

Experimental study on mechanical behavior and working mechanism of unloading pile-sheet retaining walls

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Abstract: The unloading pile-sheet retaining wall is a new type of support/retaining for embankment slopes and has demonstrated excellent performance in engineering applications. However, its mechanical behavior and operational mechanisms are still not fully understood. This study conducted model tests on unloading piles and cantilever piles. Backfilling was conducted in four stages: 30 cm for the first three layers and 10 cm for the final one. The tests focused on the evolution of earth pressure, internal forces, and deformation in both pile types. The results show that: 1) Upon completion of backfilling, the horizontal displacement at the top of the cantilever pile reaches 81.76 mm, which is 5.45 times that of the unloading pile (14.99 mm). The maximum earth pressure on the unloading pile is 10.08 kPa, accounting for 66.40% of the 15.18 kPa recorded on the cantilever pile. The unloading effect alters the distribution pattern and magnitude of earth pressure. 2) The bending moment distributions differ significantly. The cantilever pile exhibits a “fish-belly” pattern with a maximum moment of 115.8 N · m. In contrast, the unloading pile shows an “S-shaped” profile, featuring a pronounced point of contraflexure at the unloading platform and a maximum negative moment of –60.99 N · m. 3) Incorporating an unloading platform effectively reduces earth pressure and enhances the anti-overturning moment. These effects jointly improve sliding resistance and overall structural stability. These findings offer theoretical insights and technical guidance for the practical implementation of unloading pile-sheet retaining walls.

Keywords: unloading pile-sheet retaining walls; model test; earth pressure; internal forces; pile deformation

1 Introduction

Retaining structures are extensively employed in geotechnical engineering for applications such as slope stabilization, foundation pit support, slope protection, and embankment construction [1–5]. Among them, the gravity retaining wall—one of the earliest types of retaining structures (see Fig. 1) suffers from several limitations, including large cross-sectional dimensions, limited retaining height, and dependence on natural stone materials [6]. With the widespread application of reinforced concrete materials and the advancement of geotechnical engineering technology, various lightweight retaining structures, such as cantilever walls, counterfort walls, reinforced soil walls, and unloading platform walls, have emerged, with their structural forms and application scope continuously expanding [7–12]. The unloading pile-sheet retaining wall is a novel retaining structure in which an unloading platform is arranged behind the wall, which redistributes the self-weight stress of the upper soil, thereby significantly reducing the earth pressure acting on the wall and improving the overall stability of the structure [13–18]. Owing to its good engineering performance, this type of structure has gradually attracted increasing attention and recognition from

geotechnical engineering communities both domestically and internationally [19–21].

As a critical structural component, the unloading platform significantly influences both earth pressure distribution and overall deformation behavior. Tan et al. [22] verified through field measurements and numerical analysis that pile-platform structures can significantly reduce lateral wall displacement and effectively redistribute earth pressure. Zhang et al. [23] conducted model tests on unloading pile-platform retaining walls, focusing on earth pressure, restressed pile resistance, bending moments, and structural deformation characteristics. Their results revealed that the unloading platform reduces both the earth pressure behind the wall and the bending moment of the piles. Liu et al. [24] conducted 1:7 scale model tests on counterweight-type pile-slab retaining walls, demonstrating that earth pressure beneath the unloading platform approaches zero, and further analyzed the force characteristics of the lower retaining wall and pile foundation. Hu et al. [25] divided the counterweight-type pile-platform retaining wall into a load-bearing section and a fixed section, simplifying the force analysis, and proposed calculation models and formulas for structural internal forces and deformations. Based on limit equilibrium theory and OptumG2 simulations,

Received: 16 June 2025

Accepted: 16 September 2025

This work was supported by the National Natural Science Foundation of China (52164001, 52568047).

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Que et al. [26] successfully derived a calculation formula for the active earth pressure of relief-shelf cantilever retaining walls (RS-CRW), a cantilever wall with an unloading platform. To optimize the unloading effect, researchers have systematically examined key parameters such as platform burial depth and width. Through numerical analysis, Moon et al. [27] identified the optimal platform position as 0.50 times the wall height, with a minimum width of 0.45 times the wall height. Zhang et al. [28] confirmed that the platform should be buried at a depth of 0.30–0.60 times the wall height, with a width of 0.38–0.47 times the wall height. Recent studies have also examined the influence of platform layout on sliding resistance, overturning resistance, and seismic stability. Chauhan et al. [29] reported that multi-platform high retaining walls can reduce lateral thrust by 11%–26% within a certain slope range. Using the earthquake sliding line method, Que [30] analyzed failure modes under seismic loading and found that the inclusion of an unloading platform substantially enhances structural stability. Aldonkar [31] and Yang [32] further confirmed its

effectiveness in improving sliding and overturning resistance. Himanshu et al. [33] evaluated cost efficiency and concluded that structures incorporating unloading platforms offer greater practicality and cost-effectiveness compared with traditional cantilever walls.

Although the structural advantages of unloading platforms have been confirmed, their stress-path transformation mechanisms and failure-mode evolution remain insufficiently and systematically understood. This study conducted comparative model tests on unloading piles and conventional cantilever piles under identical backfilling conditions, systematically analyzing the effects of the unloading platform on internal force redistribution, shear reversal, and displacement response. The results specifically clarify the role of the platform in reconstructing the stress path and in the zoned bearing mechanism of the structure. These findings help deepen the understanding of the working mechanism of unloading pile-platform structures and provide a theoretical basis for optimizing and improving structural design.



Fig.1 Typical retaining structures for high fill slopes

2 Experimental programs

2.1 Experimental setup

To investigate the mechanical behavior of unloading piles, a comparative model study of unloading and cantilever pile systems was conducted. The experiments were performed in a model box measuring $4\text{ m} \times 2\text{ m} \times 2\text{ m}$ ($L \times W \times H$). The box was constructed using a steel frame and transparent tempered glass panels to allow visual observation. Sand backfilling was carried out using the rainfall deposition method with a controllable hopper to ensure uniform compaction. Static strain gauges connected to a PC-based system

were used to monitor and record experimental parameters in real time.

2.2 Test materials

2.2.1 Anti-slip piles

As shown in Fig. 2, twelve square steel pipe piles were used to model the support structure. Each pile had a cross section of $50 \times 50\text{ mm}$, a wall thickness of 2 mm , and a length of 150 cm . The piles were positioned 30 cm above the base of the model box, with a clear spacing of 30 cm between adjacent piles-equivalent to six times the pile width. Given this spacing and the high structural stiffness, no significant interaction between adjacent piles was observed

during testing. An unloading plate measuring 180 cm in length, 45 cm in width, and 3 mm in thickness was installed to simulate the unloading platform. The unloading platform design was based on Moon et al. [27] and Zhang et al. [28]. The embedment depth and width were set to 0.5 and 0.45 times the wall height, respectively. To facilitate comparative testing, unloading

piles and cantilever piles were arranged symmetrically at opposite ends of the model box. To prevent sand leakage through the gaps between piles, a 2 cm thick impermeable rubber sheet was affixed to the rear of the pile row. This setup effectively simulated an inter-pile slab, ensuring proper soil compaction and improving the reliability of the experiment.

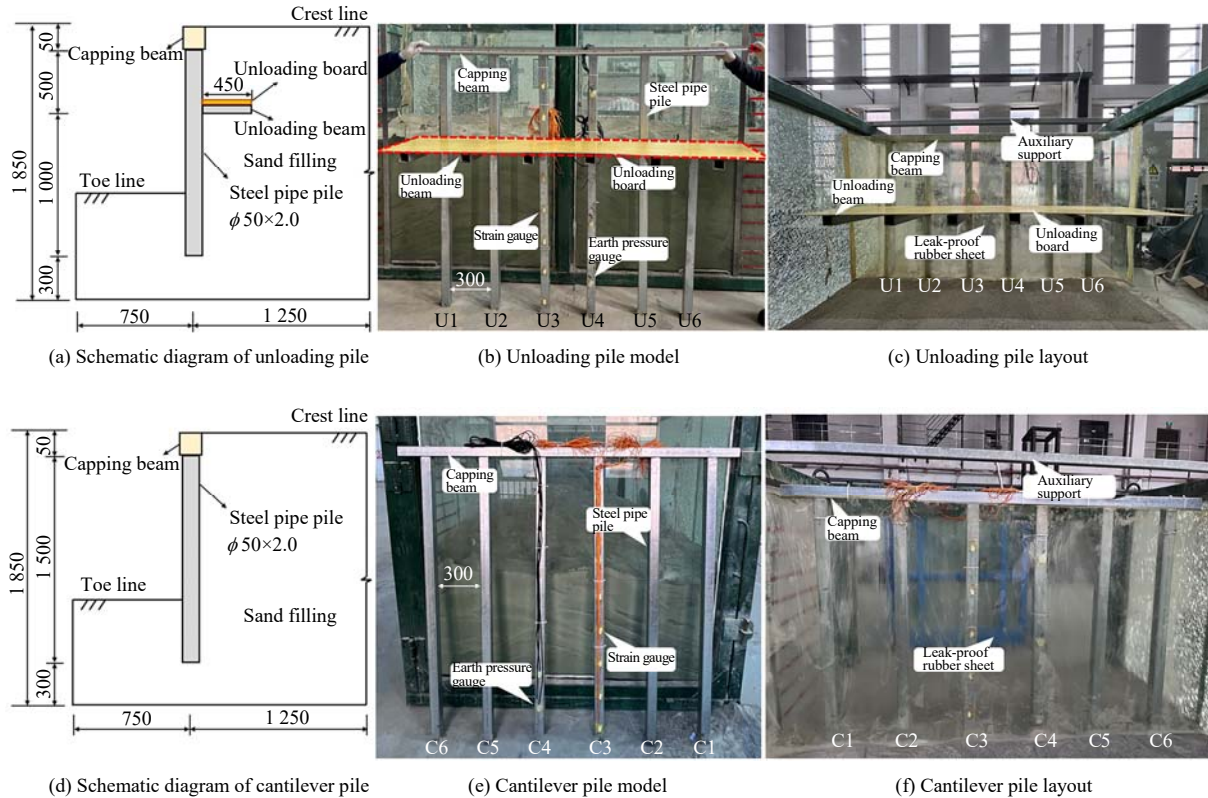


Fig.2 Anti-slip piles (unit: mm)

2.2.2 Capping beam

To ensure the stiffness and structural integrity of both the unloading and cantilever piles during testing, a square steel pipe (50 × 50 mm cross-section, 2 mm wall thickness) was used as a capping beam. The beam was securely bolted to the pile tops, forming an integrated load-bearing system that enabled effective force transfer. This configuration provided a stable structural basis for subsequent analysis of internal forces and deformation behavior.

2.3 Test soil

The test soil used in this study was dry sand sourced from a site in Guiyang, Guizhou Province, with a pre-filled volume of 14.4 m³. Prior to backfilling, representative soil samples were collected for laboratory analysis. The initial water content of the sand was 3.5%, and the dry density was 1.64 g/cm³. Shear strength parameters, including the internal friction angle ϕ and cohesion c , were determined using a ZJ-type strain-controlled direct shear apparatus. Physical parameters were obtained through conventional geotechnical testing methods, with specific values summarized in Table 1.

Table 1 Basic parameters of test sand

Specific gravity G_s	Mean particle size d_{50} /mm	Coefficient of uniformity C_u	Minimum void ratio e_{min}	Maximum void ratios e_{max}	Peak internal friction angle $\phi / (^\circ)$
2.71	0.281	2.40	0.796	0.564	38

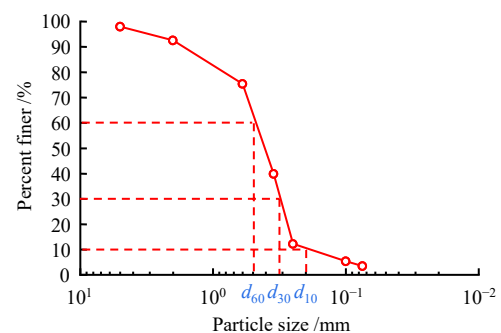


Fig.3 Grain size distribution of sand

The grain size distribution was determined by sieve analysis, using sieve sizes ranging from 0.075 mm to 5.0 mm. The effective particle diameter was $d_{10} = 0.20$ mm, the limiting particle diameter was $d_{60} = 0.48$ mm, and the coefficient of uniformity was

following Liu et al. [24]. The first three layers were each 30 cm thick, and the fourth layer was 10 cm, corresponding to cumulative filling heights of 30 cm, 60 cm, 90 cm, and 100 cm. The filling volumes were 1.5 m³ for each of the first three layers and 0.5 m³ for the fourth layer. Sand was placed using the rainfall deposition method: it was poured into a hopper and lifted by crane for placement. After each layer, the sand was compacted to ensure uniform density. During the experiment, all sensors were carefully protected to maintain stability and data accuracy. After each layer of filling, the system was left to stand for 2 h to allow the soil to settle, and data were then collected, until the final loading condition was reached. The experimental procedure is summarized in Table 2, and the overall view after backfilling is shown in Fig. 5.

Table 2 Staged backfilling process

Backfill stage /cm	Backfill thickness /cm	Backfill height /cm	Backfill volume /m ³	Rest time /h
1	30	30	1.5	2
2	30	60	1.5	2
3	30	90	1.5	2
4	10	100	0.5	2

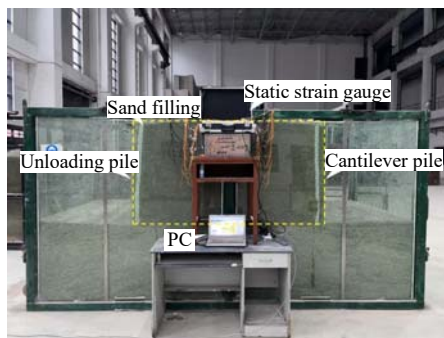


Fig.5 Overall view after backfilling completion

3 Deformation evolution characteristics

3.1 Evolution of pile top displacement with backfill depth

The readings from the dial gauges at the left, center, and right positions of the capping beam were averaged to represent the horizontal displacement of the pile tops. A curve showing the variation of pile-top horizontal displacement with backfill height was plotted, as shown in Fig. 6.

As shown in Fig. 6, the pile-top horizontal displacement of the cantilever pile u exhibits an exponential relationship with backfill height h , with the fitted curve $u = -3.16(1 - e^{0.03h})$ ($R^2 = 0.963$). As the backfill height increases, the displacement growth rate accelerates, reflecting the nonlinear accumulation of earth pressure. The pile-top horizontal displacement of the unloading pile u shows a linear relationship with backfill height h , described by $u = 1.17 + 0.15h$ ($R^2 = 0.939$), with a relatively stable displacement

growth rate. At a backfill height of 30 cm, the pile-top horizontal displacement of the unloading pile is 5.89 mm, with a growth rate of 1.96%, while that of the cantilever pile is 13.37 mm, with a growth rate of 4.46%. At 60 cm, the pile-top displacements increase to 12.56 mm and 21.22 mm, respectively, with growth rates of 2.22% and 2.62%. At 90 cm, the cantilever pile exhibits a significantly accelerated growth rate, with displacement reaching 47.13 mm, whereas the unloading pile only increases to 13.52 mm, remaining relatively stable. After completing backfilling, the cantilever pile reaches a maximum pile-top horizontal displacement of 81.76 mm, far higher than that of the unloading pile, which is 14.99 mm. The large displacement and rapid growth of the cantilever pile indicate poorer stability during loading, while the gradual displacement increase of the unloading pile demonstrates good deformation control and structural stability.

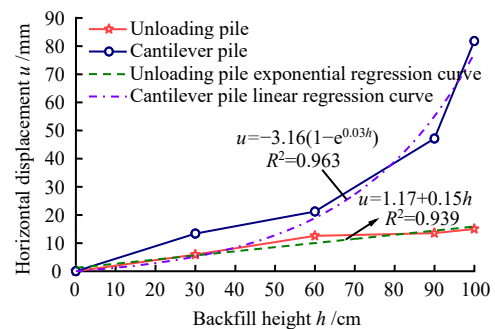


Fig.6 Variation curves of pile top horizontal displacement with backfill height

As shown in Fig. 7, after backfilling, cracks appeared in the soil on the cantilever pile side. The cracks were at a horizontal distance of approximately 35 cm from the pile top, ultimately forming a settlement failure zone about 51 cm from the pile top, accompanied by tilting deformation of the pile shaft. The measured rupture angle of the sand was 63.0°, slightly lower than the value predicted by Rankine theory (64.0°), with a small difference mainly caused by deviations between the theoretical model and the actual conditions. On the unloading pile side, no soil failure occurred, and the pile-top horizontal displacement was small, showing good deformation control ability.

3.2 Evolution of pile shaft deformation with backfill height

The displacement of the pile shaft can be obtained from the strain values measured at symmetrical sections of the pile. The rotation angle and deflection are calculated from the axial strain difference $\Delta\varepsilon$ between the two measuring points at each symmetrical section. The test pile is a square steel tube with a uniform cross-section. In the calculation, the material is assumed to be an isotropic linear elastic body, and

the effect of shear deformation is neglected. Assuming that the strain difference $\Delta\varepsilon$ between adjacent sections is linearly distributed along the pile shaft, the

pile is divided into 8 elements from bottom to top. The strain difference at a distance of 20 cm within each of the 8 elements is as follows.

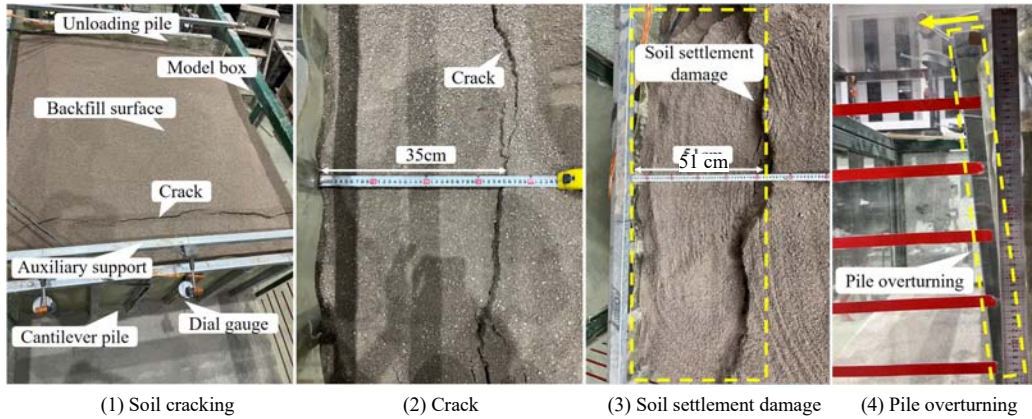


Fig.7 Cracking, settlement of the soil, and overturning of the cantilever pile

$$\Delta\varepsilon = \Delta\varepsilon_i + \frac{\Delta\varepsilon_{i+1} - \Delta\varepsilon_i}{l_i} x \quad (1)$$

where $\Delta\varepsilon_i$ and $\Delta\varepsilon_{i+1}$ are the bending strain values at sections i and $i+1$, respectively; l_i is the length of pile element i ; x is the distance from the i -th known point to the point to be calculated. The relationship between the lateral displacement y and the rotation angle θ of the pile shaft section is as follows.

$$\frac{d^2 y}{dx^2} = \frac{d\theta}{dx} = \frac{M}{EI} \quad (2)$$

where EI is the bending stiffness of the pile, E is the elastic modulus of the steel pipe pile; I is the moment of inertia of the pile cross section; and M is the bending moment of the pile shaft. From Equations (1) and (2), it follows that

$$\frac{d^2 y}{dx^2} = \frac{1}{b_0} \left(\Delta\varepsilon_i + \frac{\Delta\varepsilon_{i+1} - \Delta\varepsilon_i}{l_i} x \right) \quad (3)$$

where b_0 is the effective calculated width of the pile body.

A single integration of Equation (2) yields the rotation angle θ_{i+1} at section $i+1$, and a double integration of Equation (3) yields the deflection y_{i+1} at section $i+1$.

$$\left. \begin{aligned} \theta_{i+1} &= \theta_i + \frac{l_i}{2b_0} (\Delta\varepsilon_{i+1} + \Delta\varepsilon_i) \\ y_{i+1} &= y_i + \theta_i l_i + (\Delta\varepsilon_{i+1} + 2\Delta\varepsilon_i) \frac{l_i^2}{6b_0} \end{aligned} \right\} \quad (4)$$

The pile displacement curve can be obtained through double integration, as shown in Fig. 8.

As shown in Fig. 8(a), the horizontal displacement of the cantilever pile along the pile shaft decreases nonlinearly. When the backfill height reaches 30 cm and 60 cm, the rate of horizontal displacement increase is relatively stable, indicating that the earth

pressure distribution is relatively uniform and the pile deformation is gradual. When the backfill height reaches 90 cm, the pile deformation intensifies under increased earth pressure, and the pile-top horizontal displacement rises from 21.22 mm to 47.13 mm. After full backfilling, the displacement further increases to 81.76 mm, accompanied by settlement and cracking of the soil behind the pile, ultimately resulting in significant overturning failure of the pile shaft.

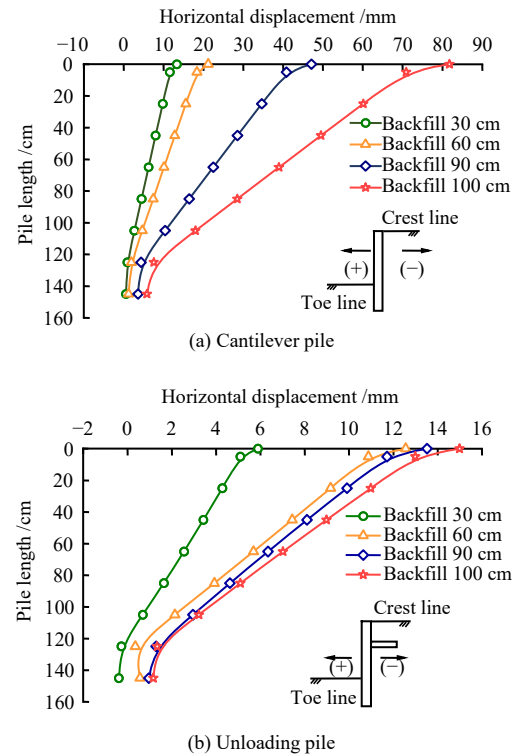


Fig.8 Horizontal displacement profiles of the pile shaft

As shown in Fig. 8(b), the horizontal displacement of the unloading pile along the pile shaft also decreases nonlinearly. When the backfill height reaches 60 cm, the pile-top horizontal displacement

increases from 5.89 mm to 12.56 mm, with a relatively stable growth rate. After full backfilling, the pile-top displacement increases only to 14.99 mm, and the growth rate decreases. This is due to the increased weight of the soil above the unloading platform, which provides a larger resisting overturning moment and effectively suppresses pile deformation. This characteristic allows the unloading pile to maintain relatively stable deformation under higher earth pressures or backfill heights, thus preventing overturning failure similar to that observed in the cantilever pile.

4 Earth pressure and internal force

4.1 Law of earth pressure variation

Based on the data collected from the earth pressure boxes installed on the retaining piles, earth pressure distribution curves at different depths along the pile shaft were plotted for both the cantilever and unloading piles, as shown in Fig. 9. As shown in Fig. 9, the measured active earth pressure behind the pile first increases and then decreases, whereas the Rankine active earth pressure curve shows a linear increase. The discrepancy between measured and theoretical values is mainly due to Rankine's earth pressure theory^[34] being based on idealized conditions, which cannot fully reflect the complex boundary conditions and soil behavior in real engineering. The maximum active earth pressure of the cantilever pile is 15.18 kPa at the bottom of the slope, while that of the unloading pile is 10.08 kPa, appearing 60 cm below the pile top, approximately 33.6% lower than the cantilever pile.

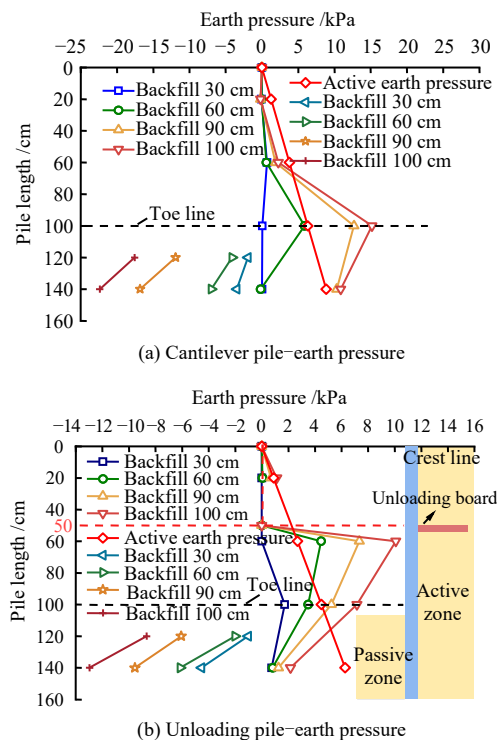


Fig.9 Earth pressure distribution curves

As shown in Fig. 9(b), at the position of the unloading platform 50 cm below the pile top, the earth pressure first drops to zero and is then redistributed along the pile shaft. The unloading platform effectively blocks the downward transmission of the self-weight stress of the upper soil, thereby reducing lateral earth pressure. The passive earth pressure of the unloading pile is only 58% of that of the cantilever pile, with values of -12.96 kPa and -22.38 kPa, respectively. Under backfill loading, the cantilever pile develops significant pile-top horizontal displacement, causing obvious pile tilting and severe compression of the soil in front of the pile, which leads to stress concentration and higher passive earth pressure. The unloading platform effectively suppresses pile-top horizontal displacement, and the soil in front of the pile does not experience obvious disturbance, resulting in smaller passive earth pressure.

The mechanisms by which the unloading pile optimizes the earth pressure distribution are mainly reflected in the following aspects: (1) The unloading platform bears and redistributes part of the active earth pressure, reducing the lateral earth pressure acting on the pile shaft. (2) The rigidity of the unloading platform limits the displacement of the soil behind the pile, slowing the development of active earth pressure and thereby improving overall structural stability. (3) The unloading platform weakens stress concentration and optimizes the earth pressure distribution, reducing pile deformation, enhancing bearing performance, and improving the structure's adaptability under long-term conditions.

To evaluate the accuracy of the theoretical model in predicting soil stress behavior, this study compares the measured earth pressure values in front of and behind the pile with the corresponding theoretical values at different backfill heights. Theoretical active and passive earth pressures were calculated based on Rankine's earth pressure theory^[34], with the active earth pressure coefficient $K_a = 0.24$, the passive earth pressure coefficient $K_p = 4.20$, and the at-rest earth pressure coefficient $K_0 = 0.52$. The theoretical results of active and passive earth pressures at various backfill heights are summarized in Table 3, providing a basis for analyzing the deviations between theoretical predictions and experimental measurements.

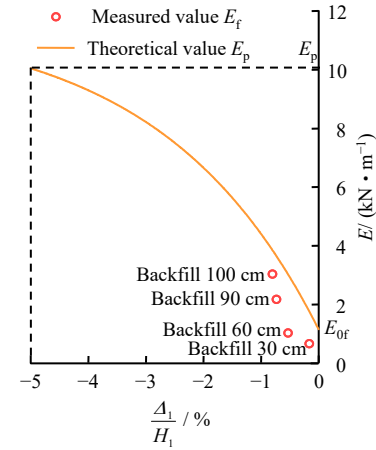
Table 3 Calculated earth pressure for unloading pile

Backfill height /cm	Soil pressure in front of pile /(kN · m ⁻¹)			Earth pressure behind pile /(kN · m ⁻¹)		
	Measured values E_f	Passive earth pressure E_p	At-rest earth pressure E_{of}	Measured values E_b	Active earth pressure E_a	At-rest earth pressure E_{ob}
30	0.66	9.88	1.22	0.84	1.45	1.75
60	1.02	9.88	1.22	2.68	2.62	3.58
90	2.17	9.88	1.22	4.26	3.96	6.04
100	3.03	9.88	1.22	5.93	4.50	7.01

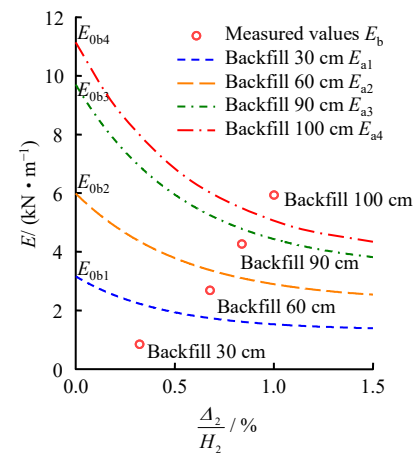
According to soil mechanics theory^[35], active earth pressure reaches the limit equilibrium state when wall displacement is approximately 0.1%–0.5% of the wall height, whereas passive earth pressure typically requires 1%–5% displacement.

As shown in Fig. 10, the displacement-earth pressure relationship of the unloading pile is plotted, where Δ_1 and Δ_2 represent horizontal displacements, and H_1 and H_2 represent wall heights. The displacement-earth pressure relationship of the unloading pile shows that, throughout the entire backfilling process, the horizontal displacement Δ_1 in the passive zone relative to wall height H_1 remains below 1%, indicating that the wall deformation is insufficient to fully mobilize passive earth pressure. At backfill heights of 30 cm and 60 cm, the measured earth pressure in front of the pile is below the at-rest pressure, and the soil remains in a low-stress state, with its resistance not yet fully mobilized. When the backfill height reaches 90 cm and 100 cm, the earth pressure in front of the pile gradually increases, falling between the at-rest and passive pressure levels. The soil in the passive zone is transitioning to a higher stress state and gradually approaching the limit, but the ultimate passive earth pressure is not yet fully mobilized. This is mainly due to the high stiffness of the pile and the constrained deformation under actual working conditions, which limit the full development of ultimate passive resistance. In the active zone behind the pile, the displacement-earth pressure relationship develops progressively with increasing backfill height. In the initial backfilling stage, the horizontal displacement Δ_2 of the unloading pile relative to wall height H_2 is less than 0.50%, insufficient to reach the activation threshold of active earth pressure, and the measured active earth pressure is relatively low. As the backfill height increases to 60 cm, 90 cm, and 100 cm, Δ_2/H_2 increases to 0.68%, 0.84%, and 1.00%, respectively, indicating that the soil behind the pile is gradually approaching an active failure state. As shown in Fig. 10, the measured active earth pressure gradually approaches the theoretical curve, aligning with theoretical predictions and confirming the reliability of the test results.

Despite general agreement, discrepancies between measured and theoretical earth pressures still exist, primarily due to the following reasons: (1) The high stiffness of the unloading pile limits overall deformation, making it difficult to generate sufficient displacement to mobilize ultimate earth pressure. (2) Insufficient compaction of the backfill soil results in loose contact between the soil and the structure, reducing interface resistance. (3) Friction effects between the soil and the structure cause nonlinear stress transfer, leading to measured values deviating from the predictions of Rankine's earth pressure theory.



(a) Soil pressure in front of pile



(b) Behind earth pressure of the pile

Fig.10 Relationship curves between earth pressure and deformation for the unloading pile

4.2 Law of bending moment variation

The distribution of the bending moment along the pile shaft is shown in Fig. 11. The bending moment at each section of the pile shaft is calculated from the tensile and compressive strain values on both sides of the pile shaft using the following formula.

$$M = EI \frac{\varepsilon_1 - \varepsilon_2}{c} \quad (5)$$

where M is the bending moment of the pile; ε_1 and ε_2 are the tensile and compressive strains of the pile, respectively; and c is the distance from the neutral axis of the section to the strain measuring point.

As shown in Fig. 11(a), the cantilever pile exhibits a pronounced "fish-belly" bending moment profile. The maximum bending moment reaches 115.8 N·m at approximately 105 cm from the pile top. As the backfill height increases, both the bending moment and pile deformation intensify, particularly in the mid-to-lower segments, where increased earth pressure leads to localized stress concentrations and raises the risk of localized instability and structural failure.

As shown in Fig. 11(b), the unloading pile exhibits an "S-shaped" bending moment distribution along the

pile shaft. At a backfill height of 30 cm, the maximum bending moment is 19.50 N·m, and after full backfilling it becomes -60.99 N·m, indicating a pronounced reverse bending phenomenon. This occurs because the upper segment of the pile is subjected to thrust from the soil behind the pile, while the lower segment is supported by passive earth pressure, causing a reversal in bending direction. As the backfill height increases, the point of contraflexure shifts upward from 65 cm to 45 cm and stabilizes once it exceeds the elevation of the unloading platform. From the perspective of the stress mechanism, when the backfill height is below 60 cm, the upper soil pressure is small, and the load primarily acts on the lower segment of the pile. As the backfill height increases, the upper soil pressure gradually increases, shifting the resultant force upward and modifying the bending moment distribution. Once the backfill height surpasses the unloading platform, the earth pressure distribution stabilizes, and the pile-soil system reaches a quasi-equilibrium stress state. During this process, the unloading platform plays a key role: on one hand, it increases the overturning resistance of the structure and effectively restrains deformation; on the other hand, it redistributes internal forces within the pile, alleviating stress concentration. The unloading platform effectively reduces the direct influence of backfill earth pressure on the pile and promotes force equilibrium, significantly enhancing the overall structural integrity and long-term stability.

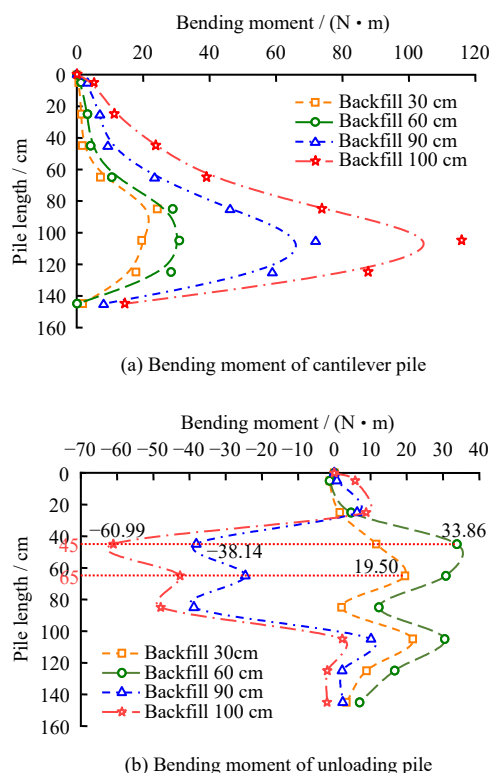


Fig.11 Variation curves of pile bending moment with backfill height

With increasing backfill height, the unloading pile gradually develops an "S-shaped" bending moment distribution, characterized by a reverse-bending region along the pile shaft where positive and negative moments alternate. The point of contraflexure ultimately stabilizes at the elevation of the unloading platform. This reverse-bending phenomenon indicates a more balanced internal force distribution, but it may also introduce certain structural risks: (1) Tensile cracking. In the reverse-bending region, the rear face of the pile changes from compression to tension. For reinforced concrete piles, especially when the backfill height exceeds the elevation of the unloading platform and the negative bending moment increases sharply, tensile cracking may occur. In engineering practice, crack-control measures such as increasing reinforcement ratio or adding stirrups should be implemented. (2) Local buckling. For hollow thin-walled steel pipe piles, the compressive stress on the rear face in the reverse-bending region increases, raising the risk of local buckling.

4.3 Axial force variation

The axial force of the pile shaft during the backfilling process is calculated using the following formula:

$$N = \frac{\varepsilon_1 + \varepsilon_2}{2} EA \quad (6)$$

where N is the axial force of the pile; and A is the cross-sectional area.

As shown in Fig. 12(a), the axial force of the cantilever pile along the pile shaft exhibits a typical "fish-belly" distribution, first increasing and then decreasing. The maximum axial force is 178.6 N, occurring 45 cm from the pile top. As the backfill height increases, the peak axial force gradually shifts upward, reflecting the enhanced contribution of shallow soil layers to the axial force and the progressive concentration of the axial load transfer region toward the upper section of the pile. During the test, the cantilever pile was primarily in a state of tension. Possible reasons include: insufficient soil consolidation, causing a rapid increase in soil pressure behind the pile over a short period and resulting in instantaneous tensile stress in the pile shaft; and weak friction at the pile-soil interface, which limits effective load transfer between the pile and surrounding soil, increases pile deformation, reduces structural stability, and heightens the risk of long-term instability.

As shown in Fig. 12(b), the axial force of the unloading pile also exhibits a "fish-belly" distribution, but the pile is predominantly in compression. The maximum axial force is -428.77 N, occurring 105 cm from the pile top. Compared with the cantilever pile, the unloading pile has a larger axial force, mainly due to the presence of the unloading platform. The

unloading platform not only supports the load from the upper soil but also effectively transfers it to the pile shaft, resulting in higher axial force within the pile. Below the unloading platform, the increase in axial force is more pronounced, reflecting the significant role of the platform in sharing the pile load. The axial force of the unloading pile is mainly concentrated in the deeper sections of the pile shaft, where the force distribution is more reasonable, contributing to improved bearing capacity and overall structural stability.

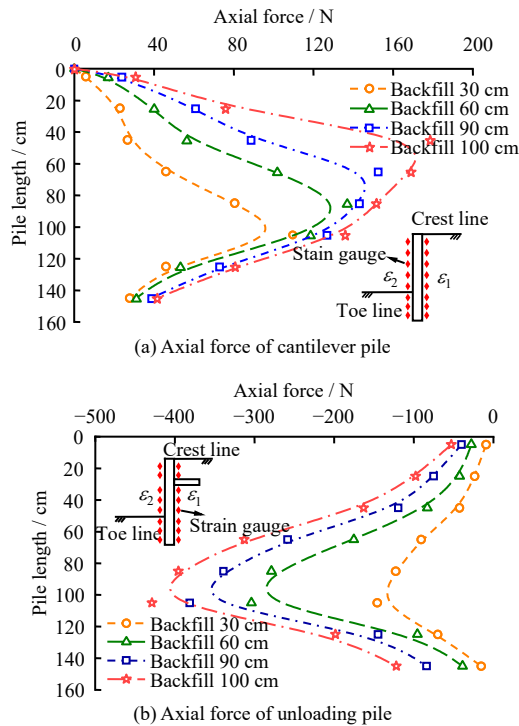


Fig.12 Variation curves of pile axial force with backfill height

In unloading piles, the pile shaft is simultaneously subjected to bending moment and axial force, and their interaction has important significance for internal force distribution and deformation control. Analysis of the bending moment and axial force distribution reveals the following mechanical characteristics: (1) Axial force contributes to bending resistance and deformation control. Test results show that the pile shaft below the unloading platform experiences high axial compression (maximum -428.77 N), which coincides with the region where the bending moment varies most significantly. Although the bending moment changes markedly, no obvious bending instability or structural failure occurs, indicating that axial force enhances the deformation control of the pile to a certain extent. (2) Bending moment affects the distribution of axial force and the load transfer path. Bending changed the stress state on the pile cross-section, thereby influencing the transmission of axial load. Above the unloading platform, the tensile stress generated behind the pile offsets part of the axial

compressive stress, making the cross-sectional stress distribution more balanced and alleviating compressive stress concentration. (3) The unloading platform causes the peaks of bending moment and axial force to be staggered, avoiding stress superposition. By altering the earth pressure transfer path, the bending moment and axial force peaks appear at different depths, reducing localized stress concentration and enhancing overall structural stability. This mechanical synergistic effect is an important mechanism for the improved mechanical performance of unloading piles.

4.4 Law of shear force variation

According to the theory of material mechanics, the shear force in the pile shaft can be obtained by differentiating the bending moment with respect to position.

$$Q = \frac{dM}{dz} \quad (7)$$

where Q is the shear force of the pile shaft, and z is the distance along the pile shaft.

As shown in Fig. 13(a), the shear force along the cantilever pile increases with backfill height, and the rate of increase becomes larger. The shape of the shear force curve and the positions of the extreme points remain essentially unchanged, but the shear force reaches a peak of 210.46 N at the base of the slope and then rapidly decreases to -366 N, reflecting a significant change in stress at the pile base. After backfilling is completed, the weight of the soil is concentrated on the pile base, causing pronounced shear force fluctuations. This stress characteristic indicates that, under high backfill conditions, the base of the cantilever pile may be subjected to large shear stress, increasing the risk of structural instability.

As shown in Fig. 13(b), the shear force along the unloading pile exhibits an "S-shaped" distribution and shows distinct dynamic adjustments as the backfill height increases. At a backfill height of 30 cm, the unloading platform has not yet contacted the fill, and the structure primarily relies on its own stiffness to bear the load. At this stage, the shear force extremum is -87.45 N, located 85 cm from the pile top. When the backfill height reaches 60 cm, with approximately 10 cm of soil above the platform, the unloading platform begins to share part of the load, and the shear force extremum shifts upward to the platform location. As backfilling continues, the mechanical effect of the unloading platform is further activated, resulting in a significant reversal of shear at the platform, with extremum values increasing to -222.15 N and -348.42 N. The shear reversal at the platform indicates a redistribution of internal forces within the pile and a change in the load transfer mechanism. During the initial loading stage, as earth pressure increases, the shear force gradually rises from the pile top to the base. Once the unloading platform begins to share the

upper load, the original internal force path is interrupted, and the shear direction reverses. This phenomenon reflects that the pile's mechanical behavior gradually transitions from being bending dominated to a combined compression-bending action. Structurally, the shear reversal point divides the pile into two functional zones: the upper zone is primarily governed by bending, and the lower zone is dominated by axial force. This internal force redistribution optimizes the pile's stress state and improves the overall stability of the unloading pile. Under high backfill conditions, the unloading platform plays a key role in adjusting the mechanical response by redistributing the load, reducing shear force and bending moment along the pile shaft, and significantly enhancing the structural stability of the system.

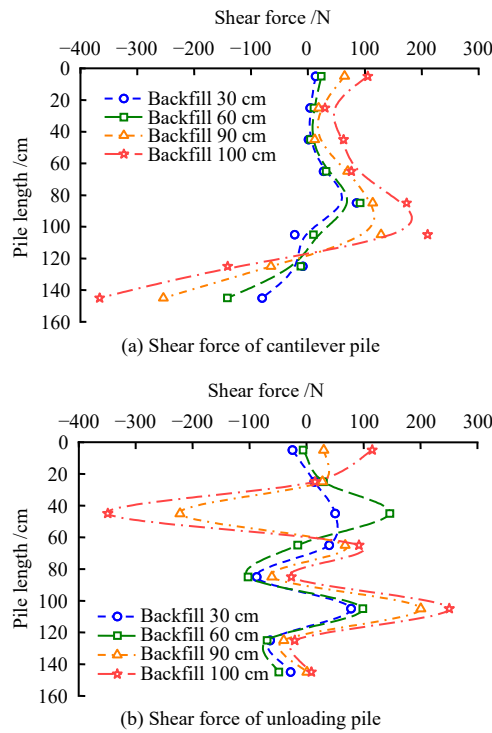


Fig.13 Variation curves of pile shear force with backfill height

4.5 Mechanism analysis

As shown in Fig. 14, the stability of the unloading pile-sheet retaining wall primarily depends on the mechanical action of the unloading platform, which is reflected in two main aspects: (1) Providing an overturning-resistance moment M_2 to enhance structural stability. The unloading platform supports the self-weight of the upper fill soil, generating a counteracting bending moment opposite to the bending moment induced by active earth pressure on the pile shaft. This effectively offsets part of the positive bending moment caused by active earth pressure, thereby reducing the stress on the pile shaft. The counteracting bending moment M_2 , together with the passive earth pressure bending moment M_4

generated by the soil in front of the pile, jointly resists the overturning moments M_1 and M_3 caused by active earth pressure and the weight of the fill, thus improving overall overturning resistance. (2) Reducing active earth pressure and optimizing the load transfer path. The unloading platform bears part of the backfill load, lowering the lateral earth pressure behind the pile and optimizing the load transfer path. In particular, in the h_2 section below the platform, it significantly reduces the stress generated by the self-weight of the soil. Under fill slope conditions, this unloading mechanism enhances the adaptability and overall stability of the structure, effectively reducing the risk of structural instability. In Fig. 14, H and H_r represent the slope height and embedded depth, respectively; E_{a1} and E_{a2} represent the active earth pressure above and below the unloading platform; z_a and z_p represent the locations of active and passive earth pressure, respectively; q is the self-weight of the soil above the platform; γ is the unit weight of the soil; b is the length of the unloading platform; h_1 and h_2 are the distances from the unloading platform to the pile top and to the slope bottom, respectively.

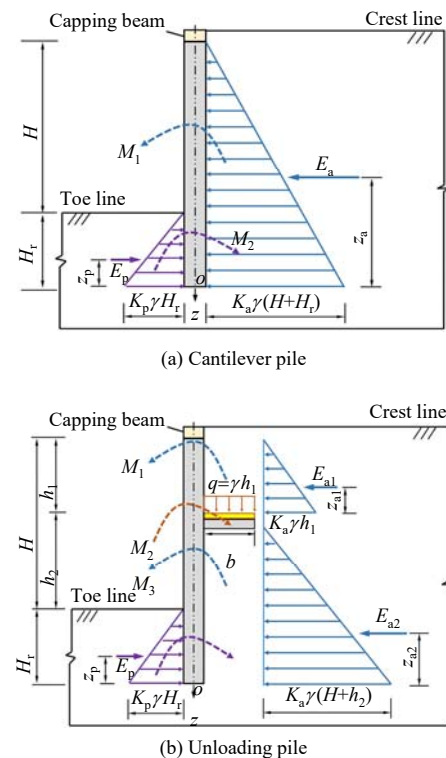


Fig.14 Schematic diagrams of anti-slip pile force

According to the Technical Code for Slope Engineering in Building Construction (GB50330-2013)^[36], the sliding stability factor F_s and the overturning stability factor F_t of cantilever piles are calculated using the following equations.

$$F_s = \frac{E_p + \mu G}{E_a} \quad (8)$$

$$F_t = \frac{E_p z_p}{E_a z_a} \quad (9)$$

where μ is the base friction coefficient, taken as 0.3; and G is the self-weight of the retaining wall (kN/m).

The sliding stability factor F_s and the overturning stability factor F_t of the unloading pile are calculated using the following equations:

$$F_s = \frac{E_p + \mu(G + qb)}{E_a} \quad (10)$$

$$F_t = \frac{E_p z_p + qb^2 / 2}{E_a z_a} \quad (11)$$

The calculated results at different backfill heights are plotted in Figs. 15 and 16 for comparison and analysis.

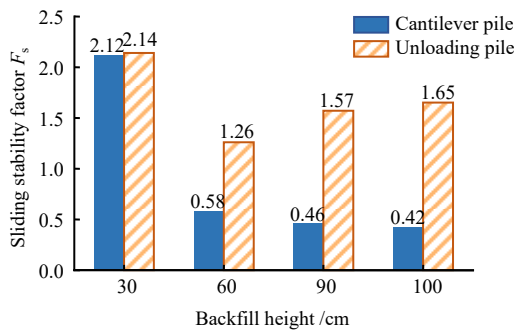


Fig.15 Variation of sliding stability factor F_s with backfill height

As shown in Fig. 15, the sliding stability factor F_s of the cantilever pile decreases significantly as the backfill height increases. When the backfill height reaches 60 cm, F_s drops below 0.60, indicating that under high backfill conditions, the sliding resistance of the cantilever pile is insufficient and there is a substantial risk to structural stability. Under the same conditions, the unloading pile exhibits superior sliding performance. Its sliding stability factor F_s shows a non-monotonic variation with backfill height, initially decreasing and then increasing, and always remains above the standard value of 1.25 specified in design codes, indicating adequate sliding resistance and good overall stability. The improvement of sliding resistance in the unloading pile is primarily attributed to the mechanical adjustment mechanism of the unloading platform, as reflected in the following three aspects: (1) Activation of mechanical effects. During the early stage of backfilling, when the fill height is below the unloading platform, the platform has not yet taken effect, and the active earth pressure behind the pile dominates the sliding tendency, causing F_s to decrease. When the backfill height reaches the platform elevation, the platform begins to share the load of the upper fill, effectively reducing the self-weight stress of the soil behind the pile, lowering the sliding force, and thereby enhancing sliding

resistance. (2) Redistribution of earth pressure. Once the unloading platform is activated, the earth pressure distribution behind the pile is adjusted. Part of the active zone is transformed into a passive zone, and the sliding failure surface moves forward or deepens, enhancing the structure's ability to resist sliding. (3) Progressive reinforcement effect. In the later stages of backfilling, the soil above the platform gradually consolidates and densifies, increasing the internal friction angle, which further improves the sliding resistance of the structure.

As shown in Fig. 16, the overturning stability factor F_t of the cantilever pile decreases significantly with increasing backfill height, falling below 0.20 when the backfill height reaches 60 cm. As the self-weight of the soil behind the pile increases, the overturning moment continues to grow. However, the cantilever pile's overturning resistance mainly depends on its stiffness and the depth of the embedded section, limiting its capacity to counteract the overturning moment induced by the combined effects of fill weight and lateral earth pressure, resulting in poor structural adaptability. In contrast, the overturning stability factor F_t of the unloading pile increases with backfill height, demonstrating superior anti-overturning performance. The mechanical advantages of the unloading pile can be analyzed in three stages: (1) Early backfilling stage. When the fill height is below the unloading platform, the platform has not yet taken effect, and the structure's overturning resistance mainly relies on the pile's stiffness. At this stage, $F_t = 1.21$, which is below the design standard of 1.60, indicating that the structure still has some risk of overturning, with the earth pressure behind the pile being the main driving force. (2) Activation stage of the unloading platform. Once the platform begins to bear the load of the upper fill, the lateral earth pressure behind the pile is significantly reduced, and the overturning resistance is noticeably enhanced. The platform, acting together with the pile, generates a counteracting bending moment, effectively increasing the structure's overturning resistance and substantially improving overall stability. (3) Backfilling completion stage. At this stage, the unloading pile shows far superior overturning resistance compared to the cantilever pile.

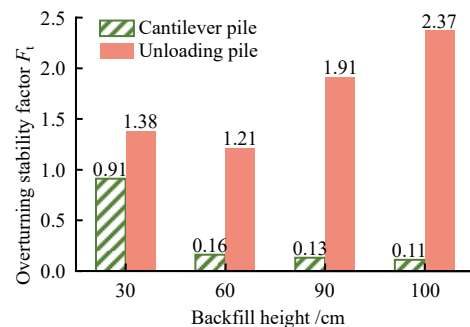


Fig.16 Variation of overturning stability factor F_t with backfill height

The F_t of the cantilever pile drops to 0.11, well below the design standard of 1.60, indicating a severe risk of overturning failure. In contrast, the F_t of the unloading pile reaches 2.37, far exceeding the requirement of the Technical Code for Slope Engineering in Building Construction^[36], demonstrating strong overturning stability and full adaptability under complex, high-fill conditions.

4.6 Discussion

This study was conducted through laboratory model tests to comparatively analyze the mechanical behavior and structural performance of cantilever and unloading piles during backfilling. However, the experiments have certain limitations that should be addressed in future research:

(1) Due to the boundary conditions of the test setup and the limited soil volume, the gravitational effects were not fully simulated. Future studies could reduce the similarity ratio to improve test accuracy and the reliability of results.

(2) The sand used in this study is homogeneous and does not fully reflect the layered or cohesive soils commonly encountered in engineering practice. To increase the engineering applicability of the findings, future research should consider incorporating a wider range of geological conditions.

(3) Steel pipe piles were employed in the tests. Their high stiffness and low pile-soil interface friction make it difficult to realistically reproduce the nonlinear behavior of actual concrete piles and the characteristics of pile-soil interaction. Future studies could consider using concrete-like or composite model piles to better simulate the mechanical response of prototype structures.

(4) This study focuses on comparative verification of structural forms rather than precise estimation of engineering parameters. Therefore, the scale design did not strictly follow similarity theory, which constitutes a methodological limitation. Future research will combine centrifuge model tests and numerical simulations to systematically and rigorously analyze similarity effects.

5 Conclusions

Based on comparative model tests of unloading piles and cantilever piles, this study systematically analyzes the evolution of earth pressure, internal forces, and deformation characteristics of retaining structures during backfilling. The main conclusions are as follows:

(1) After backfilling, the horizontal displacement at the top of the cantilever pile reaches 81.76 mm, whereas it is only 14.99 mm for the unloading pile. On the cantilever pile side, settlement failure occurs approximately 51 cm from the pile top, accompanied by overturning deformation, indicating a structural instability risk under high-fill conditions. The unloading platform effectively restrains deformation, significantly

improving the overall stability of the retaining structure.

(2) The active earth pressure of both cantilever and unloading piles initially increases and then decreases. The cantilever pile reaches a maximum active earth pressure of 15.18 kPa at two-thirds of the pile height, while the unloading pile's maximum is 10.08 kPa, about 33.6% lower than that of the cantilever pile, occurring 60 cm from the pile top. At the unloading platform, the earth pressure drops to zero, effectively blocking the transmission of the upper soil's self-weight stress and reducing the load on the pile.

(3) The cantilever pile's bending moment exhibits a typical "fish-belly" distribution, with a maximum of 115.8 N·m located 105 cm from the pile top, indicating stress concentration in the middle and lower sections. In contrast, the unloading pile exhibits an "S-shaped" bending moment distribution with pronounced reverse bending, and the inflection point ultimately stabilizes at the unloading platform. After backfilling, the anti-overturning moment at the platform is -60.99 N·m, effectively counteracting part of the positive bending moment, reducing internal force response, and limiting overturning deformation. The interaction between bending moment and axial force, optimized by the unloading platform, improves deformation control, prevents stress superposition, and enhances overall structural stability.

(4) The unloading pile-sheet retaining wall, through the incorporation of the unloading platform, increases overturning resistance and reduces earth pressure, thereby improving sliding resistance and overall stability. This effect is particularly significant under high-fill conditions, demonstrating superior engineering adaptability.

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