

# Dry-season drawdown primes cantilever failure of stratified riverbanks in the Vietnamese Mekong Delta: geotechnical and probabilistic evidence from An Giang

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## Research Article

**Keywords:** riverbank failure, cantilever mechanism, drawdown, tension crack, Mekong Delta, slope stability

**Posted Date:** July 20th, 2026

**DOI:** <https://doi.org/10.21203/rs.3.rs-10268277/v1>

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# Abstract

Riverbank collapse in the Vietnamese Mekong Delta is usually assessed as a rotational slope-stability problem driven by flood-season loading. Field data from An Giang Province suggest a different mechanism. Six 30 m boreholes with 90 sampled horizons were drilled in December 2025 at three collapse-prone sites on the Hau River, the Tien River, and the Rach Gia–Long Xuyen canal. The logs show a cohesive cap of clay and clayey silt, 9 to 10 m thick, over fine sand at the two river sites. Estimated critical shear stress of the sand is about twenty times lower than that of the cap, so scour concentrates at the toe. Surveyed cross-sections confirm steep subaqueous slopes and pools reaching – 30 m. Daily stage records show recession rates of up to 0.15 m per day, while borehole groundwater stood 1.2 to 2.0 m above the dry-season river level. Bishop analyses of the surveyed sections give factors of safety of 1.84 to 2.02 in all hydraulic scenarios: deep rotational sliding is not critical. A cantilever analysis of the undercut cap instead gives a critical niche depth of only 0.9 m, which drops to 0.6 to 0.8 m when recession-season groundwater fills the tension crack. Monte Carlo simulation yields cantilever failure probabilities above 50 percent for a 0.6 m niche, against below 0.1 percent for rotational sliding. In the thick soft mud of the canal site the ranking reverses. A framework for selecting the governing mechanism is proposed.

## 1 Introduction

Bank retreat along the distributary network of the Mekong Delta has accelerated over the past two decades. Collapses have displaced riparian communities and destroyed roads, markets, and housing in An Giang, Dong Thap, and Can Tho provinces (Anthony et al. 2015; Tuan 2025). The Vam Nao failure of April 2017 in Cho Moi District removed more than 70 m of bank and sixteen houses within minutes (MONRE 2017). Such events are episodic and fast. Regional studies point to a sediment-starved river system. Upstream impoundment has cut the suspended load reaching the delta by roughly half (Kondolf et al. 2018). Instream sand mining lowers channel beds and steepens the subaqueous bank slope (Brunier et al. 2014; Hackney et al. 2020). Groundwater-driven subsidence adds to the problem by reducing bank freeboard (Minderhoud et al. 2017).

Despite this well-documented forcing, the mechanics of individual failures remain poorly resolved. Existing studies fall into three groups. The first documents the spatial pattern and consequences of erosion (MONRE 2017; Tuan 2025). The second deploys Internet of Things (IoT) instrumentation to detect movement for early warning, but treats the bank itself as a black box (Dang et al. 2025; Lee and Osman 2026). The third applies slope-stability software to single cross-sections under assumed hydraulic scenarios, usually with a rotational failure surface by default (Nguyen et al. 2024). None of them tests competing failure mechanisms against site data. None quantifies the uncertainty of its stability estimates. A recurrent paradox is also left unexplained: many collapses occur in the dry season, weeks to months after the flood peak (Anthony et al. 2015; MONRE 2017).

Composite banks consist of an erodible basal unit under a cohesive cap. On temperate rivers they are known to fail by cantilever mechanisms once toe scour undermines the cap (Thorne and Tovey 1981; Osman and Thorne 1988; Simon et al. 2000). Rapid drawdown is a recognized destabilizing condition as well, because bank drainage lags the falling stage (Rinaldi and Casagli 1999; Rinaldi et al. 2008). Whether these two controls, and their interaction, govern the banks of a large tropical delta has not been established with site-specific geotechnical data.

This study uses new borehole, laboratory, hydrological, and bathymetric data from three sites in An Giang Province to test three hypotheses. H1: the critical hydraulic window is the post-monsoon recession, when bank drainage lags the falling stage, not the flood peak. H2: the erodibility contrast between the basal sand and the cohesive cap concentrates erosion at the toe, so an undercut niche, not a loss of overall shear strength, controls the onset of instability. H3: a cantilever mechanism on the undercut cap governs failure of the stratified river banks, whereas the rotational mechanism assumed in routine practice governs only where thick, very soft mud is present. The hypotheses are tested with limit-equilibrium and cantilever analyses of the surveyed sections. Predictive uncertainty is quantified by Monte Carlo simulation.

## 2 Study sites and data

Three collapse-prone sites in An Giang Province were investigated (Fig. 1a). Site CD lies on the right bank of the Hau River at Cai Dau market, Chau Phu District. Site LDA lies on the right bank of the Tien River at Long Dien A, Cho Moi District, within the reach influenced by the Vam Nao junction. Site MK lies on the Rach Gia–Long Xuyen canal at My Khanh, Long Xuyen City. All three have documented erosion damage and are covered by the provincial monitoring programme (MONRE 2017).

The geotechnical campaign was carried out in December 2025. Six rotary-wash boreholes, two per site, were drilled to 30 m. Ninety undisturbed samples were taken at 2 m intervals and tested for nine physical and mechanical indices, including direct shear and oedometer tests, following the current Vietnamese standards (TCVN 4195–4202:2012). Standard penetration tests (SPT) were performed every 2 m, fifteen per hole. Static groundwater depths were recorded in every hole. Table 1 summarizes the layer-average properties.

Two further datasets are used. Daily water levels cover the 2016 low-water year at Cai Dau and Long Dien, and the 2020 average year at My Khanh. Bathymetric cross-sections of the Hau and Tien channels at CD and LDA were surveyed in early 2026 (Fig. 1c). The stratigraphy is consistent between sites and typical of the delta plain: a thin sand fill, a stiff to soft cohesive package about 8 to 10 m thick, and medium-dense fine sand below (Fig. 1b). The exception is MK, where a 16 m layer of very soft organic mud (SPT 1–3, void ratio 1.90, liquidity index 1.12) lies directly beneath the upper clay.

Table 1

Layer-average geotechnical properties from the December 2025 campaign (direct shear, saturated; SPT ranges from 15 tests per hole)

| Property                       | CD L1<br>clay | CD L2<br>clayey<br>silt | CD L3<br>fine<br>sand | LDA L1<br>clay | LDA L2<br>clayey<br>silt | LDA L3<br>fine<br>sand | MK L2<br>soft<br>mud |
|--------------------------------|---------------|-------------------------|-----------------------|----------------|--------------------------|------------------------|----------------------|
| Thickness (m)                  | 3.8–4.0       | 3.8–4.0                 | > 20                  | 4.0–4.4        | 4.0                      | > 20                   | 16.0                 |
| $\gamma$ (kN m <sup>-3</sup> ) | 18.2          | 18.2                    | 18.0                  | 19.5           | 17.7                     | 18.2                   | 15.4                 |
| c (kPa)                        | 17.8          | 17.7                    | 4.5                   | 31.4           | 15.2                     | 4.9                    | 8.0                  |
| $\varphi$ (°)                  | 11.0          | 13.1                    | 31.5                  | 15.6           | 11.2                     | 30.7                   | 3.3                  |
| Void ratio $e_0$               | 0.93          | 0.93                    | 0.76                  | 0.72           | 1.07                     | 0.75                   | 1.90                 |
| Liquidity index                | 0.53          | 0.59                    | –                     | 0.15           | 0.65                     | –                      | 1.12                 |
| SPT $N_{30}$                   | 5–7           | 5–9                     | 9–28                  | 9–11           | 5–7                      | 9–30                   | 1–3                  |

## 3 Methods

### 3.1 Erodibility of the bank units

Fluvial erosion of each unit is described by the excess shear-stress law (Partheniades 1965; Hanson and Simon 2001),

$$E = k_d (\tau_b - \tau_c)$$

1

where  $E$  is the erosion rate,  $k_d$  the erodibility coefficient,  $\tau_b$  the boundary shear stress, and  $\tau_c$  the critical shear stress. Jet erosion tests were not available, so  $\tau_c$  was estimated from the measured silt–clay fraction  $SC$  with the empirical relation of Julian and Torres (2006),

$$\tau_c = 0.1 + 0.1779 SC + 0.0028 SC^2 - 2.34 \times 10^{-5} SC^3$$

2

and  $k_d$  from the inverse relation of Hanson and Simon (2001),  $k_d = 0.2 \tau_c^{-0.5}$ . These estimates carry large uncertainty; they are used only to rank the units and are varied in the Monte Carlo analysis.

### 3.2 Rotational stability

Rotational stability of the surveyed sections was computed with the simplified method of Bishop (1955),

$$F_S = \frac{\sum_i [cb_i + (W_i - u_i b_i) \tan \varphi] / m_\alpha}{\sum_i W_i \sin \alpha_i}$$

3

where  $b_i$ ,  $W_i$ ,  $u_i$ , and  $\alpha_i$  are the width, weight, base pore pressure, and base inclination of slice  $i$ , and  $m_\alpha$  is the usual Bishop term. The solver was implemented in Python and searches 6,762 trial circles per scenario on the measured geometry. Pore pressures follow a phreatic surface that connects the river level at the bank face to the borehole groundwater level 30 m inland. Three scenarios are analysed per site: flood stage with an equilibrated bank; dry-season minimum stage with the measured (lagged) groundwater level; and the same stage with a hypothetically drained bank. External water on the submerged face is included as a stabilizing load. The MK canal site is treated separately, because the mud is nearly frictionless. Its short-term stability is expressed with the classical stability number  $N_s = 5.53$  for a toe circle (Taylor 1937),  $FS = N_s s_u / (\gamma h)$ . The undrained strength  $s_u$  comes from the direct-shear envelope.

### 3.3 Cantilever stability of the undercut cap

Toe scour in the basal sand undermines the cohesive cap. The cap is treated as a block of height  $H$  bounded by a vertical tension crack of depth  $z_c$  (Thorne and Tovey 1981; Osman and Thorne 1988). The crack depth follows Rankine theory,

$$z_c = \frac{2c}{\gamma} \tan \left( 45^\circ + \frac{\varphi}{2} \right)$$

4

Moment equilibrium about the inner edge of a niche of depth  $B$  gives the factor of safety against toppling,

$$F_c = \frac{\sigma_t (H - z_c)^2 / 6}{\frac{\gamma B^2 H}{2} + \frac{\gamma_w z_w^3}{6}}$$

5

where  $\sigma_t$  is the tensile strength of the cap, taken as  $c/2$ ,  $\gamma_w$  the unit weight of water, and  $z_w$  the depth of water in the crack. Setting  $F_c = 1$  defines the critical niche depth,

$$B_{cr} = \sqrt{\frac{\sigma_t (H - z_c)^2 - \gamma_w z_w^3}{3\gamma H}}$$

6

Two crack conditions are compared: dry ( $z_w = 0$ ) and filled ( $z_w = z_c$ ), the latter representing the recession season, when the measured groundwater level stands above the crack bottom.

### 3.4 Uncertainty analysis

Parameter uncertainty was propagated by Monte Carlo simulation with 10,000 realizations per case. Cohesion, friction angle, unit weight, and the tensile ratio  $\sigma_t/c$  were treated as lognormal variates with coefficients of variation of 0.25, 0.08, 0.03, and 0.20, consistent with reported geotechnical variability (Phoon and Kulhawy 1999). The probability of failure is

$$P_f = \frac{1}{N} \sum_{i=1}^N \mathbb{I}[F_i < 1]$$

7

where  $\mathbb{I}$  is the indicator function and  $N$  the number of realizations. Sensitivity was assessed by rank correlation between inputs and computed factors of safety. The Bishop, cantilever, and Monte Carlo routines were implemented in Python; a large language model (Claude, Anthropic) was used to assist in drafting the analysis code and structuring the manuscript text, and all code, data, and results were reviewed and verified by the authors.

## 4 Results

### 4.1 Hydrological priming during recession (H1)

Figure 2 summarizes the hydrological evidence. In the 2016 low-water year the stage at Cai Dau ranged from + 0.10 to + 1.81 m and at Long Dien from + 0.09 to + 1.53 m. The fastest sustained recession reached 0.148 m per day at Cai Dau and 0.138 m per day at Long Dien, on the falling limbs of post-flood surges (Fig. 2b). The December 2025 boreholes recorded bank groundwater at + 2.14 m (CD) and + 1.86 m (LDA). These levels sit 1.2 to 1.4 m above the concurrent December river stage and 1.8 to 2.0 m above the dry-season minimum. The bank therefore drains far more slowly than the river falls. An outward hydraulic gradient persists through the entire recession and low-water period (shaded band in Fig. 2a). At MK the 2020 canal record shows a damped regime, with a range of only 0.56 m and recession rates below 0.06 m per day. H1 is supported: the recession, not the flood peak, is the window in which the bank carries excess pore pressure and loses the confining water load.

### 4.2 Stratigraphic control of toe scour (H2)

The borehole logs give the two river banks the same architecture: a cohesive cap of 9.4 m (LDA) to 9.7 m (CD) over fine sand (Table 1; Fig. 1b). The correlation-based erodibility estimates separate the units sharply. The cap units have  $\tau_c$  of 19 to 22 Pa. The basal sand has  $\tau_c$  of 0.9 to 1.2 Pa and a  $k_d$  four to five times higher. The sand is therefore stripped at boundary shear stresses an order of magnitude below the threshold of the cap, and erosion concentrates below the cap base. The surveyed geometry is consistent

with this. Near-bank subaqueous slopes at LDA reach a gradient of about 1V:2.5H in the upper 6 m (Fig. 1b). Both LDA sections also contain scour pools deeper than – 29 m in the Vam Nao confluence zone (Fig. 1c). There, confluence hydraulics and reported bed mining act together (Brunier et al. 2014; Hackney et al. 2020). H2 is supported: the geometry required for undercutting exists, and it is being maintained by erosion of the sand, not of the cap.

## 4.3 Competing mechanisms (H3)

Bishop analyses of the measured sections give minimum factors of safety of 1.87 to 2.02 at CD and 1.84 to 1.87 at LDA (Fig. 3a). The spread across the three hydraulic scenarios is smaller than 0.13. Deep rotational sliding is therefore neither critical nor sensitive to drawdown at the river sites: the critical circles gain most of their resistance in the medium-dense sand ( $\phi \approx 31^\circ$ ), which drawdown barely affects. The cantilever mechanism behaves differently (Fig. 3b). With the measured cap properties, the Rankine crack depth is 2.2 m (CD) and 3.0 m (LDA). The critical niche depth  $B_{cr}$  is only 0.91 to 0.92 m with a dry crack. When the crack fills,  $B_{cr}$  drops to 0.79 m at CD and 0.59 m at LDA. The filled condition is not hypothetical. At LDA the December groundwater level stands 2.8 m above the crack bottom, so any crack formed during recession fills from the bank itself.

The Monte Carlo results sharpen the contrast (Fig. 4a). Rotational failure probability stays below 0.1 percent at both river sites in all scenarios. Cantilever failure probability with a filled crack reaches 52 percent at LDA and 9 percent at CD for a 0.6 m niche, and exceeds 95 percent at both sites for a 1.0 m niche. Sensitivity analysis (Fig. 4b) shows that the tensile ratio  $\sigma_t/c$  dominates the resisting side. Cohesion itself has a negative net effect in the filled-crack condition: higher cohesion deepens the tension crack and enlarges the water thrust faster than it adds tensile resistance. Friction angle, the parameter that governs rotational analyses, is almost irrelevant. At MK the ranking reverses. The 16 m mud layer gives undrained factors of safety of 1.12, 0.88, and 0.74 for bank heights of 3, 4, and 5 m. Rotational failure through the mud governs, in line with the field classification of this canal reach as the highest-risk site of the three. H3 is supported at all three sites.

## 5 Discussion

### 5.1 A three-stage mechanism

The data support a three-stage description of bank collapse at the river sites. Stage 1 is preparation. Near-bank flow strips the basal sand year-round, because its erosion threshold is an order of magnitude below that of the cap; confluence scour and bed mining deepen the channel and keep the near-bank slope steep. Stage 2 is priming. During post-monsoon recession the river falls by up to 0.15 m per day while the bank drains slowly. The measured groundwater excess of 1.2 to 2.0 m drives outward seepage, and any tension crack fills from the bank side. The filled crack cuts the critical niche depth by 13 to 36 percent. Stage 3 is release. Once the niche in the sand reaches  $B_{cr}$ , less than one metre at both sites, the cap topples as a block. The mechanism explains why collapses cluster in the dry season: the flood builds the niche, but the recession pulls the trigger.

## 5.2 Selecting the governing mechanism

The three sites span the two end members of delta bank behaviour. Where a cohesive cap overlies clean sand, rotational analysis is misleading: it returns comfortable factors of safety, near 1.9, that barely respond to hydrology, while the actual margin is a sub-metre niche depth. Where thick, very soft mud is present, as at MK, the classical rotational mechanism governs, and bank height is the controlling variable. A practical screening rule follows from the data. If the boreholes show  $s_u$  below about 10 kPa over more than half the bank height, treat the bank as rotational and limit its height; here 3 m is the stability limit of the canal banks. Otherwise, treat the bank as a cantilever problem and manage the toe. This ranking is transferable to other tide-influenced deltas with similar Holocene successions, such as the Ayeyarwady, the Ganges–Brahmaputra, and the Chao Phraya (Thorne and Tovey 1981; Simon et al. 2000; Langendoen and Simon 2008).

## 5.3 Implications for monitoring and management

Three practical points follow. First, the highest-value survey measurement is the toe, not the visible bank face. The critical niche depth is smaller than 1 m, so periodic sonar or multibeam profiles of the sub-slope, at sub-metre resolution, are needed to see the hazard at all. Second, recession rate should be treated as a controllable hazard variable where regulation is possible; the canal record at MK shows how strongly water-level management damps both the range and the recession rate. Third, crest tension cracks are a cheap and meaningful precursor. Because the filled crack is what collapses the safety margin, extensometers or even periodic crack mapping complement the IoT displacement systems now being trialled in the delta (Dang et al. 2025; Lee and Osman 2026).

## 5.4 Limitations

Several limitations remain. Erodibility was estimated from grain-size correlations, not measured; jet erosion tests are the obvious next step. The groundwater levels come from a single December campaign, so the seasonal gradient history is bracketed only by the stage records. No instrumented failure was captured; the timing argument rests on the hydraulic evidence and on regional inventories. The stage records at the river sites predate the geotechnical campaign by nine years, and the analysis is two-dimensional. Unsaturated strength above the phreatic surface was neglected (Fredlund et al. 1978), which is conservative for the rotational case. A permanent piezometer and crack-monitoring array at LDA is the logical continuation of this work.

## 6 Conclusions

Borehole, laboratory, hydrological, and bathymetric data from three sites in An Giang Province identify the governing mechanisms of bank failure in the central Mekong Delta. At the Hau and Tien river sites the bank is a 9 to 10 m cohesive cap over fine sand whose erosion threshold is roughly twenty times lower. Bishop analyses of the surveyed sections give factors of safety near 1.9 in every hydraulic scenario, so deep rotational sliding is not the critical mechanism. The critical control is a scour niche in

the sand: 0.9 m suffices to topple the cap, and only 0.6 to 0.8 m once recession-season groundwater fills the tension crack. Measured groundwater stood 1.2 to 2.0 m above the low-water river stage, confirming that the filled-crack condition is the normal dry-season state. Monte Carlo analysis puts cantilever failure probability above 50 percent for a 0.6 m niche at Long Dien A, against below 0.1 percent for rotational sliding. In the thick soft mud of the Rach Gia–Long Xuyen canal the ranking reverses and bank height governs, with a stability limit near 3 m. Managing the toe, the recession rate, and the crest cracks, rather than re-analysing rotational circles, is where monitoring and mitigation effort should go.

## Declarations

### Funding

The geotechnical investigation was carried out within a science and technology research project of An Giang Province.

### Competing interests

The authors have no relevant financial or non-financial interests to disclose.

### Author contributions

Both authors contributed to the study conception and design. Hoa Thanh Thi Nguyen compiled the field and laboratory data, performed the analyses, and wrote the first draft of the manuscript. Giang Song Le contributed to the interpretation of the results and commented on previous versions of the manuscript. Both authors read and approved the final manuscript.

### Data availability

The borehole logs, laboratory test results, daily water-level records, and surveyed cross-sections analysed in this study were produced within the An Giang provincial research programme (drilling and testing by TEKCO Foundation and Construction Consulting JSC, December 2025). They are available from the corresponding author on reasonable request. The Python analysis scripts are available from the corresponding author.

### Use of artificial intelligence

A large language model (Claude, Anthropic) was used to assist in drafting and structuring the manuscript text and the analysis code, as documented in Sect. 3.4. The authors reviewed and verified all data, analyses, and text, and take full responsibility for the content.

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## Figures

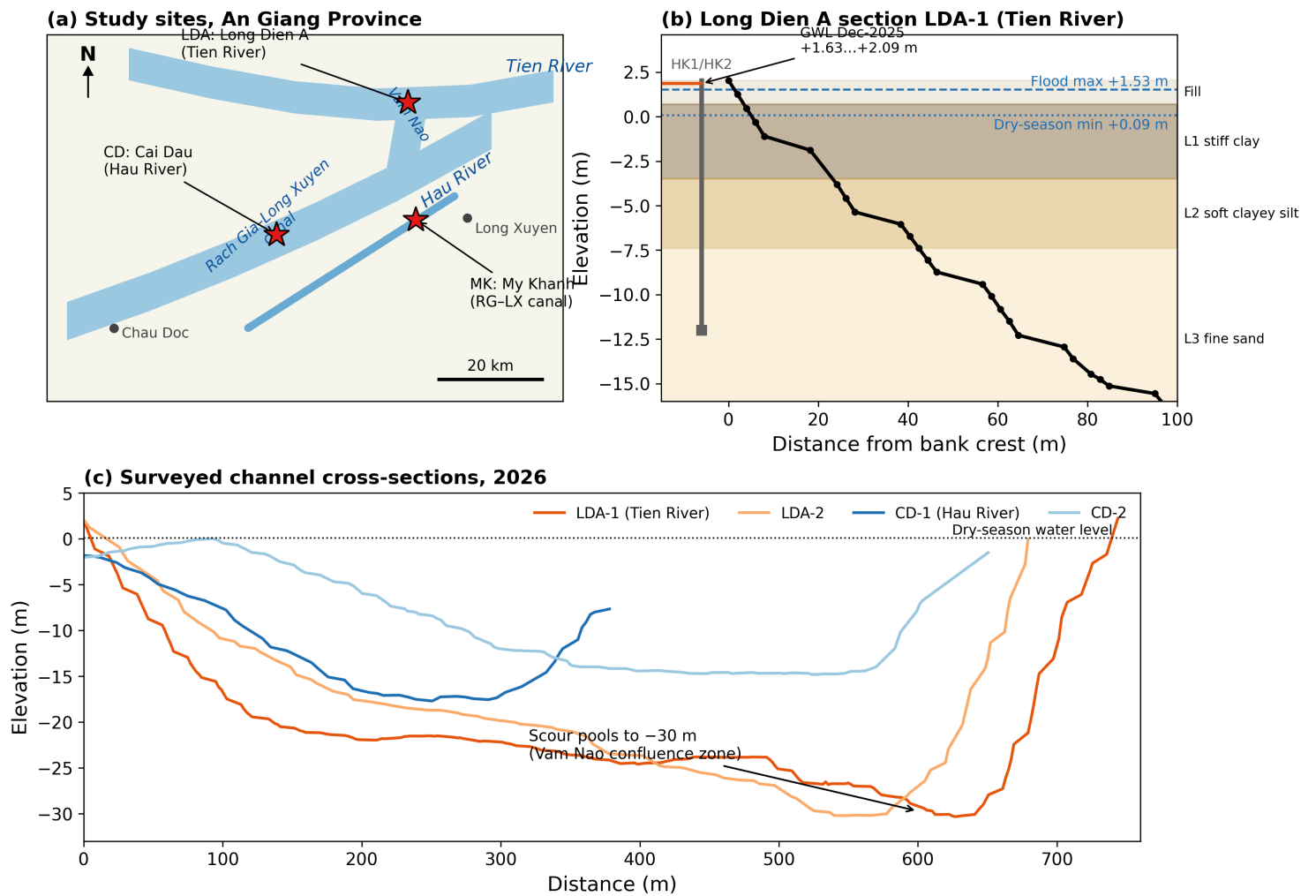


**Figure 1**

Study sites and geometry. (a) The three investigated sites in An Giang Province. (b) Surveyed near-bank profile LDA-1 with the borehole stratigraphy, the December 2025 groundwater level, and the 2016 stage extremes. (c) Full surveyed cross-sections of the Tien (LDA) and Hau (CD) channels, showing steep near-bank slopes and scour pools reaching -30 m

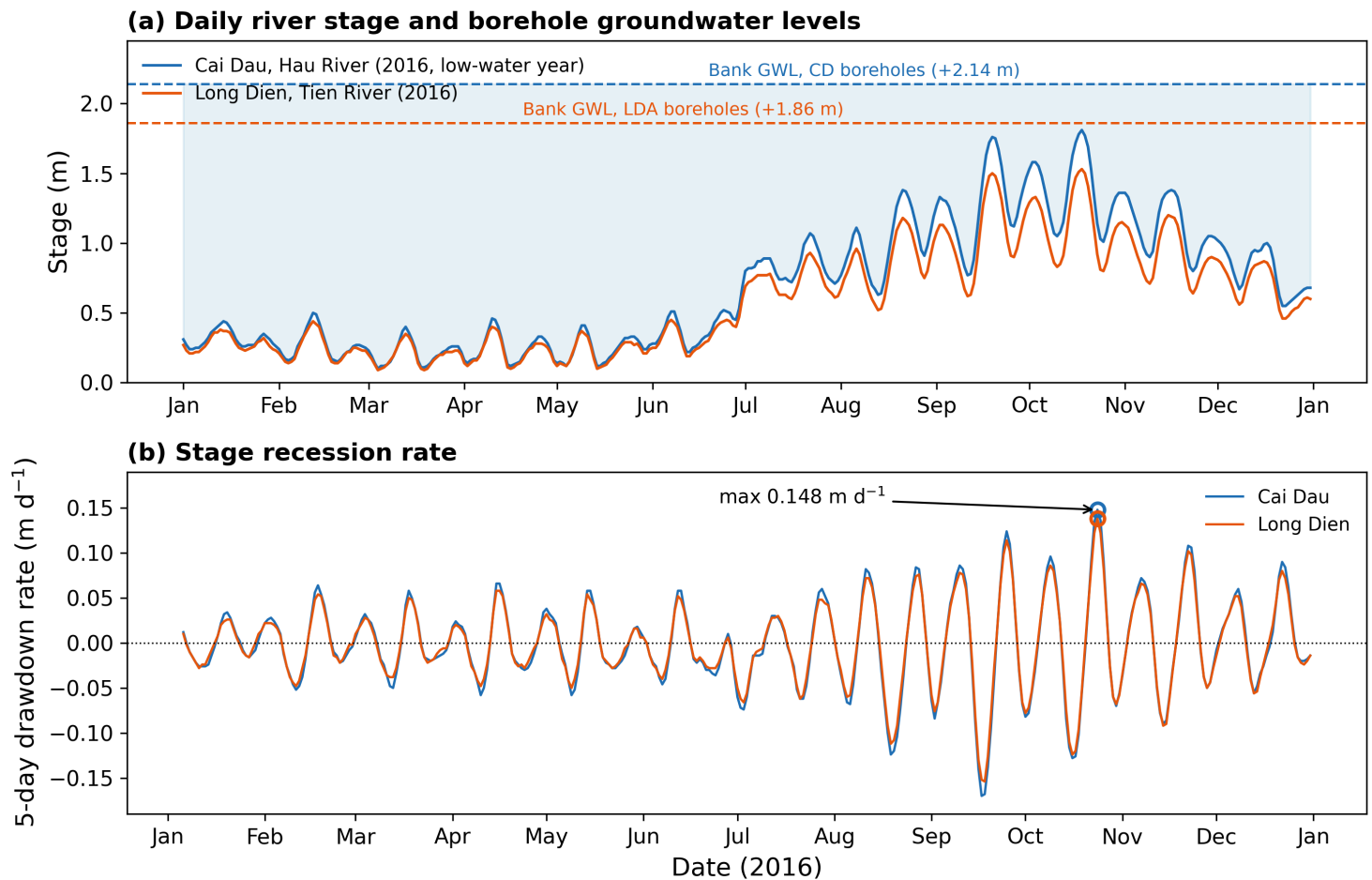


**Figure 2**

Hydrological forcing. (a) Daily stage at Cai Dau (Hau River) and Long Dien (Tien River) in the 2016 low-water year, with the groundwater levels measured in the December 2025 boreholes; the shaded band marks the persistent excess of bank groundwater over river stage. (b) Five-day recession rate; circles mark the annual maxima

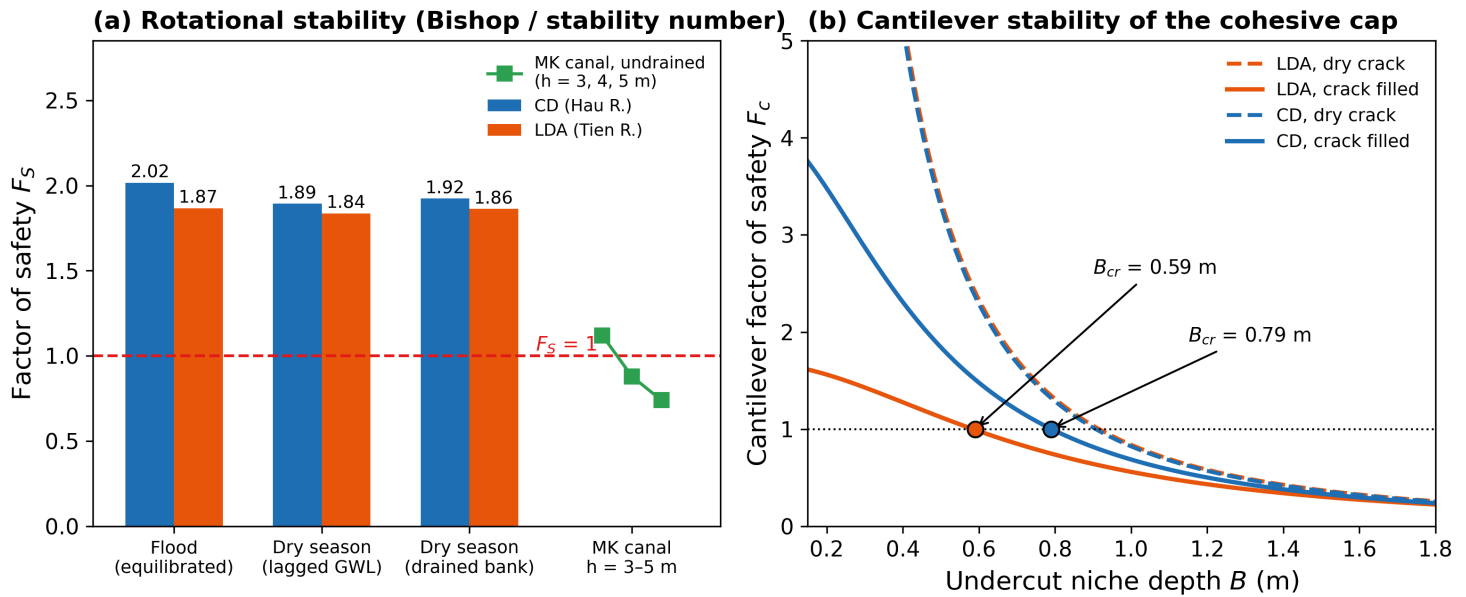


**Figure 3**

Deterministic stability. (a) Bishop factors of safety for the surveyed CD and LDA sections under three hydraulic scenarios, and undrained stability of the MK canal bank for heights of 3–5 m. (b) Cantilever factor of safety of the cohesive cap as a function of niche depth, for dry and water-filled tension cracks; dots mark the critical niche depths

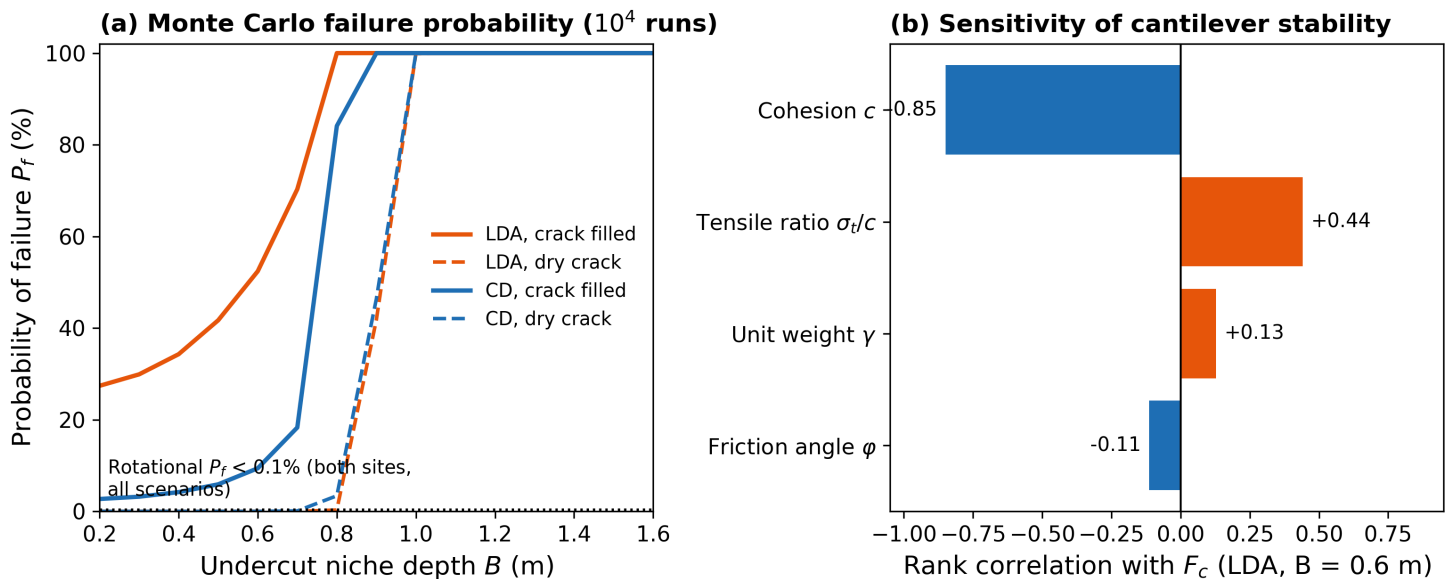


**Figure 4**

Probabilistic results. (a) Monte Carlo failure probability of the cantilever mechanism versus niche depth for dry and filled cracks at both river sites; rotational probabilities remain below 0.1 percent. (b) Rank

correlation of input parameters with the cantilever factor of safety at LDA for a 0.6 m niche and a filled crack