INTERNAL STABILITY OF GROUP COLUMN TYPE DEEP MIXING IMPROVED GROUND UNDER EMBANKMENT LOADING

MASAKI KITAZUME and KENJI MARUYAMA

ABSTRACT

The Deep Mixing Method (DMM), an in-situ soil stabilization technique using cement and/or lime as a binder, is often applied to improve soft soils. The group column type pattern is extensively applied to stabilize foundations of embankment or lightweight structures. An improved-ground design procedure in Japan assumes two failure patterns related to external and internal stabilities. For the external stability, a collapse failure pattern where the DM columns tilt like dominos could take place instead of a sliding failure pattern. For the internal stability, the DM columns show shear, bending and tensile failure mode, depending not only on the ground and loading conditions but also on the location of each column. However, the current design does not incorporate the effect of these failure modes, but only the shear failure mode. In this study, a series of centrifuge model tests was carried out to investigate the internal stability of group column type improved ground under embankment loading. This paper describes the failure modes of DM columns and a proposed simple calculation that takes into account the bending failure mode.

Key words: centrifuge test, deep mixing method, deformation, embankment, failure, soft ground, stability (IGC: K6)

INTRODUCTION

The Deep Mixing Method (DMM), an in-situ soil stabilization technique, is often applied to improve soft soils (Coastal Development Institute of Technology, 2002). The group column type pattern is extensively applied to stabilize foundations of embankment or lightweight structures. An improved ground design procedure, established in Japan mainly for reinforcing embankment (Public Work Research Center, 2004), assumes two failure patterns related to external and internal stabilities. External stability is evaluated for the possibility of sliding failure, in which the DM columns and the clay between show horizontal displacement on a stiff layer without any rearrangement of columns. For internal stability, the possibility of rupture breaking failure is evaluated by slip circle analysis, assuming the shear failure mode of DM columns (Fig. 1).

For the external stability, Kitazume et al. (1991, 2000) performed a series of centrifuge model tests for a breakwater on column type DM ground reaching a stiff layer, and demonstrated that collapse failure where the columns tilt like dominos at the bottom could take place instead of sliding failure. Kitazume and Maruyama (2005, 2006) performed another series of centrifuge model tests and proposed a design method on external stability by incorporating the collapse failure pattern.

For the internal stability, Terashi and Tanaka (1983), Miyake et al. (1991), Karastanev et al. (1997), Hashizume et al. (1998) and Kitazume et al. (1996, 1999) carried out model tests revealing that the DM columns show various failure modes: shear, bending and tensile failure mode, depending not only on the ground and external loading conditions but also on the location of each column. However, the current design method does not incorporate the effect of these failure modes, but only the shear failure mode. As the bending and tensile strengths of treated soil are much lower than the compressive strength (Terashi et al., 1980), the current design method based on shear strength alone might overestimate the internal stability. Kivelo (1998) and Broms (2004) recently proposed a new design method for the group column type improved ground, in which several failure modes of DM columns are taken into account.

The authors conducted a research project on the failure mechanism and stability of group column type improved ground subjected to embankment loading. The project involved investigating the failure pattern and criteria related to external and internal stabilities. The research results on external stability have been presented (Kitazume and Maruyama, 2006). This study targets the latter subject in which a series of centrifuge model tests and numerical calculations were carried out to investigate the effect of DM column strength and improved ground width and improved area ratio on the internal stability. In the model tests, the development of bending moment distribution in DM columns and column failure were measured in detail to address the failure mechanism. This
paper describes the failure modes of the DM columns and a proposed simple calculation that takes into account the bending failure mode.

CENTRIFUGE MODEL TESTS

Apparatus and Model Ground Preparation

A series of model tests was carried out in the Mark II Geotechnical Centrifuge at the Port and Airport Research Institute (Kitazume and Miyajima, 1995). A strong specimen box with inside dimensions of 70 cm in length, 20 cm in width and 60 cm in depth was used. One side of the box was made of glass to allow photographic measurements during the flight. Figure 2 shows a typical example of the model ground setup, in which a normally consolidated clay ground of 20 cm thick and five rows of DM columns are modeled. The model ground for all the tests was prepared by the following procedure. A drainage layer of Toyoura sand was placed at the bottom of the specimen box. The Kaolin clay slurry with water content of 120% was poured into the specimen box, and then pre-consolidated one dimensionally by vertical pressure of 9.8 kN/m² on the laboratory floor to produce 22 cm thick clay ground. After completing the preliminary consolidation, the model clay ground was subjected to high centrifugal acceleration of 50 g to allow consolidation by enhanced self-weight and then the thickness of ground became 20 cm. Due to the pre-consolidation on the laboratory floor and the self-weight consolidation in the centrifuge, the model ground had a thin layer of over-consolidated clay underlain by the thick normally consolidated clay layer.

After the self-weight consolidation, the centrifuge was stopped once for preparation of improved ground on the laboratory floor. A thin walled tube with an outer diameter of 20 mm was penetrated into the clay ground. The clay inside the tube was then carefully removed using a tiny auger to make holes, and a model DM column was inserted after removing the tube. This procedure was repeated to produce the improved ground in a square pattern with an interval of 33 mm in Cases 2 through 11, as shown in the upper part of Fig. 2. The improvement area ratio, $a_s$, was defined as the ratio of sectional area of DM column to the hypothetical cylindrical area (CDIT, 2002), and was 0.28 for all the former and latter cases.

Several earth pressure gauges are placed on the top surface of the model column and of the clay inbetween in order to investigate the stress concentration phenomenon during the embankment loading. The embankment was constructed on the model ground by means of an in-flight sand-raining device in a 50 g acceleration field. The model ground for all the tests was prepared in the same manner as for the external stability tests (Kitazume and Maruyama, 2006) except for the model column material.

In the present model tests, three types of DM columns were used: acrylic pipe and cement treated columns, as shown in Table 1. A total of 11 model tests were carried out as summarized in Table 2. The former model column (A-column) was used in Cases 2 to 5 for investigating the external stability with bending moment measurements, while the latter two (Tl-column and Th-column) were used in Cases 6 to 11 for investigating the internal stability by simulating rupture breaking failure of DM columns. As the model material and characteristics of the A-column have already been reported (Kitazume and Maruyama, 2006), the preparation and characteristics of the later two columns are described here.

The Tl- and Th-columns, 2 cm in diameter and 20 cm in length, were manufactured using a mixture of Kawasaki clay and normal Portland cement. The mixture was poured into a acrylic mold of 2 cm of inner diameter and 25 cm in length. After curing, the column was extracted from the mold by means of a motor jack for installing into the model ground. The adhesion mobilized along the cement treated column was measured by pulling the column out from the clay ground on the laboratory floor. The test revealed that the average adhesion was almost same as the undrained shear strength of clay ground, although the outer surface of the column was not of course condition (Kitazume and Maruyama, 2006).

In order to detect the model column failure during embankment loading, a carbon rod was embedded into each column before hardening, as shown in Fig. 3. Both
Table 1. Engineering properties of model columns

<table>
<thead>
<tr>
<th>Name</th>
<th>Material</th>
<th>Carbon rod Diameter (mm)</th>
<th>Strength (MN/m²)</th>
<th>Mixing condition wᵢ (%)</th>
<th>aw (%)</th>
<th>qᵤ (kN/m²)</th>
<th>σₛ (kN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Acrylic</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Tl</td>
<td>Treated soil</td>
<td>2</td>
<td>62.6</td>
<td>160</td>
<td>12.5</td>
<td>409</td>
<td>132</td>
</tr>
<tr>
<td>Th</td>
<td>Treated soil</td>
<td>3.2</td>
<td>34.5</td>
<td>160</td>
<td>10.0</td>
<td>1332</td>
<td>331</td>
</tr>
</tbody>
</table>

wᵢ: initial water content (%), aw: amount of stabilizing agent, qᵤ: unconfined compressive strength (kN/m²), σₛ: bending strength (kN/m²)

ends of the carbon rod were connected to a thin cable to measure electric resistance during the test. As the carbon has high electrical transfer, its electric resistance is quite low; however, when the carbon rod is broken due to the rupture breaking failure of the column, the electric resistance jumps to infinity. Accordingly, the measurement of electric resistance can be an indicator for detecting the point in time of column failure, although the location of the failure point would not be detected until after the test. In Cases 6 to 11, all the columns embedded in the model ground had a carbon rod, while the electric measurements were conducted in the b, c and d column lines (see Fig. 2).

The mixing conditions for the two model columns are summarized in Table 1 together with the characteristics of the carbon rod. Both columns had an initial water content, wᵢ, of 160%, but the amount of cement, aw defined as the dry weight of cement against that of soil, differed. Two types of carbon rod were used. As no suitable carbon rod had been found on the market at the beginning, high strength carbon rod was obliged to be used for Th-column, which influenced the treated soil column property dominantly. After then, low strength carbon rod, which did not influence the column property so much, was found on the market and used for Tl-column. All the columns necessary for the entire model test series, (about 300 columns for each), were manufactured at the same time to obtain the same column property as much as possible through the test series and cured under moist conditions for more than three months to prevent strength increase during the model test series. The unconfined compressive strength, qᵤ, and the bending strength, σₛ, of Tl- and Th-columns in Table 1 were obtained after curing the reference specimens of 2 cm in diameter and 4 cm in height, and of 2 cm in diameter and 20 cm in length, respectively. As the carbon rod has high strength, its characteristics strongly influence the characteristics of the model column. The large diameter of the carbon rod provides the high column strength of Th-column compared to Tl-column even with a smaller amount of cement mixed. The columns embedded in the model ground were measured for strength after being excavated, and the results are summarized in Table 2.

Figure 4 shows the stress strain curves of the model columns with the carbon rod measured in unconfined compression tests, in which the model column was trimmed to 2 cm in diameter and 4 cm in height. The curves clearly show a rapid increase in axial stress and quite a sharp peak at a very small strain, followed by a rapid decrease in stress. In the figure, laboratory data of cement treated soil having similar magnitude of strength and different mixing conditions without carbon rod are plotted together simply to show the effect of the carbon rod. By comparing the data with and without the carbon rod, a similar stress strain phenomenon can be seen prior to the peak while a sharper decrease in the axial stress can be seen in the column with the carbon rod. This indicates that the model columns with the carbon rod are more brittle compared to those of the treated soil without any carbon rod.

Figure 5 shows typical bending test data on the model columns of 2 cm in diameter and 20 cm in length. The tests were conducted in a similar manner to concrete engineering (Japan Society of Civil Engineers, 2002). The vertical load increases with increasing vertical deflection,
Fig. 5. Vertical load and deflection curves in bending test

Fig. 6. Strength ratio of model columns

Table 2. Test conditions and major test results

<table>
<thead>
<tr>
<th>Improvement condition</th>
<th>Width (cm)</th>
<th>No. of rows</th>
<th>Imp. area ratio, $a$</th>
<th>Material</th>
<th>$q_u$ (kN/m²)</th>
<th>$s_b$ (kN/m²)</th>
<th>Embk. press. at failure, $p_{ef}$ (kN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
<td>0</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>10.8*</td>
</tr>
<tr>
<td>Case 2</td>
<td>8.6</td>
<td>3</td>
<td>0.28</td>
<td>A</td>
<td>—</td>
<td>—</td>
<td>26.5*</td>
</tr>
<tr>
<td>Case 3</td>
<td>15.2</td>
<td>5</td>
<td>0.28</td>
<td>A</td>
<td>—</td>
<td>—</td>
<td>42.2*</td>
</tr>
<tr>
<td>Case 4</td>
<td>21.8</td>
<td>7</td>
<td>0.28</td>
<td>A</td>
<td>—</td>
<td>—</td>
<td>50.0*</td>
</tr>
<tr>
<td>Case 6</td>
<td>8.6</td>
<td>3</td>
<td>0.28</td>
<td>TI</td>
<td>425</td>
<td>122</td>
<td>16.9–23.7</td>
</tr>
<tr>
<td>Case 7</td>
<td>15.2</td>
<td>5</td>
<td>0.28</td>
<td>TI</td>
<td>411</td>
<td>131</td>
<td>26.2–35.3</td>
</tr>
<tr>
<td>Case 8</td>
<td>21.8</td>
<td>7</td>
<td>0.28</td>
<td>TI</td>
<td>391</td>
<td>142</td>
<td>25.4–32.6</td>
</tr>
<tr>
<td>Case 9</td>
<td>8.6</td>
<td>3</td>
<td>0.28</td>
<td>Th</td>
<td>1271</td>
<td>312</td>
<td>33.3–49.7</td>
</tr>
<tr>
<td>Case 10</td>
<td>15.2</td>
<td>5</td>
<td>0.28</td>
<td>Th</td>
<td>1290</td>
<td>367</td>
<td>34.2–50.2</td>
</tr>
<tr>
<td>Case 11</td>
<td>21.8</td>
<td>7</td>
<td>0.28</td>
<td>Th</td>
<td>1434</td>
<td>316</td>
<td>47.9–68.5</td>
</tr>
</tbody>
</table>

*a defined by curve fitting

$q_u$: unconfined compressive strength (kN/m²), $s_b$: bending strength (kN/m²), $p_{ef}$: embankment pressure at ground failure.

and shows a sudden drop in the vertical load, irrespective of model column type. The TI-column shows a lower peak value at smaller $\delta$ values compared to Th-column. In the figure, the electric resistance of the carbon rod, which is converted to a micro unit, is plotted together. The resistance of each column increases gradually with little scattering until the peak vertical load. However, it jumps to infinity at the peak vertical load, which indicates the high applicability of the carbon rod for detecting the point in time of column failure.

Figure 6 shows the relationship between $q_u$ and $s_b$, measured on the reference columns trimmed to 4 cm in length for the $q_u$ test and 20 cm for the $s_b$ test. Although there is a lot of scatter for Th-column, an average strength ratio of 0.28 was obtained, which is within the range of previous research (Terashi et al., 1980).

Embankment Loading Procedure

The model ground constructed was brought up to a 50 g acceleration field, which corresponded to a 10 m thick soft clay layer improved by DM columns of 1 m in diameter in prototype scale. The model ground was allowed to consolidate by enhanced self-weight to minimize any soil disturbance effect that might be induced during the ground preparation. Next, the model embankment was constructed stepwise under almost undrained conditions using an in-flight sand-raining device: about 1 cm in height per 30 second interval until the ground failed. During the embankment loading, the vertical stress increments at the ground surface and at the top of the model columns were measured as well as the electric resistance of the model columns, and the model ground deformation was photographed. After the loading test, the specimen box was disassembled and deformation of the model columns was observed directly.

A total of 11 model tests were performed using different materials and a varying number of columns. The test conditions and results are summarized in Table 2. Cases 2 to 5 concern external stability with the bending moment measurements. Cases 6 to 11 concern internal stability. In the present paper, the test results for Cases 6 to 11 are mainly addressed with the investigation on the bending moment measured in Cases 2 to 5. The improvement width is defined here as the distance between the outer surfaces of the forefront and rearmost columns.

TEST RESULTS

Embankment Pressure and Displacement

The embankment pressure, $p_{ef}$, and horizontal displace-
(a) Improved ground with Tl-columns  
(b) Improved ground with Th-columns  
(c) Improved ground with A-columns

Fig. 7. Embankment pressure and horizontal displacement curves

Fig. 8. Measured micro of carbon rod together with embankment pressure and horizontal displacement curves

iment, $\delta_h$, curves are shown in Figs. 7(a) to 7(c) for the improved ground with different column materials, together with the unimproved ground. In the figure, the vertical axis shows the embankment pressure at the ground surface, and the horizontal axis shows the horizontal displacement at the toe of the embankment slope.

In the unimproved ground (Case 1), a relatively small horizontal displacement takes place as long as $p_e$ remains at a very low level, but $\delta_h$ increases rapidly with further increase of $p_e$. In the improved ground with Tl-columns (Cases 6 to 8), $\delta_h$ increases with increasing $p_e$ (Fig. 7(a)), but its magnitude is slightly smaller than that of the unimproved ground. It decreases as the number of columns increases. A similar phenomenon can be seen in the improved ground with Th- and A-columns, as shown in Figs. 7(b) and 7(c), respectively. As the embankment loading continued to a relatively high value for the ground with Th- and A-columns, it can clearly be seen that the ground improvement effect on the curve becomes more dominate with increasing $\delta_h$. Comparing the figures, the magnitude of $\delta_h$ decreases as the column strength and/or number of columns increases.

Figure 8 shows an example of the measured micro of the carbon rod in the DM column together with the $p_e$ for Case 9. The measured micros, plotted as thin lines in the figure, scatter very much during the loading, and those for columns 1b, 1d, 1c and 3b jump to infinity one after another. Those for columns 2b, 2c, 2d, 3c and 3d, on the other hand, do not jump up throughout the loading. It can be concluded from the figure together with the observation after the tests that the columns 1b, 1d, 1c and 3b failed when the measured micro jump to infinity.

In order to investigate the effect of column failure in detail, the $\delta_h$-$p_e$ curves of Cases 6 to 11 are plotted again in Figs. 9(a) to 9(f). In the figure, the letters beside the curves indicate the point in time and the ID number of the column that shows rupture breaking failure. The column ID is numbered as Row 1, 2, 3 and so on from the forefront column, and Line a, b, c and so on from the window, as shown in Fig. 2. In Case 6 (Fig. 9(a)), one of the forefront columns, Tl-1d, failed first at $p_e$ of about 16.9 kN/m². As $p_e$ increased, the other forefront columns, Tl-1b and Tl-1c, failed. The second and third rows of columns failed one by one at $p_e$ of 23.7 and 35.7 kN/m², respectively. It can be concluded that the columns fail in sequence from the forefront to the rearmost column. It is of interest to note that the $p_e$ value increases to increase even after many columns fail.

In Case 9 (Fig. 9(d)), with Th-columns, the forefront columns, Th-1b failed first at $p_e$ of 33.3 kN/m², which was higher than that in Case 6 due to the high column strength. Then Th-1d and Th-1c columns failed as increasing embankment loading. The third row columns, Th-3b, failed then.

Figures 9(b) and 9(e) show the test data of Cases 7 and 10. In Case 7, one of the forefront columns, Tl-1b, failed first at $p_e$ of 26.2 kN/m², and the second and third row columns, Tl-2b, Tl-2d and Tl-3c, failed at the same time. As $p_e$ increased, the columns failed one by one in sequence from the forefront to the rearmost column, which was a similar to that in Cases 6 and 9. In Case 10, the forefront columns failed one by one at $p_e$ of 34.2 to 50.2 kN/m² (Fig. 9(e)). When $p_e$ increased to 79.6 kN/m², the rearmost columns, Th-5b, Th-5c and Th-5d, failed instead of the second, third and fourth row columns. After that, Th-4 and Th-3 failed in reverse sequence from the rearmost to the forefront column.

In Case 8, the forefront column, Th-1b, failed first at $p_e$ of 25.4 kN/m² (Fig. 9(c)), and the first, second and third row columns failed at the next loading step, similar
to Case 6. After the failure of TI-3b and TI-3c, one of the rearmost columns, TI-7b, failed before TI-4, TI-5 and TI-6 failed. As the embankment loading was terminated at a relatively small embankment pressure in this case to prevent heavy column failure, no failure took place in TI-4, TI-5 and TI-6 during the loading.

In Case 11, one of the forefront columns, Th-1c, failed first at $p_e$ of 47.9 kN/m$^2$. The other two, Th-1 and one of Th-2 columns, failed at the next several loading steps. At $p_e$ of 68.5 and 73.3 kN/m$^2$, the rearmost columns failed instead of the second and third row columns.

It is of interest to note that the embankment pressure continues to increase even after many columns fail. The residual strength of cement treated soil is dependent upon the confining stress, $\sigma_f$, and is almost zero in the case of $\sigma_f = 0$, which causes some apprehension about column failure resulting in a sort of catastrophic failure of improved ground. In response, the current design was established based on the “safe side” concept. The test results discussed above provide a possibility for changing the basic concept of the current design method.

**Embankment Pressure at Ground Failure and Improvement Width**

As shown in Fig. 9, neither a clear peak nor constant value can be seen in the $p_e$ and $d_h$ curves even after many model columns fail. As far as the model test conditions, the forefront column always fails first, irrespective of the column strength and number of column rows. Here, the ground failure is defined as the rupture breaking failure of the forefront column. The embankment pressure at ground failure, $p_{ef}$, is summarized in Table 2, and the relationship to the improvement width, $D$, is plotted in Fig. 10 for Cases 6 to 11. As discussed in Fig. 9, the model columns fail one by one at several embankment pressures even in the forefront column. The pressure ranges where the forefront columns fail are plotted as arrows. It can be seen that $p_{ef}$ increases gradually with increasing $D$, irrespective of column strength.

**Column Failure**

Figures 11(a) to 11(f) show the failure pattern of
INTERNAL STABILITY OF DMM

columns observed after the embankment loading in Cases 6 to 11, respectively. In Case 6, as shown in Fig. 11(a), all the columns tilted counterclockwise with tensile cracks at two depths even when the embankment loading was terminated at a relatively low pressure. The figure clearly shows that the column did not fail by shear failure mode but rather by bending failure mode. As discussed in Fig. 9(a), the forefront column, Tl-1d, failed first and Tl-2d and Tl-3d failed next. Although there is no clear evidence, it is reasonable to assume that bending failure took place in each column, one by one. However, the electric measurement of the carbon rod did not show which crack took place first. According to the detailed observation after the test, bending failure took place at a shallow depth first and then at a deep depth. Counterclockwise displacement can be seen in Tl-1d and Tl-2d; however, the top of the rearmost column, Tl-3d, inclined clockwise slightly, due to large ground settlement beneath the embankment (Kitazume and Maruyama, 2005).

In Case 9, the bending failure can be clearly seen in the forefront and rearmost columns (Fig. 11(d)). It is of interest to note that the depth at the bending failure in the forefront column is deeper than in Case 6, indicating the influence of column strength, as explained in detail later.

In Case 7, all the columns tilted counterclockwise with bending failure. According to Fig. 9(b), Tl-1b and Tl-2b failed first and then the other three columns, Tl-3b, Tl-4b and Tl-5b, failed at the same $p_c$ of 43.9 kN/m$^2$. In Case 10, Th-1c and Th-2c failed and tilted counterclockwise, then Th-4c and Th-5c failed and tilted clockwise, as indicated in Fig. 9(e).

Figure 11(c) shows the failure pattern of columns in Case 8. Bending failure took place in Tl-1b, Tl-2b, Tl-3b and Tl-7b. The part of the column shallower than the bending failure point tilted counterclockwise in Tl-1b, Tl-2b and Tl-3b; however, Tl-7b tilted clockwise. The other columns, Tl-4b, Tl-5b and Tl-6b, tilted counterclockwise without any column failure. A similar phenomenon can be seen in Case 11, as shown in Fig. 11(f). It is interesting to note that the location of bending failure was much deeper in Th-1b and Th-2b than in Th-7b. Again, the location of the breaking failure was much deeper in Case 11 than in Case 8.

Based on the above, the DM columns do not fail simultaneously but fail one by one by bending failure mode. It is of interest to note that the location of the bending failure is shallower in the low strength column than the high strength column, and shallower in the rear side columns than the front side columns.

**Ground Deformation**

The ground deformation obtained after the ground failure is shown in Fig. 12 for the unimproved ground (Case 1) and the improved ground with Th-column (Case 10). The data was obtained by digitizing the coordinates of the target markers placed on the side surface of the model ground. In the case of the unimproved ground (Case 1), a sort of slip circle deformation can be clearly seen at a shallow depth close to the embankment slope. After the ground failure, a large horizontal ground displacement is typically observed with further embankment loading. In Case 10, a relatively large ground defor-
mation can be seen at a shallow and mid depth. With further embankment loading, the ground displacement increased but no slip circle failure took place. The ground deformation in the other improved ground is very similar to that of Case 10, where no slip circle failure was observed.

**Horizontal Displacement Distribution**

In order to investigate the ground deformation in detail, the horizontal displacement distribution with depth measured at the toe of the embankment slope is shown in Fig. 13 for the unimproved and improved ground, in which the horizontal displacement at various loading stages is plotted. In the unimproved ground (Fig. 13(a)), a relatively small displacement took place at a shallow depth at \( p_e = 10.8 \text{ kN/m}^2 \). After that, an extremely large horizontal deformation took place with further embankment loading, especially at a shallow depth, while a small displacement took place at a deep layer. The difference in the magnitude of horizontal displacement clearly indicates that the ground failed with a slip circle failure pattern passing through the shallow layer.

In Case 7, the improved ground, horizontal displacement at the toe of the embankment slope, corresponding to the forefront column, develops with increasing \( p_e \), but its distribution is almost linear throughout the embankment loading. In the figure, the location of the forefront column failure is also plotted as arrows. The horizontal displacement distribution is almost a linear shape even after the column fails. This phenomenon can also be seen in Case 10 (Fig. 13(c)), the improved ground with Th-column. The horizontal displacement at the bottom of the column is negligible. A similar distribution can be seen in all the improved ground. As the front surface of the ground on which the target markers were placed corresponds to the intermediate point between the columns, the clay between the columns does not squeeze through but instead displaces together with the columns. These observations indicate that the improved area does not fail with a sliding failure pattern even after the columns fail, irrespective of the improvement width.

It can be concluded from Figs. 12 and 13 that DM columns have the effect of changing the ground failure mode from slip circle failure to collapse failure.

**Vertical Stresses on Top of Columns**

Figures 14(a) and 14(b) show the vertical stress increment, which was measured at the clay surface between the columns in Cases 7 and 10. It can be seen in the figures that the vertical stress at the clay surface monotonically increases with increasing \( p_e \), and the magnitude of increment is almost of the same order, irrespective of column strength.

Figures 15(a) and 15(b) show the vertical stress increment at the top of the c-line columns in Cases 7 and 10. The stress increment of the forefront and the second row column was of a quite a small level during the loading, which could be due to that the embankment height at the position did not increase so much and the columns’ top displaced horizontally beyond out of embankment. The comparatively small increment brings quite low stress concentration ratio which will discussed in Fig. 16. In the
figure, the arrows beside the curves indicate the point in time of the column failure. In Case 7, the vertical stress increased with increasing $p_e$ and peaked in value at different embankment pressures for TI-1c, TI-2c and TI-3c. As the embankment loading was terminated at a relatively low embankment pressure, TI-4c and TI-5c did not fail during the loading, while TI-4b and TI-5b failed at $p_e$ of about 43.9 kN/m². A similar phenomenon can be seen in Case 10, where the vertical stress increment, $\sigma'_v$, value at the top of the columns increased with increasing $p_e$ and peaked at different embankment pressures depending upon the column location. The time of the peak stress does not coincide with the point in time of column failure, but instead the columns failed after the vertical stress peaked.

A number of studies have been conducted on vertical stress on DM columns or sand compaction piles. Almost all the tests concluded that the decrease in vertical stress was triggered by the failure of DM columns or sand piles. However, the present study suggests that this conclusion might be incorrect.

Compared with the column strength, the vertical stress ratio at column failure is quite low, less than 0.3 for Case 7 and less than 0.1 for Case 10, which means that column failure was induced by bending moment rather than compressive stress.

**Stress Concentration Ratio**

It is well known that the embankment pressure concentrates on the column due to its higher stiffness. Figures 16(a) and 16(b) show the stress concentration ratio, $n$, in Cases 7 and 10, which is defined by the ratio of vertical stress increment at the top of the column against that at the clay surface between the columns. In Case 7, the $n$ value temporally decreases at the first loading step but increases with increasing $p_e$ and peaks in TI-2c and TI-3c columns. The $n$ value of TI-1c to TI-3c was quite a low value because the stress increment at the top of column was quite low, as shown in Fig. 15. In TI-4c and TI-5c columns, the $n$ value continues to increase with increasing $p_e$ and has no peak. Although the $n$ value varies in each column and embankment loading level, a high value is obtained at the rear side columns. In Case 10, the $n$ value increases with increasing $p_e$ and peaks at all the columns. The value and timing of the peak vary widely depending upon the column. Again, a high value is obtained at the rear side columns.

The magnitude of $n$ is a low value, less than about 2.5, irrespective of column strength. A similar phenomenon was observed in the other cases. The $n$ value is usually obtained by direct measurement of the stress, or back calculation of ground settlement in the field. Accumulated data shows the $n$ value ranging from 10 to 20 (CDIT, 2002), which is considerably higher than in this study.
Bending Moment Distribution of Column

The bending moment in the columns was measured in Cases 2 to 5, in which acrylic pipes were used as the model columns. The moment at three loading steps is plotted in Figs. 17(a) to 17(c). In Fig. 17(a), the measured moments are plotted for Case 2 corresponding to (i) before the forefront column failure, (ii) at the forefront column failure and (iii) at the rearmost column failure in Case 9. The moments in Figs. 17(b) and 17(c) are measured in Cases 3 and 4 corresponding to the three steps in Cases 10 and 11, respectively.

In the improved ground with 3 column rows (Fig. 17(a)), the moment distribution before column failure, Fig. 17(a)(i), increases with depth and shows a maximum value at a depth of \(-14\) cm, irrespective of column location. A similar phenomenon can be seen at the forefront column failure, Fig. 17(a)(ii). However, it is interesting that the largest bending moment developed in the rearmost column instead of in the forefront column that failed. At the rearmost column failure, Fig. 17(a)(iii), the negative bending moment developed at a shallow layer in the rearmost column. In the improved ground with 5 column rows, Fig. 17(b), the moment distribution before column failure, Fig. 17(b)(i), increases gradually with depth and shows a maximum value at a depth of \(-14\) cm, which is quite similar to Fig. 17(a)(i). At the column failure, Fig. 17(b)(ii), the bending moment in the forefront column was not the largest value even when the column failed. At the rearmost column failure, Fig. 17(b)(iii), a very large negative bending moment developed in the rearmost column at a depth of \(-6\) cm. In the improved ground with 7 column rows, Fig. 17(c), a large moment developed in the two forefront columns before column failure, Fig. 17(c)(i). At the forefront column failure, Fig. 17(c)(ii), a large positive moment developed in the three forefront columns at a deep layer, while a large negative moment developed in the rearmost column at a shallow layer. Again, it is of interest to note that the moment developed in the forefront column was not the largest even when the column failed. At the rearmost column failure, Fig. 17(c)(iii), the moment in the three forefront columns increased and a very large negative bending moment developed in the rearmost column.

The above results demonstrate that the column failure can not be estimated by the magnitude of bending moment alone, but the moment distribution roughly corresponds to the column failure phenomenon especially in the forefront and rearmost columns.

Vertical Stress/Bending Moment Relationship

In order to investigate the failure criteria in detail, the relationship between bending moment and vertical stress in the model columns is plotted in Fig. 18 for the improved ground with 3, 5 and 7 column rows. In the figure, the bending moment measured in A-columns (Cases 2 to 4) is plotted on the horizontal axis. In the figure, the measured moment is divided by the inertia to obtain the stress at the outer surface of column, \(\sigma\), and then normalized with respect to the column strength, \(q_u\). The vertical stress measured in Th-columns (Case 9 to 11) is plotted on the vertical axis, in which the vertical stress is normalized with respect to the column strength, \(q_u\). The point in time of column failure is marked by an arrow in the figures to the corresponding stress path. The test conditions in these two test series are similar except for the column material: acrylic in Cases 2 to 4 and cement treated soil in Cases 9 to 11. Of course, the moment distribution might be influenced by the column failure in Cases 9 to 11. However, these trials can be useful for qualitative understanding of the failure criteria of column. In the figure, two failure criteria are indicated by solid and broken lines. The solid line indicates that the compressive stress at the outer surface of the column induced by the combination of vertical stress and bending moment reaches the compressive strength, in which plus and negative value mean the counterclockwise and clockwise movements respectively. The broken line indicates that the induced tensile stress at the outer surface of column reaches the column tensile strength, \(\sigma_t\).
low vertical stress with positive bending moment. In the rearmost column, however, the vertical stress increases with a very small increase in moment at the early stage of loading, followed by a large increase in negative moment (clockwise direction) with decreasing vertical stress. The column failed under the combination of negative moment with vertical stress. The stress conditions under which the columns failed are close to the tensile strength criterion, which indicates that tensile stress might induce column failure. In the improved ground with 7 column rows (Fig. 18(c)), the two forefront columns show a stress path toward the positive moment and failure under the combination of relatively low vertical stress with positive moment, similar to the other cases. The rearmost column, Th-7, failed under the combination of a very large negative bending moment with relatively large vertical stress. The stress conditions under which Th-7 failed are beyond the tensile strength criterion, which indicates that compressive stress might induce column failure. The other columns, Th-3, Th-4, Th-5 and Th-6, show a positive moment with vertical stress, but did not fail in the test.
DISCUSSION

The following discussion on the evaluation of unimproved and improved grounds are described in the prototype scale instead of the model scale.

Evaluation of Stability for Unimproved Ground

The stability of the unimproved ground (Case 1) was evaluated by Fellenius slip circle analysis and FEM analysis, with a calculated embankment pressure at ground failure, \( p_{ef} \), of 15.7 kN/m\(^2\) and 12.0 kN/m\(^2\), respectively (Kitazume and Maruyama, 2006).

Slip Circle Failure for Improved Ground

The internal stability of DM improved ground was evaluated by the current design method first (PWRC, 2004), in which slip circle analysis with shear strength of the columns was performed. In the calculation, undrained shear strength is assumed as \( q_u \) and fully mobilized simultaneously in all the columns. In the slip circle analysis, the embankment pressure at ground failure, \( p_{ef,slip} \), is calculated by changing the embankment height until the safety factor becomes unity. The calculations are plotted in Fig. 19 along improvement width, \( D \), for various column strengths. The \( p_{ef,slip} \) increases with increasing \( D \), irrespective of \( q_u \). In the figure, the model test results and the calculations with \( q_u \) values corresponding to the model tests are plotted together. It is found that the calculation overestimates the test results by about 3 to 5 times, especially in the case of high strength column, Cases 9 to 11.

Figure 20 shows the relationship between the maximum depth of the critical slip circle, \( z_{f,slip} \), and improvement width, \( D \), which is calculated in the slip circle analysis. The \( z_{f,slip} \) value increases gradually with increasing \( D \), irrespective of column strength, but is much larger in higher column strength. In the figure, \( z_{f,slip} \) values of the model tests are also plotted. Although the measured value in Case 9 differs slightly from the norm, it increases with increasing \( D \) and column strength. The calculations show larger values compared to the test results.

As discussed above, the current design based on the slip circle analysis can not reasonably evaluate the \( p_{ef,slip} \) and \( z_{f,slip} \) values.

Shear Failure for Improved Ground

The internal stability of DM improved ground is evaluated by a simple calculation, in which the shear failure mode is assumed, as shown in Fig. 21. Full mobilization of DM column shear strength is assumed in the calculation. The formulation for the shear failure mode is expressed as Eqs. (1) to (6) for assumed depth of the shear failure plane, \( z \), which is based on the load equilibrium of active and passive earth pressures acting
on the side boundaries of the improved area and the shear strength mobilizing along the clay ground and DM columns. Rankin’s theory of ultimate active and passive earth pressures are adopted in the calculation.

\[ F_S = \frac{P_{pe} + P_{el} + P_{ec}}{P_{ae} + P_{ac}} \]  

(1)

\[ P_{ae} = g \cdot H_e \cdot \tan^2 \left( \frac{\pi}{4} - \frac{\phi_e}{2} \right) \cdot \frac{H_e}{2} \]  

(2)

\[ P_{ac} = (2 \cdot g \cdot H_e - 2 \cdot (2 \cdot W + W + k \cdot z) + g \cdot z) \cdot \frac{z}{2} \]  

(3)

\[ P_{pc} = (g \cdot z + 2 \cdot (2 \cdot c_0 + k \cdot z)) \cdot \frac{z}{2} \]  

(4)

\[ F_{el} = \frac{q_u}{2} \cdot a_s \cdot D \]  

(5)

\[ F_{ec} = (c_0 + k \cdot z) \cdot (1 - a_s) \cdot D \]  

(6)

where

- \( F_c \): Cohesive strength of clay along failure plane (kN/m²)
- \( F_{el} \): Shear strength of DM column along failure plane (kN/m²)
- \( F_s \): Safety factor
- \( P_{ae} \): Active earth pressure of clay ground (kN/m²)
- \( P_{ac} \): Active earth pressure of embankment (kN/m²)
- \( P_{pc} \): Passive earth pressure of clay ground (kN/m²)
- \( a_s \): Improvement area ratio
- \( c_0 \): Undrained shear strength at ground surface (kN/m²)
- \( H_e \): Thickness of clay ground (m)
- \( H_p \): Height of embankment (m)
- \( q_u \): Unconfined compressive strength of DM column (kN/m²)
- \( z \): Assumed depth of shear failure plane (m)
- \( k \): Undrained shear strength increasing ratio with depth (kN/m²)
- \( \gamma_e \): Unit weight of clay ground (kN/m³)
- \( \gamma_p \): Unit weight of embankment (kN/m³)
- \( \phi_e \): Internal friction angle of embankment (degree)

After substituting Eqs. (2) to (6) into Eq. (1), the following quadratic equation is obtained with respect to the embankment height, \( H_e \), for assumed \( z \). As the magnitude of the left-hand terms is negative when \( H_e = 0 \), two real number solutions are always obtained while the meaningful solution is the positive one.

\[ \frac{\gamma_e}{2} \cdot \tan^2 \left( \frac{\pi}{4} - \frac{\phi_e}{2} \right) \cdot H_e^2 + \gamma_e \cdot z \cdot H_e \]

Figure 22 shows the relationship between \( z \) and \( p_{ef,\text{shear}} \) for \( D = 7.7 \) m and \( a_s = 0.28 \). In the figure, the relationship with various column strengths is plotted. In the case of \( q_u = 50 \) kN/m², the \( p_e \) value changes very slightly but shows a minimum value at \( z = 3 \) m. The relationship shows a concave shape in the case where \( q_u \) is lower than about 500 kN/m². However, when \( q_u \) equals or exceeds 500 kN/m², \( p_e \) monotonically decreases with \( z \). The \( p_{ef,\text{shear}} \) value, defined as a minimum value and shown by an arrow in the figure, increases with increasing \( q_u \) and \( z \). A series of similar calculations was carried out for various \( D \) values, and the relationship between \( z_{ef,\text{shear}} \) and \( D \) is shown in Fig. 23 for various \( q_u \) values. The \( z_{ef,\text{shear}} \) value increases monotonically with increasing \( D \), and with increasing \( q_u \), indicating that shear failure takes place at the deep depth as \( D \) and/or \( q_u \) increases. The \( z_{ef,\text{shear}} \) value increases with increasing \( q_u \), and reaches 10 m when \( q_u \) equals or exceeds 500 kN/m², which means that no column shear failure takes place. In the figure, the model test results are plotted together. The calculations
give much larger $z_{f,shear}$ values compared to the tests.

The $p_{ef,shear}$ is defined as the minimum value for each case, and is shown along $D$ in Fig. 24. The figure shows that $p_{ef,shear}$ increases with increasing $D$ and/or $q_u$. The model test results are also plotted in the figure. In comparison with the model test results, the calculated $p_{ef,shear}$ values are considerably higher. The overestimation is quite dominant as $D$ increases.

In order to investigate the cause of overestimation in detail, the resistance force components in the calculation are shown in Fig. 25. The passive earth pressure component of resistance force, $P_{pc}$, increases with increasing $q_u$, which is due to the increase of $z_{f,shear}$. When the column strength becomes 1300 kN/m$^2$ (Cases 9 to 11), $P_{pc}$ becomes constant, irrespective of $D$. The column strength component, $F_{rf}$, has a dominant role in the entire resistance load throughout $D$. Its degree increases with increasing $q_u$, and reaches about 65% in Case 11. According to Fig. 24, the column strength should be underestimated to the unrealistic value of 1/8 to 1/10 to evaluate the test results accurately.

The magnitude and shape of the passive earth pressure distribution are greatly influenced by many factors such as adhesion of the retaining wall, movement of the wall, etc., which have not yet been clarified despite numerous research efforts made over the years. Figures 26(a) and (b) show the effect of the mobilization degree of passive earth pressure on $p_{ef,shear}$ and $z_{f,shear}$. In Fig. 26(a), the $p_{ef,shear}$ value decreases with decreasing mobilization degree, but still overestimates the model tests even when the degree decreases to 25% of the initial value. As the mobilization degree decreases, the $z_{f,shear}$ value increases due to the increasing relative column strength, as shown in Fig. 26(b). This causes further discrepancy with the model tests.

According to the parametric calculations, the overestimation by the shear failure mode cannot be explained by the accuracy of soil parameters, but should be explained by the difference of failure pattern: shear failure pattern instead of bending failure pattern is assumed in the current design method.

**Bending Failure for Improved Ground**

Here, a simple stability calculation is proposed. In the calculation, all the DM columns are assumed to fail simultaneously in bending failure mode and the improved area above a failure plane is assumed to deform as a simple shear, schematically shown in Fig. 27. However, the assumption of full mobilization of bending strength does not correspond to the model test results where the columns fail one by one. As described before, the DM columns are subjected to not only the bending moment but also the axial stress. In the calculation, the columns are assumed to fail when the induced tensile stress reaches the ultimate bending strength, $\sigma_b = \alpha q_u$, as shown in Fig. 28, where $\alpha$ value is assumed as 0.28 according to Fig. 6. For the calculation, the moment equilibrium at the assumed failure plane, $z$, is analyzed as follows:

The driving moments by the active earth pressure of the
embankment, $M_{ae}$, and of the clay ground, $M_{ac}$, are expressed as Eqs. (9) and (10), respectively, where Rankin’s theory on earth pressure is assumed.

$$M_{ae} = \gamma_e \cdot H_e \cdot \tan \left( \frac{\pi}{4} - \frac{\phi_e}{2} \right) \cdot \frac{H_e^2 + 3 \cdot H_e \cdot z}{6} \tag{9}$$

$$M_{ac} = \frac{z^2}{6} \cdot (3 \cdot \gamma_e \cdot H_e + \gamma_c \cdot z - 6 \cdot c_{so} - 2 \cdot k \cdot z) \tag{10}$$

Similar to the shear failure calculation, the embankment shape is assumed as a trapezoid extending from the foremost to the rearmost DM column as shown in Fig. 27, for ease of parametric calculations. The resistance moment per unit breadth by the adhesion mobilizing on the side surface of DM columns, $M_{re}$, by the weight of DM columns, $M_{rt}$, by the weight of embankment on DM columns, $M_{re}$, by the shear strength of clay between DM columns, $M_{sc}$, by the passive earth pressure of clay ground, $M_{pc}$, and by the bending failure of DM columns, $M_{pb}$, are expressed as Eqs. (11) to (16), respectively.

$$M_{re} = B^3 \cdot z \cdot \frac{2 \cdot c_{so} + k \cdot z}{2} \cdot \frac{N_c}{S} \cdot \frac{1}{S} \tag{11}$$

$$M_{re} = B^3 \cdot z \cdot \frac{2 \cdot c_{so} + k \cdot z}{2} \cdot \frac{N_c}{S} \cdot \frac{1}{S} \tag{12}$$

$$M_{rt} = \frac{\pi}{6} \cdot \frac{B^3 \cdot H_e \cdot \gamma_e \cdot \left( \frac{1}{(n-1) \cdot a_t} \right)}{1} \cdot \frac{N_c}{S} \cdot \frac{1}{S} \tag{13}$$

$$M_{sc} = S \cdot (1 - a_b) \cdot \frac{2 \cdot c_{so} + k \cdot z}{2} \cdot \frac{(N-1) \cdot z}{2} \tag{14}$$

$$M_{pc} = \frac{z^2}{6} \cdot (\gamma_c \cdot z + 4 \cdot c_{so} + 2 \cdot k \cdot z) \tag{15}$$

$$M_{pb} = \frac{\pi}{32} \cdot B^3 \cdot \sigma_c \cdot N_c \cdot \frac{1}{S} \tag{16}$$

where

$M_{ae}$: Driving moment by active earth pressure of clay ground (kN $\times$ m)

$M_{ac}$: Driving moment by active earth pressure of embankment (kN $\times$ m)

$M_{re}$: Resistance moment by adhesion on side surface of DM columns (kN $\times$ m)

$M_{rt}$: Resistance moment by weight of embankment (kN $\times$ m)

$M_{re}$: Resistance moment by weight of DM columns (kN $\times$ m)

$M_{pc}$: Resistance moment by shear strength of clay between DM columns (kN $\times$ m)

$M_{pb}$: Resistance moment by passive earth pressure of clay ground (kN $\times$ m)

According to the moment equilibrium at the failure plane, the following equation can be satisfied:

$$M_{ae} + M_{ac} = M_{re} + M_{rt} + M_{re} + M_{sc} + M_{pc} + M_{pb} \tag{17}$$

After substituting Eqs. (11) to (16) into Eq. (17) and expanding the equation, the following cubic equation is obtained with respect to the embankment height, $H_e$.

$$- \frac{\gamma_e}{6} \cdot \tan \left( \frac{\pi}{4} - \frac{\phi_e}{2} \right) \cdot \frac{H_e^2 - \gamma_e}{2} \cdot \frac{\tan \left( \frac{\pi}{4} - \frac{\phi_e}{2} \right)}{S} \cdot z \cdot H_e^2 + D \cdot \left( \frac{4}{\pi} \cdot a_s \cdot \frac{2 \cdot c_{so} + k \cdot z}{2} + \frac{1}{2} \cdot a_s \cdot B \cdot \gamma_s \cdot \mu_s \cdot \frac{1}{S} \right) - \left( 1 - a_s \right) \cdot \frac{2 \cdot c_{so} + k \cdot z}{2} - \frac{z^2}{2} \cdot \gamma_e \cdot \frac{1}{S} \cdot H_e$$

$$+ \left\{ a_s \cdot B \cdot z \cdot \gamma_s \cdot \frac{1}{S} + \frac{z^2}{3} \cdot (6 \cdot c_{so} + 2 \cdot k \cdot z) + \frac{\pi}{32} \cdot B^3 \cdot \alpha \cdot q_s \cdot N_c \cdot \frac{1}{S} \right\} = 0 \tag{18}$$

Three solutions, either three real numbers or one real and two imaginary numbers, are obtained by Cardano’s formula. The meaningful solution should be a real number and positive value. As there are many variables in the equation, a solution, $H_{el,bending}$, is numerically calculated for specific ground conditions and assumed bending failure plane, $z$. The embankment pressure at ground failure, $p_{el,bending}$, is similarly calculated by Eq. (8).
Figure 29 shows the relationship between assumed depth of bending failure plane, \( z \), and \( p_e \) for \( D = 7.7 \text{ m} \) and \( a_s = 0.28 \). The relationship for various column strengths is plotted in the figure and shows a concave shape, irrespective of the \( q_u \) value. The \( z_{f,bending} \), giving \( p_{ef,bending} \), as shown by an arrow, increases slightly with increasing \( q_u \). It can be seen that \( p_{ef,bending} \) also increases slightly with increasing \( q_u \).

A series of calculations was carried out for different improvement widths and column strengths, and the relationship between \( D \) and \( z_{f,bending} \) is shown in Fig. 30 for various \( q_u \) values. The \( z_f \) value increases monotonically with increasing \( D \), and with increasing \( q_u \). However, the effect of \( q_u \) is not so dominant compared to that in the shear failure pattern in Fig. 23. In the figure, the model test results are also plotted. The calculation gives a reasonable estimation of the depth of failure plane, slightly overestimated compared to the model tests for Cases 6 to 8, but underestimated for Cases 9 to 11.

The \( p_{ef,bending} \) value, shown along \( D \) in Fig. 31, increases with increasing \( D \) and \( q_u \). However, the effect of \( q_u \) is relatively small. The model test results are also plotted in the figure. The calculations give a reasonable estimation compared to the model tests.

The resistance moment components for the bending failure mode, shown in Fig. 32, are calculated by the proposed calculation. The passive earth pressure component, \( M_{pc} \), has a dominant role in the entire resistance load throughout \( D \). Its degree increases with decreasing \( D \) and with increasing \( q_u \). The component of the clay strength between the columns, \( M_{sc} \), also has a dominant role. However, the column strength component, \( M_{pb} \), has a relatively small role of about 10 to 15% of the whole resistance, which is quite a different phenomenon from the shear failure pattern as shown in Fig. 25. This indicates that the accuracy of evaluating \( p_{ef,bending} \) is dominantly governed by the accuracy of estimating the passive earth pressure.

The effect of passive earth pressure on \( p_{ef,bending} \) is studied next. Figures 33 and 34 show the effect of the passive earth pressure mobilization degree on \( p_{ef,bending} \) and \( z_{f,bending} \) for the improved ground with TI- and Th-columns, respectively. In the calculation, the mobilization degree is changed to 75\%, 50\%, and 25\% while its distribution shape is kept constant as a triangle.

In can be seen in Figs. 33(a) and 34(a) that the \( p_{ef,bending} \) value decreases at about the same magnitude with decreasing the mobilization degree. In the case of TI-column, Fig. 33(a), the calculated \( p_{ef,bending} \) value can be reasonably coincided with the model tests when the mobilization degree is about 25 to 75\%. In Fig. 33(b), the relationship between \( D \) and \( z_{f,bending} \) is plotted, showing that the calculated \( z_{f,bending} \) increases with decreasing passive earth pressure mobilization degree. A mobilization degree of about 100\% gives a reasonable estimation compared to the model tests throughout \( D \). In the case of
Th-column, Fig. 34(a), on the other hand, the calculation underestimates the test data even the mobilization degree of 100%. The calculated $z\_f,\_bending$ for Th-column, in Fig. 34(b) shows the mobilization degree of about 50 to 100% gives a reasonable estimation to the model tests.

**Effect of Improvement Area Ratio**

Here, the effect of improvement area ratio on the stability is addressed. Figure 35 shows the relationship between $p\_e\_f,\_bending$ and $z\_f,\_bending$ and $D$, which is calculated for various column strengths. In can be seen that $p\_e\_f,\_bending$ increases monotonically with increasing $D$, irrespective of $a_s$. The magnitude of $p\_e\_f,\_bending$ in $a_s=0.5$ is about 24% higher than that in $a_s=0.28$. The effect on $z\_f,\_bending$ (Fig. 35(b)), is not as large: the magnitude of $z\_f,\_bending$ in $a_s=0.5$ is about 20% higher than that in $a_s=0.28$.

**Stress Concentration Ratio**

The calculated $p\_e\_f,\_bending$ and $z\_f,\_bending$ values for $n=2$ and 5 are plotted in Fig. 36 along $D$. The $p\_e\_f,\_bending$ and $z\_f,\_bending$ values increase with increasing stress concentration ratio, $n$, but the effect is quite small.

**Effect of DM Column Diameter**

The effect of the DM column diameter, $B$, on internal stability is addressed in this section. Figure 37(a) shows the relationship between $p\_e\_f,\_bending$ and $D$ for $q_u=500$ kN/m² with various $B$. The $p\_e\_f,\_bending$ value increases almost
linearly with increasing $D$, irrespective of $B$. Comparing the effect of column strength as shown in Fig. 31, the improvement effect of $B$ is greater on $P_{ef,bending}$ than the effect of column strength. However, the embankment pressure increases more rapidly with increasing $B$. According to Eqs. (11) to (16), $M_{rc}$ increases with the power of two and $M_{re}$, $M_{re}$, and $M_{pb}$ increase with the power of three with increasing $B$. These increases in the resistance moment bring about the embankment pressure increase with increasing DM column diameter. By increasing the column diameter, the depth of failure plane, $z_{f,bending}$, increases, as shown in Fig. 37(b).

As the DM column diameter is dependent upon the machine capacity and is about 1.0–1.5 m in Japan (CDIT, 2002), the calculation for a diameter exceeding 2 m is not realistic. However, it becomes realistic when the columns are overlapped to create treated soil mass having a relatively large sectional area. According to literatures (e.g., Rathmayer, 2000), honeycomb type and column wall type improved ground are proposed for improving the stability of embankment slope, where DM columns are overlapped. The calculation results confirm that such improved ground can considerably improve the internal stability.

**CONCLUSIONS**

The failure pattern of group column type DM improved ground subjected to embankment loading was investigated through a series of centrifuge model tests and a simple calculation. The major conclusions derived in this study are as follows:

1) The embankment pressure monotonically increases with increasing ground displacement without peak-2

2) The embankment pressure at ground failure, which is defined as the forefront column failure, increases gradually with increasing improvement width.

3) The DM columns do not fail simultaneously but instead fail one by one in sequence from the forefront column toward the rearmost column in the case of small improvement width. When the improvement width becomes large, the forefront column fails first and then the second and third row columns fail. However, the rearmost column then fails due to large ground settlement.

4) The current design method cannot reasonably evaluate the embankment pressure and the depth of failure plane at ground failure of the model test results because a shear failure mode is assumed instead of a bending failure mode for the columns. The overestimation can not be explained by estimating the accuracy of soil parameters.

5) A simple calculation based on the bending failure mode of the columns has relatively high applicability for evaluating the internal stability of the group column type improved ground.

6) The improvement area ratio has a dominant effect on
the internal stability of the improved ground. The increasing DM column diameter has the effect of improving the internal stability of improved ground. The overlapping of DM columns is effective for increasing the internal stability.

7) The importance of simulating a suitable failure pattern of improved ground is demonstrated for accurately evaluating the internal stability.

REFERENCES


