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Experimental study of the progressive collapse mechanism of excavations retained by cantilever piles

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ABSTRACT:

An increasing number of catastrophic progressive collapses of deep excavations have occurred throughout the world. However, the research on progressive collapse mechanisms is limited. In this paper, two categories of model tests were conducted to investigate the mechanism of partial collapse (sudden failures of certain retaining piles) and progressive collapse, respectively. The model test results show that partial collapse can cause a sudden increase in the bending moments of adjacent piles via an arching effect. The load transfer coefficients are defined to be equal to the peak increase ratios of the maximum bending moments in adjacent piles (peak moments caused by collapse over the values before the collapse). When the maximum load transfer coefficient is larger than the bearing capacity safety factor of the piles, the partial failure will lead to progressive collapse. The influential factors of the progressive collapse mechanism, such as the partial collapse extent, excavation depth and capping beam, were also investigated. During progressive collapse, the previous failed pile could cause new stress arching; simultaneously, the soil behind certain nearest intact piles could become loosened and destroy the arch springing of the stress arching, causing the progressive collapse to cease gradually.

KEYWORDS: deep excavation; progressive collapse; arching effect; retaining piles; trapdoor
INTRODUCTION

The design of retaining structures of excavations generally proceeds element-by-element (Zheng et al. 2011). Therefore, retaining systems always lack integrity and sufficient redundancy. Consequently, a partial failure of a retaining structure can easily evolve into a large-scale collapse, which can be referred to as “progressive collapse” and is similar to the progressive collapse problem in structural engineering: one or several structural members suddenly fail and then the building collapses progressively until the complete failure of the building or a major part of it (Menchel et al. 2009).

Many large-scale excavation collapses have been triggered by a partial failure. For instance, the accident records of the Singapore metro station collapse indicate that the collapse proceeded with a domino-like effect (Davies 2007; COI 2005; Artola 2005; Whittle and Davies 2006; Puzrin et al. 2010; Corral and Whittle 2014; Ishihara and Lee 2008). Additionally, progressive collapse along the length of the excavation is especially serious. The excavation collapses of the metro stations in Cologne (Haack 2009), Hangzhou (Li et al. 2010; Chen et al. 2013) and Singapore developed to be approximately 50 m, 70 m and 100 m in length, respectively.

The progressive collapse problem in structural engineering has been investigated substantially (Dusenberry 2002; Masoero et al. 2009; Starossek 2006; Marjanishvili 2004; Frangopol and Curley 1987; Dusenberry 2002; GSA 2003; National Research Council of Canada 2003, BS8110-1; EN1991-1-7; Menchel et al. 2009; Murtha-Smith 1988). However, the occurrence frequency and consequences of excavation collapses could be even higher than those in structural engineering. Therefore, it is urgent to establish a related theory and design methodology for resisting the progressive collapse of deep excavations.
The progressive collapse mechanism of an excavation involves large deformation and nonlinearity and is a three-dimensional problem. Conventional analysis methods (Reul 2006; Bjerrum and Eide 1956; Chen 1975; Chang 2000) cannot be applied directly to analyze it. This paper investigates the progressive collapses along the length using model tests. Four partial collapse tests (sudden initial failures of certain retaining piles) were conducted to examine the influence of partial collapse on an adjacent retaining structure. Furthermore, a progressive collapse test, in which additional and further failures occurred due to partial collapse, was conducted. Based on the model test results, the occurrence and evolution mechanism of the progressive collapse in excavations were discussed.

MODEL TEST INTRODUCTION

Soil tank and devices

A soil tank with dimensions of 2.50 m × 2.46 m × 1.40 m (length × width × height) was constructed to conduct the model tests, as shown in Fig. 1. Teflon (PTFE) film was attached to the internal wall surfaces of the tank (except the windows) to reduce any friction between the soil and the sidewalls. As shown in Fig. 1 (a), the sand raining equipment was installed above the soil tank.

Similitude relationship

Altaee and Fellenius (1994) and Gibson (1997) proposed a scaling law for small-scale testing under the 1-g condition in sand. They noted that the chief condition for agreement between the model and prototype is that the initial soil states of both must have equal proximity to the steady-state line, as shown in Fig. 2. The scaling relations, in terms of the geometric scaling ratio $n$ and stress scaling ratio $N$ (ratios of
prototype/model), presented by them are presented in Table 1. Because the density difference between the model and prototype is small and can be ignored (Gibson 1997, Scott 1989), the stress scaling ratio \( N \) approximately equals to the geometric scaling ratio \( n \). The geometric scaling ratio \( n \) was set at 16 for all of the models in the tests. The corresponding scaling relations under this condition are also listed in Table 1.

**Soil in the model test**

The soil used in this model test was dry fine sand. The main parameters of the sand are listed in Table 2. The critical state friction angle of the sand sample at the middle height of the model, measured by the direct shear test, was 30.96°. Because the height of the sandbox is fixed, the sands at different depths in the soil tank have different falling heights and void ratios, as shown in Fig. 3.

A series of consolidation undrained (CU) triaxial tests was performed to estimate the steady-state behavior of the sand (Gibson 1997). As illustrated in Fig. 4, the slope of the steady-state line (SSL) \( \lambda \) in the \( e \)-\( \ln p \) plane is -0.0389. Based on the scaling relation \( e_m = e_p + \lambda \ln(1/n) \), the void ratio in the prototype can be determined, which varies from 0.65 to 0.51 and lies within the range defined by the maximum and minimum void ratios (0.85-0.43) of the sand, indicating that the void ratios are reasonable.

In this study, the size of the sand grains is not scaled by the geometric scaling ratio \( n \), which might have an influence on the model response (Sedran et al. 2001). According to the catalogue of scaling laws provided by the Technical Committee TC2 (Physical Modelling in Geotechnics) of the ISSMGE (Garnier et al. 2007), the grain size would not significantly influence the response of model piles when the ratio of the pile diameter to the mean grain size \((B/D_{50})\) was larger than 45 (Nunez et al. 1988) or 60.
(Remaud 1999). In the performed tests, a relatively fine sand with a $B/D_{50}$ (60/0.23) of 261 was chosen so that the grain-size effect was deemed to be negligible.

**Model piles and capping beam**

The model pile was composed of a rectangular PVC tube with a cross-section of 60 mm $\times$ 40 mm $\times$ 3 mm (length $\times$ width $\times$ wall thickness). The length of the model pile was 1.2 m. As shown in Fig. 1 (b), 40 total contiguous piles were placed along the width of the soil tank (2.46 m) to serve as the retaining structure.

To conduct the model tests of the progressive collapse, several types of piles with different functions were designed, including the intact pile (IP), initiating-failure pile (IFP), and following-failure pile (FFP). Most of the intact piles were composed of the original PVC tube without a significant amount of extra processing. Certain intact piles and following-failure piles were monitored during the tests and will be called monitoring piles (MPs) in the following sections.

(1) Monitoring pile (MP)

For each test, 8 to 10 monitoring piles were used among 40 retaining piles. To monitor the bending moment of certain intact piles, 8 monitoring sections were set in a pile, as shown in Fig. 1 (a). For each monitoring section, two strain gauges were installed on the inner walls of the compression and tension sides.

Through the loading test on the simply supported MP, the flexural stiffness $EI$ of the pile was measured to be 560 N·m$^2$; the strain value of the strain gauge was 29 µ when the pile section was subjected to the unit pure bending moment (1 N·m). Based on the similitude relationship in Table 1, the prototype retaining piles (made of C30 reinforced concrete) were approximately 0.8 m in diameter and 1.0 m in spacing, values that are
frequently used in practical engineering. Other geometric and physical parameters of the model and prototype excavations are presented in Table 3.

Horizontal displacements of the pile heads were measured using dial indicators. The earth pressures acting on the pile were monitored by earth pressure cells installed on the active zone side of the monitoring pile at a burial depth of 40 cm, as shown in Fig. 1 (a). The strain and earth pressure were collected by a high-speed data acquisition system with a sampling frequency of 100 Hz.

(2) Initiating-failure pile (IFP)

To simulate the partial failure of the retaining structure, certain piles will remain intact in the excavation stage and fail to initiate the partial collapse when they receive the control signal. Such a pile is called an initiating-failure pile (IFP) in this paper, and the working mechanism is shown in Fig. 5.

(3) Following-failure pile (FFP)

The maximum bending moment of the retaining piles in the model tests (approximately 30 N•m) is much lower than the ultimate bending strength of an intact pile (147 N•m). Therefore, the intact pile cannot fail under the condition of partial collapse. To simulate the failure of the neighboring piles caused by the partial collapse (progressive collapse), the following-failure pile (FFP) was designed; the working mechanism is illustrated in Fig. 6. During the excavation, the thin steel wire rope (0.6 mm in diameter) is subjected to the tensile force. Therefore, the ultimate bending strength of the FFP is determined by the tensile strength of the steel wire rope, which is 208.75 N. Through loading tests, the ultimate bending strength of the FFP was measured to be 8.70 N•m.

(4) Capping beam
The capping beam was modeled using the same PVC tube as for the piles; the width and height of the cross-section of the capping beam were 60 mm and 40 mm, respectively. In addition to 4 IFPs, the heads of 36 IPs were fixed to the capping beam, as shown in Fig. 7.

**Information regarding the 5 model tests and test procedures**

The model information for 5 tests, including the arrangement of piles, instrument locations and excavation depths, is illustrated in detail in Fig. 7. Test 1–Test 4 were model tests of partial collapses caused by failures of certain retaining piles. Tests 2, 3 and 4 were conducted to investigate the influence of the partial failure extent, excavation depth and capping beam, respectively, on the progressive collapse mechanism. Based on the results of partial collapse tests, a model test (Test 5), in which the progressive collapse induced by a partial collapse occurred, was conducted by using the IFPs and FFPs.

For Tests 1, 2, 3 and 5, the piles were separate and no capping beam was installed. The boundary of window 1 was considered to be a symmetrical plane, as shown in Fig. 7. In Test 4, the retaining piles were connected by a continuous capping beam. Because the failure of the capping beam was not considered in this study, the partial collapse (IFPs) was arranged in the middle of the retaining piles. Four IFPs were used in Test 4, which was equivalent to 2 IFPs in Test 1. The cross-section in the middle of the 4 IFPs can be considered a symmetrical plane of this model.

Except for Test 3, the total excavation depth in other tests was 60 cm and each excavation step was 10 cm. For Test 3, the excavation depth was 75 cm. When the excavation depth was smaller than 60 cm, each excavation step was 10 cm; after that,
the excavation step became 5 cm. In all tests, after the excavation, the rupture of the IFPs was controlled to initiate the partial collapse of the excavation.

RESULTS OF THE MODEL TESTS IN THE EXCAVATION STAGE

To easily understand the model test results and associate them with practical engineering, the results discussed in the following sections were all converted to prototype scale based on the scaling relationships in Table 1.

Displacement of the pile top

In Test 1, when the excavation was complete (9.6 m), the horizontal displacements of the monitoring piles ranged from 183.20 to 193.92 mm. The difference between the maximum and minimum displacements was 5.5%, indicating that the boundary effect of this model test was not obvious.

During the excavation stage, the average horizontal displacements of the pile heads increased with increasing excavation depths, as shown in Fig. 8. From Test 1 to Test 4, the displacement curves were similar when the excavation depth was smaller than 9.6 m, which indicates that the repeatability of this series of tests was good. For Test 3, when the excavation depth was greater than 9.6 m, the horizontal displacement increased rapidly with the excavation depth. In Test 5, most of the retaining piles were FFPs whose bending stiffness at the location of the preset rupture plane was lower than that of the intact piles. Therefore, the displacements of Test 5 were slightly larger than those of other tests.

Bending moment of the pile

Fig. 9 (a)–(c) shows the average bending moment curves of the monitoring piles at different excavation depths for Test 1 to Test 3; the results in Test 4 and Test 5 were
similar to these curves. Based on the bending moment curves of the 5 tests, when the excavation depth was 9.6 m, the average value of the bending moments at the elevation of the preset rupture plane (9.6 m below the ground surface) in Test 1–Test 5 was 439.10 kN•m, which is considered to be the bending moment of the FFPs at the location of the rupture plane. As mentioned in the section entitled “Model piles and capping beam”, the ultimate bending strength of the FFP was 570.20 kN•m (i.e., 8.70 N•m at the model scale). Therefore, when the excavation depth was 9.6 m, the safety factor of the bending strength was 1.3 (= 570.20/439.10) for the FFPs in Test 5.

Earth pressure acting on the pile

For these tests, the earth pressure acting on the piles at a depth of 6.4 m prior to excavation was approximately 53.12 kPa; the corresponding static earth pressure coefficient was 0.52, which was similar to the theoretical value of $K_0 = 1 - \sin\phi = 0.49$. After the excavation, the earth pressure decreased to 33.92 kPa and the corresponding active earth pressure coefficient was 0.33, which was also similar to the value based on Rankine's earth pressure theory, i.e., $K_a = \tan^2(45^\circ - \phi/2) = 0.32$.

ANALYSIS OF THE LOAD TRANSFER MECHANISMS UNDERLYING THE PARTIAL COLLAPSE TESTS (TEST 1–TEST 4)

Analysis of the collapse processes and consequences in partial collapse tests

General processes in partial collapse tests

When the excavation was finished, the rupture of the IFPs was controlled to initiate the partial collapse. In Test 1–Test 4, after the partial failure of the IFPs, the adjacent piles remained intact. Test 1 was adopted as a typical case to introduce the general
process of the collapse. As shown in Fig. 10, the collapse process can be divided into three characteristic phases, as described below.

(1) Phase (a): As shown in Fig. 10 (a)-(b), in the first phase (from 0 s to 1.12 s after the failure of the IFPs), the soil outside the excavation in Zone 1 moved immediately due to the overturning of the IFPs. The displacement vectors of the soil at the end of Phase (a), derived using the PIV (Particle Image Velocimetry) technique (Stanier et al. 2015), are shown in Fig. 10 (b).

In the horizontal cutting plane of the model above the elevation of the preset rupture plane of the IFP, the horizontal movements of the IFPs were similar to those of the trapdoor test, as shown in Fig. 11. In the conventional trapdoor test, the trapdoor usually moves downwards in the vertical direction, whereas the IFPs served as a trapdoor that moved in the horizontal direction. Therefore, the movements of the IFPs in this phase caused a horizontal stress arching in the soil around the initial failure area, which would thus lead to the increase of the horizontal earth pressure acting on the piles adjacent to the IFPs. Because the development of the arching effect only requires a very small soil displacement (Pardo and Sáez 2014), the horizontal arching effect in this model can sufficiently form within a very short period of time after the partial failure. The detailed consequences of the arching effect will be discussed in the section entitled “Analysis of the load transfer mechanisms underlying Test 1”.

(2) Phase (b): In the second phase (from 1.12 to 2.88 s), a slip surface was created inside Zone 1, as shown in Fig. 10 (c), which is similar to the slip surface of the overturning failure of a retaining wall. In this phase, the failed piles and the soil inside the slip surface (in Zone 2) experienced a very large displacement; however, the soil outside the slip surface nearly remained still, as shown in Fig. 10 (c) and (d). The
stability of the soil in Zone 3 was due to the arching effect that developed in the first phase.

(3) Phase (c): In the third phase (after 2.88 s), the soil in the upper part of Zone 3 (i.e., Zone 4) began to fail and slide into the excavation, as shown in Fig. 10 (e) and (f). The sliding of the sand lasted for more than 40 s until a steady slope formed, as shown in Fig. 10 (f). During this phase, the soil outside Zone 2 and Zone 4 continued to remain still.

Note that the process of collapse is a transition process that changes from the development of the arching effect to the overturning failure of the retaining wall and then to the landslide failure of a slope. Finally, the sliding of unstable sand causes a deep crater outside the excavation and a steady slope inside the excavation.

During Phases (b) and (c), due to the failure of the soil in Zones 2 and 4, the soil behind the intact piles adjacent to the IFPs and inside the boundary of the crater (i.e., the unloading effect extent P3-P7 in Fig. 11) became loosened and gradually slid into the excavation, which would destroy part of the arch springing of the stress arching behind these piles. Consequently, the horizontal load acting on these piles and the bending moments of these piles would become smaller. Therefore, this phenomenon was an unloading effect for certain adjacent piles (P3-P7 in Test 1); its consequences will also be discussed in detail in the section entitled “Analysis of the load transfer mechanisms underlying Test 1”.

The scenarios after collapse in partial collapse tests

The collapse processes of Test 2–Test 4 were similar to that of Test 1. When the collapses stopped, the scenarios in these tests were as shown in Fig. 12. For Test 1–Test 4, the sizes of the craters outside the excavations, which represent the unloading effect
extents, are summarized in Table 4. Due to the partial collapse extent (4 IFPs) in Test 2
being larger than that in Test 1 (2 IFPs), the length of the crater became larger; however,
their widths and depths were nearly the same. The excavation depth of Test 3 (12.0 m)
was larger than that of Test 1 (9.6 m); therefore, the size of the crater was much larger
than that in Test 1. Because the initial failure condition of Test 4 was the same as that of
Test 1, the size of the half-crater in Test 4 was close to that in Test 1.

**Analysis of the load transfer mechanisms underlying Test 1**

**Changes in the bending moments of adjacent piles**

(1) Phase (a):

As discussed in the previous sections, in the first phase, a stress arching developed
instantaneously after the failure of the IFPs in the active zone of the excavation. Fig. 13
(a) indicates that due to the arching effect, the maximum bending moments of piles at a
certain distance from the partial collapse (approximately 14 piles: P3-P16) increased
rapidly after the collapse and reached their peak values within approximately 1.12 s.

The peak increase ratios of the pile moments (which were equal to the ratios of the
peak moments after collapse over the moments before partial collapse) are illustrated in
Fig. 11. The increase ratio for the pile closest to the partial collapse (P3) was largest at
1.42. As the distance between the pile and partial collapse becomes larger, the increase
ratio becomes smaller.

As shown in Fig. 11, if the width of the trapdoor (initial failed extent) in this test was
$D$ for the entire model (the width of 4 piles), then the influence range of the trapdoor
was approximately $3.5D$ (the width of 14 piles). In the trapdoor tests conducted by other
researchers, the influence range was approximately $2-3D$ (Pardo and Sáez 2014; Adachi
et al. 2003). By contrast, the influence range of the arching effect in this test was
relatively large. This is because adjacent piles, which served as the base around the
trapdoor, were not fixed and rigid. When the IFPs failed, the arching effect began to
develop and some adjacent piles were forced to move towards the excavation.
Consequently, the movements of these piles (which worked as another trapdoor) further
causd a new stress arching, which would in turn cause more piles to move towards the
evacation. This dynamic development process continued until the arching effect nearly
could not cause additional piles to move. However, in the conventional trapdoor test,
the bases adjacent to the trapdoor are usually rigid. Thus, there is no such dynamic
development process in the conventional trapdoor test.

(2) Phases (b) and (c):
As shown in Fig. 13 (a), due to the unloading effect caused by the failure of the soil
behind P3-P7 in Phases (b) and (c), the maximum bending moments of P3, P4 and P6
began to decrease after reaching their peak values until a steady state was reached. The
residual increase ratios of the bending moments are shown in Fig. 11. However, the
maximum bending moments of P8 and other piles that were distant from the partial
collapse did not exhibit an unloading phenomenon because the crater caused by the
partial collapse only extended to P7.

Based on the above analysis, certain piles adjacent to the partial collapse experienced
loading and unloading phases after the partial collapse. However, the unloading effect
lagged behind the loading effect.

Changes in earth pressures
(1) Phase (a):
Fig. 13 (b) shows the changing curves of the earth pressures acting on the pile
sections 6.4 m below the ground surface. In the first phase after the collapse (0-1.12 s),
the earth pressures acting on the adjacent piles should increase rapidly due to the arching effect. The sharp increasing of the bending moments of the piles can prove this. However, the earth pressures acting on P3, P4 and P6 at a depth of 6.4 m decreased instantaneously after the collapse for a very short period of time (approximately 0.8 s). This phenomenon can also be found in other tests and can be interpreted based on the pile displacement, as below.

The arching effect can cause adjacent piles to move horizontally towards the excavation. However, the horizontal pile displacements can, in turn, cause the decrease of the earth pressures acting on them. Therefore, the change in earth pressure was the combined action of both the arching effect and the pile displacement. In the first phase, as shown in Fig. 10 (b), because the soil displacements in the upper part of Zone 1 (shallower than approximately 4.8 m) were larger, the arching effect in this part was stronger than that in the lower part of Zone 1 (deeper than approximately 4.8 m, including the elevation of the earth pressure cells). For P3, P4 and P6, in the upper part of Zone 1, the earth pressure increase caused by the arching effect was dominant, whereas in the lower part, the earth pressure decrease caused by the pile displacement was dominant. Therefore, it is reasonable that the earth pressures acting on the nearest piles adjacent to partial collapse at a depth of 6.4 m decreased immediately and temporarily after the collapse.

For piles that were relatively distant from the partial collapse, such as P11, the earth pressures at a depth of 6.4 m increased immediately after the collapse. This is because the displacements of these piles were relatively small; thus, the changing of the earth pressures was dominated by the arching effect.

(2) Phases (b) and (c):
During Phases (b) and (c), due to the unloading effect caused by the failure of the soil behind P3-P7, the earth pressures exerted by the sliding soil should decrease. The decrease of the bending moments in P3-P7 can indirectly prove this. However, the earth pressures acting on P3 and P4 at the depth of 6.4 m gradually increased in Phases (b) and (c). This is mainly due to the new arching effect created by the continuous sliding of the soil through the partial collapse area during Phases (b) and (c). After 20 s, when the sliding of the soil nearly stopped, the earth pressures gradually approached the steady state.

The horizontal arching effect caused by the partial collapse can be considerably intensive. For instance, the depth of the crater behind P3 was 4.8 m, as shown in Table 4; therefore, the soil cover depth of the earth pressure cell installed in P3 reduced from 6.4 m to 1.6 m after the collapse. Consequently, the vertical soil pressure at this point should have decreased significantly. However, the horizontal pressure acting on the earth pressure cell increased by a factor of 1.86 due to the horizontal arching effect.

Based on the above analysis, the change of the earth pressures acting on adjacent piles was considerably complicated, being a combined action of both the arching effect and the pile displacement. Additionally, the variation patterns of the earth pressures acting on different piles and at different depths were significantly distinct.

**Load transfer coefficients**

The peak increase ratio of the bending moment in an adjacent pile induced by partial collapse is defined as load transfer coefficient \( T_n \), where \( n \) denotes the \( n \)th pile from the partial collapse. As shown in Fig. 14, the load transfer coefficient of the first adjacent pile \( (T_1) \) was largest and thus is defined as the maximum load transfer coefficient \( T_{\text{max}} \). If the failure of a pile is dominated by the bending capacity, assuming that the safety
factor of the bending strength of the first adjacent pile is $K_d$, if $T_{\text{max}} > K_d$, then the pile would fail and a follow-up progressive collapse would begin. However, if $T_{\text{max}} < K_d$, then progressive collapse would not occur. Based on the above analysis, under the condition of partial collapse, $T_{\text{max}}$ is a crucial factor for determining whether a progressive collapse could occur. To avoid progressive collapse, $K_d$ should be designed to be larger than $T_{\text{max}}$.

Parametric study of the influential factors of the load transfer mechanism

The influence of the initial partial collapse extent (analysis of Test 2)

In addition to the number of IFPs, other conditions of Test 2 were the same as for Test 1. As shown in Fig. 15 (a) and (b), the changing rules of the maximum bending moments and earth pressures in Test 2 were similar to those in Test 1. The phenomena exhibited in Test 1 can also be found in Test 2. As shown in Fig. 14, the load transfer coefficients of Test 2 are much larger than those of Test 1. $T_{\text{max}}$ (1.64) and the influenced extent (30 piles) in Test 2 were both larger than those in Test 1 (1.42 and 14 piles). The reason for this comparison result is that when more piles failed (which means the trapdoor was wider), the unbalanced soil load became larger and therefore the arching effect was more intensive. Thus, the influence degree was larger. Additionally, as shown in Fig. 16, the increased displacements in Test 2 caused by the arching effect were much larger than those of Test 1.

Based on the above analysis, it can be speculated that as a progressive collapse induced by a small scale partial failure evolves, $T_{\text{max}}$ can gradually become larger before the collapse extent and the unloading effect are larger than a certain degree.

The influence of the excavation depth (analysis of Test 3)
The excavation depth of Test 3 was 12.0 m, which was much deeper than that of Test 1 (9.6 m). As shown in Fig. 17 (a), changes in the bending moments of the piles in Test 3 were similar to those of Test 1. However, as shown in Fig. 17 (b), after the earth pressures at a depth of 6.4 m acting on P3 and P4 reached their peak values, they began to decrease significantly, which was different from Test 1. This was because, due to the excavation depth being larger than that in Test 1, more sand slid into the excavation. For instance, as shown in Table 4, the depth of the crater behind P3 after the collapse was 5.92 m, which indicates that the cover depth of the earth pressure cell was only 0.48 m. Therefore, the unloading effect on the earth pressures of P3 and P4 was very significant, which was the reason for the decrease of these earth pressures.

As shown in Fig. 14, a comparison of the load transfer coefficients of Test 3 with those of Test 1 shows that when the excavation was deeper, the load transfer coefficients of the piles near the partial collapse (including $T_{\text{max}}$) were lower. However, the load transfer extent was larger. This is because the lateral stiffness of the pile in Test 3 was much lower than that in Test 1. Therefore, the pile displacements in Test 3 caused by the arching effect were much larger than those in Test 1, as shown in Fig. 16. Consequently, because larger pile displacements created a larger arching effect in the surrounding soil, the dynamic development process (as discussed in the section entitled “Changes in the bending moments of adjacent piles”) in Test 3 extended for a larger extent until the equilibrium state was achieved. Correspondingly, the load transfer coefficients of the closest piles were lower.

In practical engineering, for a specific excavation, when the excavation depth is greater, $T_{\text{max}}$ becomes lower. Therefore, when partial collapse occurs, it seems that the occurrence possibility of the progressive collapse would be lower. However, for a
greater excavation depth, the occurrence possibility of the partial collapse is larger; moreover, the safety factors of the IPs are lower. Therefore, the occurrence possibility of progressive collapse might be higher, and thus should be further examined. Additionally, when progressive collapse occurs in a deeper excavation, it would develop to a much larger extent.

*The influence of the capping beam (analysis of Test 4)*

As shown in Fig. 18 (a) and (b), the changing rules of the maximum bending moments and earth pressures in Test 4 were similar to those in Test 1. Because the capping beam can coordinate the deformations of the piles, the displacements of the piles did not vary significantly in Test 4, as shown in Fig. 16.

Fig. 19 shows a comparison of the load transfer coefficients of Test 1 and Test 4. For the capping-beam case, the differences in the load transfer coefficients of different piles were smaller and more piles were influenced by the partial collapse. This is because the continuous capping beam could transfer the loads acting on the closer piles to the more distant piles. Therefore, the increase ratios of the bending moments of the nearest piles could be reduced.

For a specific excavation, a retaining structure that consists of a continuous capping beam may prevent progressive collapse. For instance, if the safety factor of the bending capacity of the pile $K_d$ is 1.30, because the $T_{\text{max}}$ of the capping-beam case (1.2) is smaller than $K_d$ when 2 piles fail, then a progressive collapse can be avoided. However, for the non-capping-beam case (i.e., Test 1), $T_{\text{max}}$ (1.42) is larger than $K_d$; therefore, progressive collapse would occur. Thus, the continuous capping beam plays an important role in preventing progressive collapses and should always be properly installed in practice.
ANALYSIS OF THE PROGRESSIVE COLLAPSE MECHANISM IN TEST 5

Test 5 was designed based on Test 2. When the excavation was finished, the rupture of 4 IFPs was controlled to initiate the partial collapse. $T_{\text{max}}$ under the condition that 4 piles failed in Test 5 was considered to be the same as that in Test 2 (1.64), which is much larger than the safety factor $K_d$ of the FFPs (1.3). Therefore, a progressive collapse occurred, which verified the occurrence condition ($T_{\text{max}} > K_d$) of the progressive collapse discussed in the section entitled “Load transfer coefficients”.

Collapse processes of the progressive collapse test

Generally, from the side view of the excavation, the collapse process was identical to that in Test 1 and could also be divided into three phases. 0-0.8 s, 0.8-2.24 s and after 2.24 s were Phase (a), Phase (b) and Phase (c), respectively.

From the front view of the contiguous piles, as shown in Fig. 20, the failure sequence of piles can be observed. After 4 IFPs failed, 4 adjacent piles (P5-P8) rapidly failed one-by-one at 0.52, 0.68, 1.88 and 2.40 s, respectively, which was similar to a domino effect.

The progressive failure of the piles can be interpreted as below. After the partial collapse, the arching effect began to develop and the first adjacent pile (P5) reached the ultimate flexural capacity and failed first. After that, the arching effect continued to develop. Because the failed pile P5 became a new trapdoor, the earth pressure acting on the adjacent pile P6 suddenly increased. This phenomenon also occurred in the trapdoor test performed by Adachi et al. (2003), who lowered several trapdoors one-by-one. Therefore, the maximum bending moment of P6 reached the ultimate value and failed. After the failure of P5 and P6, when the collapse process entered the second phase, the soil behind certain adjacent intact piles began to loosen and destroy the arch springing
of the stress arching (the unloading effect for these piles). However, at the beginning of this unloading phase, the arching effect caused by the previous failed piles was still much larger than the unloading effect. Consequently, P7 and P8 failed one-by-one. Afterwards, when the unloading effect became sufficiently large, the soil arching effect could not cause the failure of P9. Hence, the progressive collapse ceased. The unloading effect for the adjacent intact piles caused by the unstable and loosened soil behind them was the main reason for the stop of the progressive collapse.

**Changes in earth pressures and bending moments**

Fig. 21 (a) shows the changes in earth pressures acting on certain piles 6.4 m deep. Because of the pile displacements of P5, P6 and P7 after the partial collapse, the earth pressures acting on them at the depth of 6.4 m decreased slightly first and then increased. These phenomena were similar to those in the partial collapse tests. However, when P5-P8 failed because of the sudden movements of these failed piles, the earth pressures acting on them began to decrease. When P5-P8 fell down to the bottom of the excavation, the earth pressures acting on them began to oscillate significantly.

Fig. 21 (b) shows the changes in the maximum bending moments of adjacent piles. When pile P5 failed, its maximum bending moment decreased suddenly because the upper half of pile P5 could not sustain the earth pressure. When P5 fell to the bottom, the bending moment of P5 began to oscillate.

The moments at which the piles (P5-P8) failed and fell down to the bottom of the excavation, derived from the video analysis, matched with the changes of the earth pressures and bending moments.
DISCUSSIONS

The 1-g model tests using dry sand were conducted in this study. Therefore, there are several limitations as following.

(1) The effect of underground water on the failure of excavation was not taken into account. Obviously, the seepage or inrush of the water may intensify both local and global failure of excavation.

(2) The scaling law of the 1-g model tests in this study is only applicable to the sand models. The progressive collapse modes of excavation in clayey soil may be different because the soil type significantly influences the characteristic of the arching effect and the kinematic performance of the soil mass.

(3) For a given prototype, the scaling law of the 1g model tests adopted in this study can be satisfied only with the void ratio precisely controlled as the model response in low stress level is very sensitive to the relative density. It may not be feasible to precisely achieve an expected sand density using the sand rain method, and hence it is difficult to establish a model via controlling void ratio to completely reproduce a targeted prototype (Sedran et al. 2001). Based on this kind of understanding, the model tests in this study only focus on how to repetitively obtain a relatively low density under controlled laboratory conditions so that the model response is reasonable after being extrapolated to a field response. Therefore, the emphasis of this study is mainly to address the basic mechanism and key influencing factors of the progressive collapse in excavations, instead of to imitate the absolute response in a real project. The authors believe, in current stage, the physical model tests with those reliable findings, can help the designers have an initial insight of both the occurrence and evolution of the
progressive collapse, and hence promote the development of current design theory for the retaining system so as to prevent such catastrophes.

Centrifuge tests have been considered for the next work due to the more reasonable stress state as well as the easily fulfilled scaling rules for various soil types, although more sophisticated modelling devices need to be developed to capture progressive collapse in flight.

CONCLUSIONS

In this study, model tests of partial collapse and progressive collapse of a long-strip excavation, retained by cantilever contiguous piles, were conducted and analyzed. The occurrence and evolution mechanisms of the progressive collapse were discussed. The main conclusions of this research are summarized below.

(1) After the partial collapse, a horizontal stress arching formed and led to the sharp increase of the bending moments in adjacent piles (loading effect). After that, the soil behind certain adjacent piles became loosened and gradually slid into the excavation, which led to the decrease of the bending moments in these piles (unloading effect). Partial collapse has both a loading effect and an unloading effect on adjacent piles.

(2) Because the piles adjacent to the partial collapse can move under the arching effect, the influenced extents of the arching effect in the partial collapses of excavations are generally larger than those in the conventional trapdoor test. Additionally, the change of the earth pressures acting on adjacent piles was a combined action of both the arching effect and the pile displacement.

(3) The load transfer coefficients are defined to be equal to the peak increase ratios of the maximum bending moments in adjacent piles caused by the partial collapse over the
values before partial collapse. When the maximum load transfer coefficient $T_{\text{max}}$ is larger than the bearing capacity safety factor of the adjacent intact piles, the partial failure will lead to progressive collapse.

(4) Three influential factors of the load transfer mechanism were investigated. 1) Partial collapse extent: when more piles fail abruptly, the load transfer coefficients and the number of influenced piles become larger. 2) Excavation depth: when an excavation is deeper, $T_{\text{max}}$ is smaller but more piles will be influenced; however, the occurrence possibility of progressive collapse should be further examined because the safety factor of this excavation becomes lower. 3) Capping beam: a continuous capping beam can reduce $T_{\text{max}}$ and should be properly installed to resist progressive collapse.

(5) During the progressive collapse process, a previously failed pile could cause new stress arching acting on the next adjacent pile, which is an evolution mechanism for progressive collapse. Simultaneously, the soil behind certain nearest intact piles could be loosened. When this unloading effect becomes sufficiently large, the soil arching effect cannot cause the failure of additional piles. Hence, the progressive collapse ceases.
ACKNOWLEDGEMENTS

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collapse, (h) the collapse has stopped, and (i) the collapse has stopped (another perspective).

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Table 1 Scaling relations of 1-g physical modeling in sand (Altaee and Fellenius 1994, Gibson 1997, Meymand 1998)

<table>
<thead>
<tr>
<th>Quantity to be scaled</th>
<th>Prototype to model ratio</th>
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<td>$e_m = e_p + \lambda \ln(1/n)$</td>
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Table 2 Parameters of the sand used in the model test

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<th>Specific gravity $G_s$</th>
<th>Mean grain size $D_{50}$ (mm)</th>
<th>Coefficient of nonuniformity $C_u$</th>
<th>Maximum void ratio $e_{max}$</th>
<th>Minimum void ratio $e_{min}$</th>
<th>Critical state friction angle $\phi$ (°)</th>
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<td>0.85</td>
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Table 3 Parameters of the piles and excavations in the prototype and model

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<th></th>
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<td>$EI$ (N·m)</td>
<td>Diameter (m)</td>
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Table 4 Sizes of the craters caused by collapses in different tests

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<th>Length of crater (cm)</th>
<th>Width of crater (cm)</th>
<th>Depth of crater (cm)</th>
<th>Pile to which the crater extended</th>
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<tr>
<td>Test 3</td>
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<tr>
<td>Test 4</td>
<td>48.0</td>
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<td>Test 5</td>
<td>92.0</td>
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<td>32.0</td>
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</table>
Fig. 1. Schematic model setup (taking Test 1 as an example, and all the dimensions are in mm): (a) Side elevation, and (b) Plan view from the top.
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\[ y = -0.0389 \ln(x) + 0.8490 \]
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54x34mm (600 x 600 DPI)
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252x542mm (300 x 300 DPI)
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51x30mm (600 x 600 DPI)
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