 days. On this basis, the collapse processes of composite riverbanks during two periods were simulated using the improved model respectively. The calculation cases are shown in Table 2.
Table 1. Soil parameters

| Types of soil | Unit weight (kN/m³) | Cohesion (kpa) | Friction angle (°) | Erosion coefficient (m³/N·s) | Critical shear stress (N/m²) |
|---------------|---------------------|----------------|-------------------|-----------------------------|-----------------------------|
| Silty clay    | 18.6                | 25.0           | 18.3              | 0.141                       | 0.50                        |
| Fine sand     | 20.2                | 10.4           | 19.3              | 0.354                       | 0.08                        |

Table 2. Calculation cases

| Case | River level variation | Water surface slope (m/m) | Tensile strength of clay (kpa) | Thickness of cohesive layer (m) |
|------|-----------------------|---------------------------|--------------------------------|--------------------------------|
| 1    | Falling from 34m to 24m | 0.012                     | 4                              | 3.5                            |
| 2    | Rising from 24m to 34m  | 0.008                     | 7                              | 3.5                            |

3.2. Influence of river level variation on cantilever failure of composite riverbanks

3.2.1. Influence of river level variation on critical overhang width of cantilever failure

The analysis model judges the occurrence of cantilever failure of composite riverbanks according to the relative size between the overhang width of cohesive upper layer and its critical value, the critical formula of which has been improved by taking the hydraulic effects into account. The critical overhang widths during the falling period (Case 1) and the rising period (Case 2) of river level, calculated by the formulas that improved and unimproved respectively, are shown in figure 8.

![Figure 8](image.png)

Figure 8. Critical widths of cantilever in the cases of different river level variations

As can be seen from figure 8 that the critical width of cantilever is a fixed value mainly related to the soil properties, when not considering external hydraulic actions. And the fixed critical value in Case 1 is smaller than that in Case 2. The high water level at the initial stage of river level decline immerses the clay, reducing its tensile strength and the critical width during that period as a result.

After taking the hydraulic effects into account, the supporting role of river water (i.e., its hydrostatic confining pressure) is conducive to the stability of cantilever, and enlarges the critical width during the high water level period consequently. The beneficial impact diminishes as the river level decreases, thus the critical values in two cases both show a positive correlation with the water level. Moreover, the critical widths tend to be fixed ones that independent of hydrological environment once the river level varies to below 31m, where is approximately the interface of two soil layers.

3.2.2. Influence of river level variation on collapse process of composite riverbanks

The collapse processes of composite riverbanks during the period of river level decline (Case 1) and that of river level rise (Case 2) are shown in figure 9 and figure 10 respectively. And the specific bank collapses in two cases are compared as shown in table 3.
Figure 9. Collapse process as river level decline

Figure 10. Collapse process as river level rise

Table 3. Comparison of bank collapse in the cases of different river level variations

|                         | Case 1 (River level decline) | Case 2 (River level rise) |
|-------------------------|------------------------------|---------------------------|
| Erosion rate (×10^6 m/s)| 2.56                         | 0.86                      |
| Erosion distance (m)    | 10.6 (16.8)                  | 9.0 (11.0)                |
| Collapse times          | 9                            | 3                         |
| Specific bank collapse  | 1 2 3 5 7 9 1 2 3            |
| Collapse water level (m)| 33.4 32.9 32.5 31.8 31.2 29.7 31.6 32.3 33.1 |
| Collapse width (m)      | 1.25 1.06 0.83 0.60 0.58 0.55 0.77 0.92 1.09 |
| Accumulated width (m)   | 0.93 0.47 0.32 0.21 0.20 0.35 0.73 0.48 0.48 |

a Erosion rate refers to the maximum erosion rate of sandy soil at the foot of bank (elevation of 13m).
b Erosion distance refers to the total erosion distance of bank foot that considers the protective effect of accumulated collapsed soil, while the value in parentheses refers to one that not considers the effect.

Combing with the figures and table 3, the retreat distance and the total collapse times of bank slope in Case 1 are both larger than that in Case 2. On the one hand, the critical width of cohesive cantilever in Case 1 is smaller because of the smaller tensile strength of clay. On the other hand, in accordance with relevant hydrological statistics, the water surface slope of river (J) in Case 1 is larger than that in Case 2, which leads to a higher speed of sandy layer retreating and of upper cantilever widening. The relatively large overhang width of cohesive layer and its relatively small critical value increase the frequency of cantilever failure of composite riverbanks during river level declining. Besides, it seems to be a dangerous period of bank collapse when the river level changes close to the interface of two soil layers (33m-30m), which may be related to the reduction of the critical overhang width to a fixed minimum value at that time.

Although the erosion rate of sandy soil at the foot of bank in Case 1 is much larger than that in Case 2, yet the final erosion distances of bank foot in two cases are relatively close, which is attributed to the different protective effects of collapsed soil: In Case 1, the bank collapsed for first time at the initial stage of river level declining (33.4m), since then the collapsed soil piled up at the foot of bank and enhanced its resistance to erosion. The cohesive covering was not completely removed until the middle and late stage (28.4m). Thus the bank foot was continuously scoured at a relatively small rate during a long period of time (i.e., during the water level decreased from 33.4m to 28.4m), which reduced its final erosion distance to a large degree. While the bank collapsed for first time in Case 2 at the later stage of water level rising (31.6m). Only during a short period of time (i.e., during the water level rose from 31.6m to 34m) was the bank toe influenced by the protective effect of collapsed soil and slowed down its scouring rate. Therefore, it can be concluded that the covering of collapsed soil accumulated at bank foot affects the bank retreat process. The earlier the bank collapses for first time, the more times the collapse occurs, and the longer the covering duration is, the greater the impact is.
3.3. Influence of thickness of cohesive layer on critical overhang width of cantilever failure

Based on Case 1 (Period of water level decline, and cohesive layer thickness of 3.5m), Case 3 and Case 4 took the thickness of cohesive upper layer as 1m and 5m respectively, under the condition of keeping the total height of bank slope 21.5m unchanged. The critical overhang widths during the period of river level decline in three cases (Case 1, Case 3 and Case 4) are shown in figure 11.

![Critical overhang widths in the cases of different thicknesses of cohesive upper layer](image)

Figure 11. Critical overhang widths in the cases of different thicknesses of cohesive upper layer.

According to previous analysis, the supporting role of river water enlarges the critical width of cantilever during the high water level period, thus the critical widths in three cases both decrease with the declining of river level and tend to be fixed values after the water level drops to the interface of two soil layers, as shown in figure 11. The thicker the cohesive upper layer, the greater the impact of river body on the stability of cantilever, the larger the decrement of critical width is as river level decline. The critical values in Case 3, Case 1 and Case 4 decrease to fixed ones separately after the water level drops to 33.5m, 31m and 29.5m, with respective decrements of 20%, 66% and 89%.

Moreover, the overturning effect of groundwater (i.e., its pore pressure, mean level of 32m), which is unfavorable to the stability of cantilever, becomes gradually distinct as river level decline. The overhang blocks in Case 3 (bottom elevation of 33.5m) and Case 1 (bottom elevation of 31m) are not adversely affected by groundwater due to their relatively high bottom elevation. While the cantilever in Case 4 (bottom elevation of 29.5m) is greatly influenced by hydraulic effects, its critical width decreases sharply with the dropping of river level, the weakening of the supporting role of river water and the enhancement of the overturning effect of groundwater. More specifically, its critical width reduces from the maximum one of three cases at the initial stage to the minimum one at the later stage.

4. Conclusions

In this study, the formula of critical overhang width of beam-type cantilever failure was improved, taking the hydraulic effects into account. The formula of accumulated width of failed blocks at bank foot was deduced. And a collapse analysis model of composite riverbanks was subsequently established to simulate three main stages of bank retreat process. On this basis, the collapse process of a typical bank slope in the LJY was simulated using the model, the conclusions include:

1. The occurrence of cantilever failure of composite riverbanks can be judged by the relative size between the actual width of cantilever and its critical value. The overhang part of cohesive upper layer is generated and widened by the erosion of sandy lower layer, thus the increase rate of its overhang width is proportional to the water level. The critical width decreases with the dropping of river level, the weakening of the supporting role of river water and the enhancement of the overturning effect of groundwater. When the river level changes close to the interface of two soil layers, the relatively large overhang width and the small critical value makes it a dangerous period of cantilever failure.

2. The cantilever failure of composite riverbanks is more prone to occur during the period of river level decline than that of water level rise. The thicker the cohesive upper layer is, the greater the impact of hydraulic actions on the stability of cantilever is, i.e., the greater the decrement of its critical width is with river level dropping.

3. The protective effect of collapsed soil accumulated at bank foot reduces its erosion distance. The
more times the collapse takes place, the longer lasting the cohesive covering, the greater the impact is.

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