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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

49 INTRODUCTION

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

57 Many large-scale excavation collapses have been triggered by a partial failure. For
58 instance, the accident records of the Singapore metro station collapse indicate that the
59 collapse proceeded with a domino-like effect (Davies 2007; COI 2005; Artola 2005;
60 Whittle and Davies 2006; Puzrin et al. 2010; Corral and Whittle 2014; Ishihara and Lee
61 2008). Additionally, progressive collapse along the length of the excavation is
62 especially serious. The excavation collapses of the metro stations in Cologne (Haack
63 2009), Hangzhou (Li et al. 2010; Chen et al. 2013) and Singapore developed to be
64 approximately 50 m, 70 m and 100 m in length, respectively.

65 The progressive collapse problem in structural engineering has been investigated
66 substantially (Dusenberry 2002; Masoero et al. 2009; Starossek 2006; Marjanishvili
67 2004; Frangopol and Curley 1987; Dusenberry 2002; GSA 2003; National Research
68 Council of Canada 2003, BS8110-1; EN1991-1-7; Menchel et al. 2009; Murtha-Smith
69 1988). However, the occurrence frequency and consequences of excavation collapses
70 could be even higher than those in structural engineering. Therefore, it is urgent to
71 establish a related theory and design methodology for resisting the progressive collapse
72 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 $\times$ 2.46 m $\times$ 1.40 m (length $\times$ width $\times$ 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

97 prototype/model), presented by them are presented in Table 1. Because the density
 98 difference between the model and prototype is small and can be ignored (Gibson 1997,
 99 Scott 1989), the stress scaling ratio N approximately equals to the geometric scaling
 100 ratio n . The geometric scaling ratio n was set at 16 for all of the models in the tests. The
 101 corresponding scaling relations under this condition are also listed in Table 1.

102 ***Soil in the model test***

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

108 A series of consolidation undrained (CU) triaxial tests was performed to estimate the
 109 steady-state behavior of the sand (Gibson 1997). As illustrated in Fig. 4, the slope of the
 110 steady-state line (SSL) λ in the e - $\ln p$ plane is -0.0389. Based on the scaling relation
 111 $e_m = e_p + \lambda \ln(1/n)$, the void ratio in the prototype can be determined, which varies from
 112 0.65 to 0.51 and lies within the range defined by the maximum and minimum void
 113 ratios (0.85-0.43) of the sand, indicating that the void ratios are reasonable.

114 In this study, the size of the sand grains is not scaled by the geometric scaling ratio n ,
 115 which might have an influence on the model response (Sedran et al. 2001). According
 116 to the catalogue of scaling laws provided by the Technical Committee TC2 (Physical
 117 Modelling in Geotechnics) of the ISSMGE (Garnier et al. 2007), the grain size would
 118 not significantly influence the response of model piles when the ratio of the pile
 119 diameter to the mean grain size (B/D_{50}) was larger than 45 (Nunez et al. 1988) or 60

120 (Remaud 1999). In the performed tests, a relatively fine sand with a B/D_{50} (60/0.23) of
121 261 was chosen so that the grain-size effect was deemed to be negligible.

122 ***Model piles and capping beam***

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

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

133 (1) Monitoring pile (MP)

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

138 Through the loading test on the simply supported MP, the flexural stiffness EI of the
139 pile was measured to be $560 \text{ N}\cdot\text{m}^2$; the strain value of the strain gauge was 29μ when
140 the pile section was subjected to the unit pure bending moment ($1 \text{ N}\cdot\text{m}$). Based on the
141 similitude relationship in Table 1, the prototype retaining piles (made of C30 reinforced
142 concrete) were approximately 0.8 m in diameter and 1.0 m in spacing, values that are

143 frequently used in practical engineering. Other geometric and physical parameters of the
144 model and prototype excavations are presented in Table 3.

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

150 (2) Initiating-failure pile (IFP)

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

155 (3) Following-failure pile (FFP)

156 The maximum bending moment of the retaining piles in the model tests
157 (approximately 30 N•m) is much lower than the ultimate bending strength of an intact
158 pile (147 N•m). Therefore, the intact pile cannot fail under the condition of partial
159 collapse. To simulate the failure of the neighboring piles caused by the partial collapse
160 (progressive collapse), the following-failure pile (FFP) was designed; the working
161 mechanism is illustrated in Fig. 6. During the excavation, the thin steel wire rope (0.6
162 mm in diameter) is subjected to the tensile force. Therefore, the ultimate bending
163 strength of the FFP is determined by the tensile strength of the steel wire rope, which is
164 208.75 N. Through loading tests, the ultimate bending strength of the FFP was
165 measured to be 8.70 N•m.

166 (4) Capping beam

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

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

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

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

187 Except for Test 3, the total excavation depth in other tests was 60 cm and each
188 excavation step was 10 cm. For Test 3, the excavation depth was 75 cm. When the
189 excavation depth was smaller than 60 cm, each excavation step was 10 cm; after that,

190 the excavation step became 5 cm. In all tests, after the excavation, the rupture of the
191 IFPs was controlled to initiate the partial collapse of the excavation.

192

193 **RESULTS OF THE MODEL TESTS IN THE EXCAVATION STAGE**

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

197 ***Displacement of the pile top***

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

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

211 ***Bending moment of the pile***

212 Fig. 9 (a)–(c) shows the average bending moment curves of the monitoring piles at
213 different excavation depths for Test 1 to Test 3; the results in Test 4 and Test 5 were

214 similar to these curves. Based on the bending moment curves of the 5 tests, when the
 215 excavation depth was 9.6 m, the average value of the bending moments at the elevation
 216 of the preset rupture plane (9.6 m below the ground surface) in Test 1–Test 5 was
 217 439.10 kN•m, which is considered to be the bending moment of the FFPs at the location
 218 of the rupture plane. As mentioned in the section entitled “Model piles and capping
 219 beam”, the ultimate bending strength of the FFP was 570.20 kN•m (i.e., 8.70 N•m at the
 220 model scale). Therefore, when the excavation depth was 9.6 m, the safety factor of the
 221 bending strength was 1.3 ($= 570.20/439.10$) for the FFPs in Test 5.

222 *Earth pressure acting on the pile*

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

229

230 **ANALYSIS OF THE LOAD TRANSFER MECHANISMS UNDERLYING THE** 231 **PARTIAL COLLAPSE TESTS (TEST 1–TEST 4)**

232 *Analysis of the collapse processes and consequences in partial collapse tests*

233 *General processes in partial collapse tests*

234 When the excavation was finished, the rupture of the IFPs was controlled to initiate
 235 the partial collapse. In Test 1–Test 4, after the partial failure of the IFPs, the adjacent
 236 piles remained intact. Test 1 was adopted as a typical case to introduce the general

237 process of the collapse. As shown in Fig. 10, the collapse process can be divided into
238 three characteristic phases, as described below.

239 (1) Phase (a): As shown in Fig. 10 (a)-(b), in the first phase (from 0 s to 1.12 s after
240 the failure of the IFPs), the soil outside the excavation in Zone 1 moved immediately
241 due to the overturning of the IFPs. The displacement vectors of the soil at the end of
242 Phase (a), derived using the PIV (Particle Image Velocimetry) technique (Stanier et al.
243 2015), are shown in Fig. 10 (b).

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

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

261 stability of the soil in Zone 3 was due to the arching effect that developed in the first
262 phase.

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

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

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

281 *The scenarios after collapse in partial collapse tests*

282 The collapse processes of Test 2–Test 4 were similar to that of Test 1. When the
283 collapses stopped, the scenarios in these tests were as shown in Fig. 12. For Test 1–Test
284 4, the sizes of the craters outside the excavations, which represent the unloading effect

285 extents, are summarized in Table 4. Due to the partial collapse extent (4 IFPs) in Test 2
286 being larger than that in Test 1 (2 IFPs), the length of the crater became larger; however,
287 their widths and depths were nearly the same. The excavation depth of Test 3 (12.0 m)
288 was larger than that of Test 1 (9.6 m); therefore, the size of the crater was much larger
289 than that in Test 1. Because the initial failure condition of Test 4 was the same as that of
290 Test 1, the size of the half-crater in Test 4 was close to that in Test 1.

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

292 *Changes in the bending moments of adjacent piles*

293 (1) Phase (a):

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

299 The peak increase ratios of the pile moments (which were equal to the ratios of the
300 peak moments after collapse over the moments before partial collapse) are illustrated in
301 Fig. 11. The increase ratio for the pile closest to the partial collapse (P3) was largest at
302 1.42. As the distance between the pile and partial collapse becomes larger, the increase
303 ratio becomes smaller.

304 As shown in Fig. 11, if the width of the trapdoor (initial failed extent) in this test was
305 D for the entire model (the width of 4 piles), then the influence range of the trapdoor
306 was approximately $3.5D$ (the width of 14 piles). In the trapdoor tests conducted by other
307 researchers, the influence range was approximately $2-3D$ (Pardo and Sáez 2014; Adachi
308 et al. 2003). By contrast, the influence range of the arching effect in this test was

309 relatively large. This is because adjacent piles, which served as the base around the
310 trapdoor, were not fixed and rigid. When the IFPs failed, the arching effect began to
311 develop and some adjacent piles were forced to move towards the excavation.
312 Consequently, the movements of these piles (which worked as another trapdoor) further
313 caused a new stress arching, which would in turn cause more piles to move towards the
314 excavation. This dynamic development process continued until the arching effect nearly
315 could not cause additional piles to move. However, in the conventional trapdoor test,
316 the bases adjacent to the trapdoor are usually rigid. Thus, there is no such dynamic
317 development process in the conventional trapdoor test.

318 (2) Phases (b) and (c):

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

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

329 *Changes in earth pressures*

330 (1) Phase (a):

331 Fig. 13 (b) shows the changing curves of the earth pressures acting on the pile
332 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):

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

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

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

375 *Load transfer coefficients*

376 The peak increase ratio of the bending moment in an adjacent pile induced by partial
377 collapse is defined as *load transfer coefficient* T_n , where n denotes the n^{th} pile from the
378 partial collapse. As shown in Fig. 14, the load transfer coefficient of the first adjacent
379 pile (T_1) was largest and thus is defined as the *maximum load transfer coefficient* T_{max} .
380 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 $T_{\max} > K_d$, then the pile would fail and a follow-up progressive collapse would begin. However, if $T_{\max} < K_d$, then progressive collapse would not occur. Based on the above analysis, under the condition of partial collapse, $T_{\max}$ 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 $T_{\max}$.

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. $T_{\max}$ (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, $T_{\max}$ 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)

404 The excavation depth of Test 3 was 12.0 m, which was much deeper than that of Test
405 1 (9.6 m). As shown in Fig. 17 (a), changes in the bending moments of the piles in Test
406 3 were similar to those of Test 1. However, as shown in Fig. 17 (b), after the earth
407 pressures at a depth of 6.4 m acting on P3 and P4 reached their peak values, they began
408 to decrease significantly, which was different from Test 1. This was because, due to the
409 excavation depth being larger than that in Test 1, more sand slid into the excavation. For
410 instance, as shown in Table 4, the depth of the crater behind P3 after the collapse was
411 5.92 m, which indicates that the cover depth of the earth pressure cell was only 0.48 m.
412 Therefore, the unloading effect on the earth pressures of P3 and P4 was very significant,
413 which was the reason for the decrease of these earth pressures.

414 As shown in Fig. 14, a comparison of the load transfer coefficients of Test 3 with
415 those of Test 1 shows that when the excavation was deeper, the load transfer
416 coefficients of the piles near the partial collapse (including $T_{\max}$) were lower. However,
417 the load transfer extent was larger. This is because the lateral stiffness of the pile in Test
418 3 was much lower than that in Test 1. Therefore, the pile displacements in Test 3 caused
419 by the arching effect were much larger than those in Test 1, as shown in Fig. 16.
420 Consequently, because larger pile displacements created a larger arching effect in the
421 surrounding soil, the dynamic development process (as discussed in the section entitled
422 “Changes in the bending moments of adjacent piles”) in Test 3 extended for a larger
423 extent until the equilibrium state was achieved. Correspondingly, the load transfer
424 coefficients of the closest piles were lower.

425 In practical engineering, for a specific excavation, when the excavation depth is
426 greater, $T_{\max}$ becomes lower. Therefore, when partial collapse occurs, it seems that the
427 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 $T_{\max}$ 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), $T_{\max}$ (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.

452

453 **ANALYSIS OF THE PROGRESSIVE COLLAPSE MECHANISM IN TEST 5**

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

460 ***Collapse processes of the progressive collapse test***

461 Generally, from the side view of the excavation, the collapse process was identical to
462 that in Test 1 and could also be divided into three phases. 0-0.8 s, 0.8-2.24 s and after
463 2.24 s were Phase (a), Phase (b) and Phase (c), respectively.

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

467 The progressive failure of the piles can be interpreted as below. After the partial
468 collapse, the arching effect began to develop and the first adjacent pile (P5) reached the
469 ultimate flexural capacity and failed first. After that, the arching effect continued to
470 develop. Because the failed pile P5 became a new trapdoor, the earth pressure acting on
471 the adjacent pile P6 suddenly increased. This phenomenon also occurred in the trapdoor
472 test performed by Adachi et al. (2003), who lowered several trapdoors one-by-one.
473 Therefore, the maximum bending moment of P6 reached the ultimate value and failed.
474 After the failure of P5 and P6, when the collapse process entered the second phase, the
475 soil behind certain adjacent intact piles began to loosen and destroy the arch springing

476 of the stress arching (the unloading effect for these piles). However, at the beginning of
477 this unloading phase, the arching effect caused by the previous failed piles was still
478 much larger than the unloading effect. Consequently, P7 and P8 failed one-by-one.
479 Afterwards, when the unloading effect became sufficiently large, the soil arching effect
480 could not cause the failure of P9. Hence, the progressive collapse ceased. The unloading
481 effect for the adjacent intact piles caused by the unstable and loosened soil behind them
482 was the main reason for the stop of the progressive collapse.

483 ***Changes in earth pressures and bending moments***

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

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

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

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499

500 DISCUSSIONS

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

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

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

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

523 progressive collapse, and hence promote the development of current design theory for
524 the retaining system so as to prevent such catastrophes.

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

529

530 **CONCLUSIONS**

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

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

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

545 (3) The load transfer coefficients are defined to be equal to the peak increase ratios of
546 the maximum bending moments in adjacent piles caused by the partial collapse over the

547 values before partial collapse. When the maximum load transfer coefficient $T_{\max}$ is
548 larger than the bearing capacity safety factor of the adjacent intact piles, the partial
549 failure will lead to progressive collapse.

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

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

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575 **REFERENCES**

576 Adachi, T., Kimura, M., and Kishida, K. 2003. Experimental study on the distribution
577 of earth pressure and surface settlement through three-dimensional trapdoor tests.

578 *Tunnelling and Underground Space Technology*, **18**(2): 171-183.

579 Altaee, A., and Fellenius, B. H. 1994. Physical modeling in sand. *Canadian*
580 *geotechnical journal*, **31**(3): 420-431.

581 Artola, J. 2005. A solution to the braced excavation collapse in Singapore. M.Sc. thesis,
582 Department of Civil and Environmental Engineering, Stevens Institute of
583 Technology, Boston, USA.

584 Bjerrum, L., and Eide, O. 1956. Stability of strutted excavations in clay. *Geotechnique*,
585 **6**(1): 32-47. doi: 10.1680/geot.1956.6.1.32.

586 BS8110-1 structural use of concrete, part 1: code of practice for design and
587 construction; 1997.

588 Chang, M. 2000. Basal stability analysis of braced cuts in clay. *Journal of Geotechnical*
589 *& Geoenvironmental Engineering*, **126**(3): 276-279. doi: 10.1061/(ASCE)1090-
590 0241(2000)126:3(276).

591 Chen, R. P., Li, Z. C., Chen, Y. M., Ou, C. Y., Hu, Q., and Rao, M. 2013. Failure
592 investigation at a collapsed deep excavation in very sensitive organic soft clay.
593 *Journal of Performance of Constructed Facilities*, **29**(3): 04014078.

- 594 Chen, W. F. 1975. Limit analysis and soil plasticity. Elsevier Scientific Press Co., New
595 York, N.Y.
- 596 Chevalier, B., Combe, G., and Villard, P. 2012. Experimental and discrete element
597 modeling studies of the trapdoor problem: influence of the macro-mechanical
598 frictional parameters. *Acta Geotechnica*, **7**(1), 15-39.
- 599 COI. 2005. Report of the Committee of Inquiry into the incident at the MRT circle line
600 worksite that led to collapse of Nicoll Highway on 20 April 2004. Ministry of
601 Manpower, Singapore.
- 602 Corral, G., and Whittle, A. J. 2014. Re-Analysis of Deep Excavation Collapse Using a
603 Generalized Effective Stress Soil Model. *Earth Retention Conference 3 ASCE*:
604 720-731. doi: 10.1061/41128(384)72
- 605 Dusenberry, D. O., and Juneja, G. 2002. Review of existing guidelines and provisions
606 related to progressive collapse. Available from: <http://www.nibs.org>.
- 607 EN1991-1-7 Eurocode I: actions on structures, parts 1-7: general actions - accidental
608 actions; 2006.
- 609 Frangopol, D. M., and Curley, J. P. 1987. Effects of damage and redundancy on
610 structural reliability. *Journal of Structural Engineering*, **113**(7): 1533-1549. doi:
611 10.1061/(ASCE)0733-9445(1987)113:7(1533)
- 612 Garnier, J., Gaudin, C., Springman, S. M., et al. 2007. Catalogue of scaling laws and
613 similitude questions in geotechnical centrifuge modelling. *International Journal of*
614 *Physical Modelling in Geotechnics*, **3**(3): 1-23.
- 615 Gibson, A. D. 1997. Physical scale modeling of geotechnical structures at one-G. Ph.D.
616 thesis, California Institute of Technology.

- 617 Haack, A. 2009. Construction of the north-south-metro line in Cologne and the accident
618 on March 3rd, 2009. *In* Proceedings of the International Symposium on Social
619 Management Systems; p. 10.
- 620 Ishihara, K., and Lee, W. F. 2008. Forensic diagnosis for sitespecific ground conditions
621 in deep excavations of subway constructions. *In* Geotechnical and geophysical site
622 characterization. *Edited by* R.Q. Coutinho, P.W. Mayne. Taylor & Francis Group,
623 Boca Raton, FL. pp. 31-59.
- 624 Li, G. X., and Li, X. M. 2010. The shear strength in stability analysis of subway pit in
625 soft clay. *Geotechnical Investigation & Surveying*, **38**(1): 1–4.
- 626 Marjanishvili, S. M. 2004. Progressive analysis procedure for progressive collapse.
627 *Journal of Performance of Constructed Facilities*, **18**(2): 79-85. doi:
628 10.1061/(ASCE)0887-3828(2004)18:2(79).
- 629 Masoero, E., Wittel, F. K., Herrmann, H. J., and Chiaia, B. M. DEM simulations of the
630 progressive collapse of framed structures. Available from:
631 <http://citeseerx.ist.psu.edu/viewdoc/download?doi=10.1.1.385.1819&rep=rep1&type=pdf>.
632
- 633 Menchel, K., Massart, T. J., Rammer, Y., and Bouillard, P. 2009. Comparison and study
634 of different progressive collapse simulation techniques for RC structures. *Journal*
635 *of Structural Engineering*, **135**(6): 685-697. doi: 10.1061/(ASCE)0733-
636 9445(2009)135:6(685).
- 637 Meymand, P. J. 1998. Shaking table scale model tests of nonlinear soil-pile-
638 superstructure interaction in soft clay. Ph.D. thesis, University of California,
639 Berkeley.

- 640 Murtha-Smith, E. 1988. Alternate path analysis of space trusses for progressive
641 collapse. *Journal of Structural Engineering*, **114**(9): 1978-1999. doi:
642 10.1061/(ASCE)0733-9445(1988)114:9(1978).
- 643 National Research Council of Canada. 2003. National Building Code of Canada. Ottawa,
644 Canada.
- 645 Pardo, G. S., and Sáez, E. 2014. Experimental and numerical study of arching soil effect
646 in coarse sand. *Computers and Geotechnics*, **57**(4), 75-84.
- 647 Puzrin, A. M., and Alonso, E. E., and Pinyol, N.M. 2010. Braced excavation collapse:
648 Nicoll Highway, Singapore. *In Geomechanics of failures. Edited by A.M. Puzrin,*
649 *E.E. Alonso, and M. Nuria. Springer Verlag, Netherlands. pp. 151-181.*
- 650 Reul, O., and Gebreselassie, B. 2006. Excavations and foundations in soft soils.
651 Springer Verlag, Netherlands.
- 652 Scott, R. F. 1989. Centrifuge modeling and technology: a survey. *Rev. Franc. Geotech*,
653 48: 15-34.
- 654 Sedran, G., Stolle, D. F., and Horvath, R. G. 2001. An investigation of scaling and
655 dimensional analysis of axially loaded. [J]. *Canadian Geotechnical Journal*,
656 **38**(38): 530-541.
- 657 Starossek, U. 2006. Progressive collapse of structures: nomenclature and procedures.
658 *Structural Engineering International*, **16**(2):113–117. doi:
659 10.2749/101686606777962477.
- 660 Starossek, U. 2007. Typology of progressive collapse. *Engineering Structures*, **29**(9):
661 2302-2307. doi: 10.1016/j.engstruct.2006.11.025.

- 662 Stanier, S.A., Blaber, J., Take, W.A., and White, D.J. 2015. Improved image-based
663 deformation measurement for geotechnical applications. *Canadian Geotechnical*
664 *Journal*, **53**(5): 727-739. doi: 10.1139/cgj-2015-0253.
- 665 U.S. General Services Administration. 2003. Progressive Collapse Analysis and Design
666 Guidelines for New Federal Office Buildings and Major Modernization Projects.
667 USA.
- 668 Whittle, A. J., and Davies, R. V. 2006. Nicoll Highway collapse: evaluation of
669 geotechnical factors affecting design of excavation support system. *In Proceedings*
670 *of the International Conference on Deep Excavations*. Singapore.
- 671 Zheng, G., Cheng X. S., and Diao, Y. 2011. Concept and design methodology of
672 redundancy in braced excavation. *Geotechnical Engineering Journal of the*
673 *SEAGS & AGSSEA*, **42**(3): 13-21.
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Table 1 Scaling relations of 1-g physical modeling in sand (Altaee and Fellenius 1994, Gibson 1997, Meymand 1998)

Quantity to be scaled	Prototype to model ratio	Prototype to model ratio (for $N=n$)	Prototype to model ratio (for $n=16$)
Linear dimension	n	n	16
Area	n^2	n^2	256
Volume	n^3	n^3	4096
Acceleration	1	1	1
Stress	N	n	16
Strain	1	1	1
Displacement	n	n	16
Force	Nn^2	n^3	4096
Moment	Nn^3	n^4	65536
Void ratio	$e_m=e_p+\lambda\ln(1/N)$	$e_m=e_p+\lambda\ln(1/n)$	$e_m=e_p-0.0389\times\ln(1/16)$
Flexural stiffness of pile	Nn^4	n^5	1048576
Time	$n^{1/2}$	$n^{1/2}$	4

Table 2 Parameters of the sand used in the model test

Specific gravity	Mean grain size	Coefficient of nonuniformity	Maximum void ratio	Minimum void ratio	Critical state friction angle
G_s	D_{50} (mm)	C_u	e_{max}	e_{min}	φ (°)
2.67	0.23	2.25	0.85	0.43	30.96

Table 3 Parameters of the piles and excavations in the prototype and model

	Pile				Excavation		
	EI (N·m)	Diameter (m)	Spacing (m)	Length (m)	Width (m)	Length (m)	Depth (m)
Model	506		0.0615	1.2	0.9	2.46	0.6 (0.75)
Prototype	5.87e8	0.8	1.0	19.2	14.4	39.36	9.6 (12.0)

Table 4 Sizes of the craters caused by collapses in different tests

	Length of crater (cm)	Width of crater (cm)	Depth of crater (cm)	Pile to which the crater extended
Test 1	45.5	43.5	30.0	8
Test 2	66.0	43.0	32.0	11
Test 3	57.0	61.0	37.0	10
Test 4 (half model)	48.0	44.5	30.0	8
Test 5	92.0	43.0	32.0	15

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54x34mm (600 x 600 DPI)

Fig. 9. Bending moments for different tests: (a) Test 1, (b) Test 2, and (c) Test 3

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Fig. 13. Changing curves of the bending moments in adjacent piles and earth pressures acting on adjacent piles in Test 1: (a) the increase ratios of the maximum bending moments and (b) the increase ratios of the earth pressures at a depth of 6.4 m

442x593mm (240 x 240 DPI)

Fig. 14. Load transfer coefficients in different tests (without a capping beam)

51x30mm (600 x 600 DPI)

Fig. 15. Changing curves of the bending moments in adjacent piles and earth pressures acting on adjacent piles in Test 2: (a) the increase ratios of the maximum bending moments and (b) the increase ratios of the earth pressures at a depth of 6.4 m

443x538mm (240 x 240 DPI)

Fig. 16. Increases in the displacements of the pile heads induced by partial failure in the different tests

51x30mm (600 x 600 DPI)

Fig. 17. Changing curves of the bending moments in adjacent piles and earth pressures acting on adjacent piles in Test 3: (a) the increase ratios of the maximum bending moments and (b) the increase ratios of the earth pressures at a depth of 6.4 m

443x539mm (240 x 240 DPI)

Fig. 18. Changing curves of the bending moments in adjacent piles and earth pressures acting on adjacent piles in Test 4: (a) the increase ratios of the maximum bending moments and (b) the increase ratios of the earth pressures at a depth of 6.4 m

445x596mm (240 x 240 DPI)

Fig. 19. Comparison of load transfer coefficients in tests with and without a capping beam

51x30mm (600 x 600 DPI)

Fig. 20. Progressive collapse process of Test 5: (a) moment of partial collapse, (b) 2.0 s after partial collapse, (c) 2.8 s after partial collapse, (d) 3.2 s after partial collapse, (e) 3.72 s after partial collapse, (f) 4.28 s after partial collapse, (g) 4.8 s after partial collapse, (h) the collapse has stopped, and (i) the collapse has stopped (another perspective).

Fig. 21. Changing curves of the bending moments in adjacent piles and earth pressures acting on adjacent piles in Test 5 (progressive collapse test): (a) the increase ratios of the earth pressures at a depth of 6.4 m and (b) the increase ratios of the maximum bending moments

448x915mm (240 x 240 DPI)