3-D NUMERICAL ANALYSIS OF PARTIAL FLOATING SHEET-PILE METHOD AS COUNTERMEASURE FOR LIQUEFACTION

Kakuta FUJIWARA¹, Nanase OGAWA² and Kentaro NAKAI³

¹ Member of JSCE, Assistant Professor, Dept. of Civil Eng., Tokai University
(4-1-1, Kitakaname, Hiratsuka-shi, Kanagawa 259-1292, Japan)
E-mail: fujiwara.kakuta.s@tokai.ac.jp (Corresponding Author)

² Member of JSCE, IPA Support Department., GIKEN LTD.
(5F, Sanwa Konan Bldg, 2-4-3 Konan, Minato-ku, Tokyo 108-0075, Japan)
E-mail: ogawa@giken.com

³ Member of JSCE, Associate Professor, Dept. of Civil Eng., Nagoya University
(Furo-cho, Chikusa-ku, Nagoya 464-8601, Japan)
E-mail: nakai@civil.nagoya-u.ac.jp

The Partial Floating Sheet-pile (PFS) method is a method used to install sheet-piles near the toe of an embankment to inhibit the settling of the embankment built on soft clay ground. This method combines partially floating sheet-piles and end-bearing sheet-piles. Originally, the PFS method was developed as an anti-subsidence countermeasure by the weight of a river embankment built on soft clay ground in a residential area. The PFS method could be effective against soil deformation caused by consolidation and liquefaction during the 2016 Kumamoto earthquake. However, quantitative discussions are still needed to clarify its effectiveness against liquefaction. Therefore, the authors carried out numerical analyses using LIQCA3D17 to evaluate the effectiveness of the PFS method quantitatively. As a result, the interval between the end-bearing sheet-piles was found to be an important factor for embankment settlement, and the installation of the end-bearing sheet-piles alternately could be reasonable.

Key Words: numerical analysis, liquefaction, countermeasure, PFS, three-dimension

1. INTRODUCTION

The 2016 Kumamoto earthquake, which had a magnitude of 7 on the Richter scale, caused severe seismic damage to river embankments in Japan’s Kumamoto region⁴. Another powerful earthquake, such as a possible Nankai Trough Earthquake, could occur in the near future, and the resulting strong motions could damage river embankments.

Several countermeasures such as soil improvement, soil compaction, gravel drain, and others have been proposed to protect embankments during an earthquake. The steel sheet-pile method is one of these countermeasures. It has long been used as a temporary measure in construction work, but in recent years it has also been used as a permanent measure alongside port and harbour structures.

The sheet-piles are usually installed into a supporting layer at the toe of an embankment, as a countermeasure against earthquakes⁵.

It has been reported that the deformation of an embankment built on sandy ground has been reduced because the sheet-piles prevented the lateral flow of the
liquefied sand foundation.

The PFS method is one of the methods used for constructing sheet-piles near the toe of an embankment. It is similar to sheet-pile methods used previously; however, in this method, floating sheet-piles are incorporated into the structure, as illustrated in Fig.1. The PFS method utilizes a structure that combines floating sheet-piles and end-bearing sheet-piles. Such a structure can easily be realized in terms of cost effectiveness and feasibility of ground construction.

Originally, the PFS method was developed as an anti-subsidence countermeasure to support the weight of a river embankment built on soft clay ground in a residential area. The purpose of the method is to inhibit subsidence in a residential area due to the weight of a river embankment. In the case of the 2016 Kumamoto Earthquake, damage was suppressed on a river embankment where the PFS method had been applied. As the ground which included layers of sandy soil liquefied by the motion, the PFS method could be effective against soil deformation due to liquefaction as well as consolidation⁶. This result indicates that in addition to original expectations, the PFS method could be applied for more comprehensive purposes and conditions, such as anti-settlement countermeasure for an embankment on a ground that includes sand layers.

The sheet-pile method in which all sheet-piles are installed in a supporting layer along the toes of an embankment on sand ground has been proposed. It has been confirmed that the settlement of an embankment is reduced by the sheet-piles during liquefaction caused by an earthquake⁵.

Based on these facts, the authors investigated the effectiveness of the PFS method in inhibiting the settlement of an embankment built on a ground including sand layers. As not all the sheet-piles in a PFS structure are installed in a supporting layer, liquefied soil could pass through underneath the floating sheet-piles. In that case, the PFS method would be less effective than the method where all end-bearing sheet-piles were dispensed with. Further analysis is required to clarify the effectiveness of the PFS method during liquefaction.

Therefore, the authors carried out numerical analyses using LIQCA3D17⁶,⁷ to obtain a quantitative evaluation of the effectiveness of the PFS method. For simplicity, only sand layers were considered in the analyses, as will be discussed later. As the PFS method uses different lengths of the sheet-piles, soil deformation is complex and three-dimensional. Accordingly, the authors prepared a three-dimensional analytical model of these analyses.

2. ANALYTICAL CONDITIONS

1) Outline of the numerical code LIQCA

The numerical analyses for the PFS structure were conducted using LIQCA3D17⁶,⁷. LIQCA is a numerical code for analyzing the liquefied ground deformation during an earthquake. This section includes a discussion of the principles on which LIQCA is based and examples of its application.

a) Principle on which the code is based

The governing equations describing the coupling between the soil skeleton and the pore water pressure were formulated on the basis of the two-phase mixture theory⁸, and a u-p formulation (where u, the displacement of the solid-phase soil skeleton and p, the pore water pressure, are the two unknowns) was adopted in the three-dimensional analysis.

The finite element method was employed for the spatial discretization of the equation of motion, whereas the finite difference method was employed for the spatial discretization of pore water pressure in the continuity equation.

b) Application examples

LIQCA has been used in numerical analyses of the behaviours of embankments, retaining walls and buildings, as well as in other constructions, under the liquefaction of foundation. The verification of LIQCA analyses has been conducted by comparing the LIQCA results with experimental results. It has been confirmed that the deformation of an embankment during liquefaction can be accurately reproduced by applying LIQCA-based analyses. In addition, the application of LIQCA-based analyses has accurately reproduced the effectiveness of the reinforcement of an embankment by sheet-piles⁹. Given these successes, the authors chose to use LIQCA for the numerical analysis of the deformation of an embankment during liquefaction.

2) Numerical models for the PFS structure

To perform the numerical analysis, parameters such as dimensions, soil parameters, and countermeasures need to be determined. The following subsections describe the values used in this study.

a) Models

The target structure in the numerical model was a plane field subjected to a vertical load representing the weight of an embankment, as depicted in Fig.2 (a). The imaginal embankment is also shown in Fig.2 (b). The height of the embankment was 5 m; the widths of the top and bottom edges were 6 m and 26 m, respectively; the inclination of the slope was 1:2; and the density of the soil was 1.8 t/m³. The half embankment was considered in the numerical model. For simplicity, the trapezoid shape of the embankment was ignored. Therefore, all vertical loads were
assumed to equate to the weight divided by the area of the bottom surface of the embankment.

The coordinate system was set up with the y-axis extending along the length of the embankment, the x-axis perpendicular to the y-axis in the horizontal plane, and the z-axis pointing vertically upward. The x-z dimensions of the model were as follows: 40 m in the x-direction and 12 m in the z-direction. The dimension in the y-direction varied with each case. All the meshes except joint elements were modeled as cubes and cuboids.

The locations of target points D, P1, and P2 (to be discussed later) are identified in Fig.2 (c), which depicts a vertical cross section. The point D indicates the vertical displacement, while points P1 and P2 indicate the excess pore water pressure.

**b) Soil conditions**

It was assumed that the plane field consisted of a liquefiable layer with a relative density of 40% and a thickness of 8 m, and a supporting layer with a relative density of 90% and a thickness of 4 m, which were modelled as cyclic elasto-plastic model and R-O model, respectively.

Before numerical analyses of the target structure could proceed, the values of various soil parameters needed to be selected. These values should represent the structure’s tendency to deform and the degree of liquefaction of the meshes. Soil parameters were determined from the results of an undrained cyclic triaxial compression test. In this study, the authors carried out simulations on only one mesh, varying the values of the soil parameters and comparing the results with those of the compression tests, in terms of the stress–strain relation, the effective stress path, and the liquefaction resistance curve. Through trial

**Table 1 Parameters used in the numerical analyses.**

| Parameters used in the numerical analyses | (a) Soil | (b) Sheet-pile | (c) Joint element |
|-------------------------------------------|---------|---------------|------------------|
| **Numerical model** | Cyclic elasto-plastic model | R-O model | | |
| Permeability coefficient (m/s) | k | 2.2E-05 | 6.7E-06 | | |
| Initial void ratio | ε | 0.821 | 0.683 | | |
| Compression index | λ | 0.015 | - | | |
| Swelling index | s | 0.002 | - | | |
| Initial shear modulus ratio | Gv/hr | 1000 | - | | |
| Phase transformation stress ratio | Mg | 0.909 | - | | |
| Failure stress ratio | Mf | 1.122 | - | | |
| Hardening parameter | Bh | 7000 | - | | |
| Parameter of anisotropy | Cj | 2000 | - | | |
| Dilatancy parameter | Dp | 3 | - | | |
| Reference strain parameter | n | 3 | - | | |
| Poisson’s ratio | v | 0.02 | 0.32 | - | |
| Cohesion (kN/m²) | c | 0 | 0 | - | |
| Friction angle (rad) | θ | 0.68 | 0.5 | - | |
| Parameter of shear elastic modulus | α | - | 23665 | | |
| Parameter of R-O model | r | - | 0.3 | | |
| Spring stiffness in shear direction | ks | 2.0×10⁶ | kN/m² | | |
| Spring stiffness in perpendicular direction | kn | 2.0×10⁶ | kN/m² | | |
| Cohesion | c | 0 | kN/m² | | |
| Friction angle | φ | 15 | degree | | |

![Fig.2 The numerical model.](image)
and error, the authors were able to obtain simulated results that approximated the compression test results. The values of the soil parameters are summarized in Table 1(a). The stress–strain relation, effective stress path and liquefaction resistance curve for liquefiable sand are represented in Figs. 3, 4, and 5.

c) Countermeasures

The sheet-pile was installed along the y-direction at x = 13 m, which was the location of the toe of the imaginal embankment. The sheet-pile was modelled using cubic meshes with a Young’s rigidity of 7.3×10^7 kN/m² and a thickness of 200 mm, such that the flexural rigidity was equivalent to that of a 25H steel sheet-pile (50000 kNm² per unit with a width of 1 m), as listed in Table 1(b). The sheet-pile had a width of 1 m in the y-direction and was modelled as a linear elastic element with no yield. Joint elements were placed between the sides of the sheet-pile and the surrounding soil, taking friction and slip into consideration. The parameters of the joint elements are listed in Table 1(c).

d) Boundary conditions

Taking symmetry considerations into account, fixed boundary conditions were assigned at the locations of the side nodes in the x-direction, at x = 0 m and x = 40 m. In the y-direction, fixed boundary conditions were assigned at the locations of the side nodes at y = 0 m and at variable values ‘y’ m (with the value of ‘y’ selected for each case). In the z-direction, the bottom nodes were fixed at z = 0 m in all cases. The drainage boundary was set on the surface ground level, with the assumption that the water level corresponded with the surface level.

e) Analytical cases

The results for 14 analytical cases are listed in Table 2. The various analyses were conducted by changing the lengths of the floating sheet-piles and the intervals between the end-bearing sheet-piles. The length of a floating sheet-pile is defined as “L” and the interval between end-bearing sheet-piles is defined as “B,” as depicted in Fig.6.

For Case-1, no sheet-piles were set in the numerical model. For Case-2, the end-bearing sheet-piles were installed at the toe of the imaginal embankment, such that the tip of each pile reached the supporting layer. For Case-3 to Case-14, end-bearing sheet-piles and floating sheet-piles were installed together, thus constituting PFS structures.

f) Input motion

The input motion in the x-direction was a sine wave with a frequency of 3 Hz, a maximum acceleration of 9.0 m/s² and a duration of 10 s, as represented by the plot shown in Fig.7. The authors

![Fig.3](image1.png)  
(a) Simulation  
(b) Experiment  
Fig.3 The stress–strain relation.

![Fig.4](image2.png)  
(a) Simulation  
(b) Experiment  
Fig.4 The mean effective stress path.
applied an extremely large input motion to clarify the amount of deformation and the effectiveness of the sheet-piles. The Newmark method was used to solve for the responses of the soil and the structure. An incremental time interval of 0.001 s was used, while a value of 0.3025 was used for the coefficient β in the Newmark method and the value of γ was set at 0.6. We used a stiffness-proportional coefficient of 0.003 for Rayleigh damping.

3. RESULT AND DISCUSSIONS

(1) Overview of the ground deformation

For Case-1 and Case-2, the deformations with excess pore water pressure ratio, corresponding to the end of shaking, (t = 10 s) are represented by the heat maps shown in Figs.8 (a) and 8 (b). The excess pore water pressure ratio is obtained by dividing the excess pore water pressure by the initial effective stress. For Case-6, which is a representative example of the PFS method, the deformations on two cross sections (y = 0 m and y = 5.5 m) are represented by the heat maps shown in Figs.8 (c) and 8 (d) respectively. For Case-1, Case-2, and Case-6, the center-of-ground (extending from x = 0 m to x = 13 m) was settled by the vertical load. Although the increase in excess pore water pressure ratio at the center-of-ground was relatively small, due to the increase of effective stress by the vertical load, the ground without vertical load (extending from x = 13 m to x = 40 m) reached the point of liquefaction. The ground flowed laterally to the right, as depicted in these figures, due to the vertical load during liquefaction causing settlement of the center-of-ground.

For Case-1, Case-2, Case-3, and Case-6, the residual settlements of surface ground along the y-direction at x = 0 m are indicated in Fig.9. These lines are extended in accordance with the mirrored boundary conditions. The results for Case-1 and Case-2 are indicated with dotted lines for the sake of comparison. Small differences are seen along the y-direction, not only for Case-1 and Case-2, but also for Case-3 and Case-6. Small differences were also confirmed in the results for the other cases, which are omitted from this paper. Therefore, the authors focused on D as a representative point in the center-of-ground. The location of D (x = 0 m, y = 0 m, z = 12 m) is depicted in Figs.2 (a) and 2 (c).

(2) The effectiveness of the end-bearing sheet-piles

For Case-1 and Case-2, the time histories of the settlement at D are indicated in Fig.10. The settlements increase during motion (t = 0 ~ 10 s) and they gradually converge to constant values after the motion has ceased (t > 10 s). The residual settlements for Case-1 and Case-2 are 1.13 m and 0.27 m, respective-

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Table 2 Analytical cases.

| Case | B (m) | L (m) | Detail  |
|------|-------|-------|---------|
| 1    | -     | -     | No countermeasure |
| 2    | 0     | -     | All end bearing sheet-piles |
| 3    | 1     | 1     |       |
| 4    | 2.5   | 1     |       |
| 5    | 5     | 1     |       |
| 6    | 10    | 1     |       |
| 7    | 1     | 4     | The PFS structure |
| 8    | 2.5   | 4     |       |
| 9    | 5     | 4     |       |
| 10   | 10    | 4     |       |
| 11   | 1     | 7     |       |
| 12   | 2.5   | 7     |       |
| 13   | 5     | 7     |       |
| 14   | 10    | 7     |       |

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Fig.5 The liquefaction curve.

Fig.6 The definitions of “B” and “L”.

Fig.7 The input motion.
As mentioned above, the settlement is caused by the lateral flow of the ground. For all the cases, the values of (1) the “residual lateral volume” (due to lateral displacements at \(x = 13\) m, which includes sheet-piles and soil), and (2) the “residual vertical volume” (due to the settlements of the surface ground) are listed in columns 4 and 5, respectively, of Table 3. These volumes are expressed as cubic meters per unit meter in the y-direction. Figure 13 depicts the conceptual interpretations associated with volumes (1) and (2). Volume (1) increases with increasing values of B, because soil passes through between the end-bearing sheet-piles. Large lateral volume (1) can be seen because the input motion was given as an extremely large earthquake. As the vertical volume (2) is approximately equal to the lateral volume (1), the lateral flow may be regarded as the main factor that determines the settlement of an embankment under a large earthquake.

**c) Displacement of lateral deformation (Vertical section)**

The residual horizontal displacements in the vertical cross section (at \(x = 13\) m) are indicated in Fig.11. The results for Case-1 and 2 are represented by dotted lines in these figures, for the sake of comparison. The settlements for Case-3 to Case-14 (except for Case-6) exceed the settlement for Case-2 and are less than the settlement for Case-1. Although the settlement for Case-6 appears a little larger than that for Case-1 along the curve, the residual settlement (\(t = 100\) s) for Case-6 is 1.11 m, which is slightly smaller than that for Case-1, which is 1.13 m. This result is discussed in detail in a later section.

In Figs.12 (a) and 12 (b), the residual settlements are indicated for the entire range of cases, for various choices of the interval “B” and the length “L,” respectively. In Fig.12 (a), the settlements are seen to increase as the interval between end-bearing sheet-piles increase, for fixed values of L. In Fig.12 (b), the settlements are seen to increase as the length of floating sheet-piles become shorter (for fixed values of B), approaching the results for Case-1 as a limit. Comparing these figures, changing the interval B has a more dramatic effect on the eventual settlements than changing the length L has.

**b) Volume of lateral deformation**

Figures 9 and 10 show the results (for various cases) for two sections, one section running along the center of the end-bearing sheet-pile (\(y = 0\) m) and the other running along the center of the floating sheet-piles (\(y = 0.5 + B/2\) m). The lines include the displacements of both soils and sheet-piles except for the joint elements. For \(y = 0.5 + B/2\) m, the deformations of the sheet-piles appear in the upper part of the plot (\(z = 12 - L \sim 12\) m) and the deformations of the soil in the lower part (\(z = 0 \sim 12 - L\) m) of the plot. In all the plots that appear in Fig.14, the results for Case-1 and Case-2 are represented by dotted lines for the sake of comparison. For all cases, soils and sheet-piles deformed laterally, toward the right-hand side.
Fig. 10 The time histories of the settlement at D.

(a) Case 3–Case 6 (L=1 m)

(b) Case 7–Case 10 (L=4 m)

(c) Case 11–Case 14 (L=7 m)

Fig. 11 The time histories of the settlement at D.

Fig. 12 The residual settlement at D.

Table 3 Comparison of the deformation volumes.

| Case | B (m) | L (m) | Volume (1) (m$^3$/m) | Volume (2) (m$^3$/m) | (1) / (2) |
|------|-------|-------|----------------------|----------------------|-----------|
| 1    | -     | -     | 10.92                | 11.45                | 0.95      |
| 2    | 0     | -     | 3.31                 | 3.53                 | 0.94      |
| 3    | 1     | 1     | 6.40                 | 5.99                 | 1.07      |
| 4    | 2.5   | 1     | 8.27                 | 7.74                 | 1.07      |
| 5    | 5     | 1     | 9.40                 | 9.01                 | 1.04      |
| 6    | 10    | 1     | 10.21                | 10.01                | 1.02      |
| 7    | 1     | 4     | 6.35                 | 5.96                 | 1.06      |
| 8    | 2.5   | 4     | 8.16                 | 7.69                 | 1.06      |
| 9    | 5     | 4     | 9.18                 | 8.78                 | 1.05      |
| 10   | 10    | 4     | 10.21                | 9.88                 | 1.03      |
| 11   | 1     | 7     | 5.69                 | 5.49                 | 1.04      |
| 12   | 2.5   | 7     | 7.83                 | 7.43                 | 1.05      |
| 13   | 5     | 7     | 9.01                 | 8.57                 | 1.05      |
| 14   | 10    | 7     | 9.66                 | 9.22                 | 1.05      |

Fig. 13 The volumes (1) and (2).
For $y = 0$ m, the deformation of the sheet-piles increased as the interval B became larger. The horizontal displacement for $B = 1$ m (Case-3, Case-7 and Case-11) is smaller than doubled displacement for $B = 0$ m (Case-2). The deformation of the end-bearing sheet-piles increased as the length L increased. However, the length L had a relatively small effect on the deformations compared with the interval B.

Comparing two figures for the same case, the horizontal displacement of the floating sheet-piles is seen to be almost identical to that of the adjoining end-bearing sheet-piles. The horizontal displacement of the soil became larger with increasing interval “B” and with decreasing length “L,” which indicates that soil passed through below the floating sheet-piles.

d) Displacement of lateral deformation (Horizontal section)

The residual horizontal displacement in the horizontal cross section (at $z = 7$ m) is indicated for all cases in Fig.15. The results for Case-1 and Case-2 are indicated by dotted lines for the sake of comparison.

In Figs.15 (a) and 15 (b), the short straight lines parallel to the abscissae (y coordinates) represent the sheet-piles, while the curved lines represent soils passing through, in between the sheet-piles. In Fig.15 (c), the almost straight lines seen in the plot indicate the floating sheet-piles. The deformations of both the sheet-piles and the soil increased with an increasing interval “B.” The displacement of the soil was less than that for the case of no countermeasure (Case-1), except for Case-6. In that case, the displacement partially exceeded the value for the case of no countermeasure when the interval B = 10 m. This was because soil stopped by the sheet-piles had leaked in between the sheet-piles.

The installation of the end-bearing sheet-piles with too wide an interval caused soil deformation that was partially greater than the case for no countermeasure, under strong earthquake conditions. As a result, the settlement for Case-6 was larger than that for Case-1 at $t = 20$ s, as indicated in Fig.11 (a).
e) The distribution of lateral load

The horizontal load due to the liquefied ground was applied to the soil as well as to the sheet-piles. This load pushed soil in between the end-bearing sheet-piles and pushed those sheet-piles laterally as well. The authors calculated the load applied to the soil and the sheet-piles, respectively, to clarify the distribution of the lateral load. The authors only focused on the liquefiable layer because the lateral flow mainly occurred in the liquefiable layer.

The z-x shear stress was calculated along the line where the plane perpendicular to \( x = 13 \text{ m} \) met the plane perpendicular to \( z = 4 \text{ m} \). Figure 16 illustrates the fundamental principle that the z-x shear stress is calculated by dividing the sum of the horizontal force applied on the sheet-piles of the liquefiable layer by the cross section area of the sheet-pile on the x-y plane. The shear stress for all cases is indicated in Figure 17. The data points corresponding to \( B = 0 \text{ m} \) represent Case-2. The shear stress increased with increasing values of \( B \). The shear stress for the interval \( B = 1 \text{ m} \) (Case-3, Case-7, Case-11) was slightly lower than the doubled value of Case-2. Therefore, the sheet-piles for Case-3, Case-7, and Case-11 had smaller deformations than the doubled values for Case-2, as indicated in Figure 14(a). Similar behaviours can be derived from the results for the rest of the cases, compared with that of Case-2. In other words, an end-bearing sheet-pile installed with a certain interval between them can support a more comprehensive range of soils than an installation without any intervals.

(4) The search for a reasonable structure

The authors also considered the PFS method in economic terms. Table 4 lists the values obtained for “Weight,” “Inhibition,” and “Reasonability” for all the cases. “Weight” is defined as the weight of the

![Image](https://example.com/image.png)

![Image](https://example.com/image.png)

**Table 4** The effectiveness of the sheet-piles.

| Case | \( B \) (m) | \( L \) (m) | Weight (kN/m) | Inhibition (m) | Reasonability (m/kN/m) |
|------|------------|------------|--------------|----------------|----------------------|
| 1    | -          | -          | 0.00         | -              | -                    |
| 2    | 0          | -          | 78.00        | 0.86           | 1.1E-02              |
| 3    | 1          | 1          | 42.90        | 0.57           | 1.3E-02              |
| 4    | 2.5        | 1          | 27.86        | 0.30           | 1.1E-02              |
| 5    | 5          | 1          | 19.50        | 0.13           | 6.7E-03              |
| 6    | 10         | 1          | 14.18        | 0.02           | 1.4E-03              |
| 7    | 1          | 4          | 54.60        | 0.60           | 1.1E-02              |
| 8    | 2.5        | 4          | 44.57        | 0.36           | 8.1E-03              |
| 9    | 5          | 4          | 39.00        | 0.20           | 5.1E-03              |
| 10   | 10         | 4          | 35.45        | 0.07           | 2.0E-03              |
| 11   | 1          | 7          | 66.30        | 0.80           | 1.2E-02              |
| 12   | 2.5        | 7          | 61.29        | 0.46           | 7.5E-03              |
| 13   | 5          | 7          | 58.50        | 0.26           | 4.4E-03              |
| 14   | 10         | 7          | 56.73        | 0.13           | 2.3E-03              |
sheet-piles per meter in the y-direction. The material cost is directly related to the weight of material. The density of the sheet-piles was taken as 7.8 g/cm³. “Inhibition” is defined as the difference in the value of the settlement for any particular case and the value for Case-1. “Reasonability” is defined as the Inhibition divided by the Weight. The maximum value of effectiveness was found for Case-3, which meant the the end-bearing sheet-piles and the floating sheet-piles have to be installed alternately and the floating sheet-piles have to be as short as possible.

4. CONCLUSIONS

The authors carried out numerical analyses using LIQCA3D17 to make a quantitative evaluation of the effectiveness of the PFS method during liquefaction. Fourteen analytical cases were carried out, with varying values of the intervals between the end-bearing sheet-piles and the lengths of the floating sheet-piles. The following results were obtained:

1) The conventional method in which the end-bearing sheet-piles were installed into the toe of an embankment, without any intervals between them, could reduce the settlement of an embankment by 77%.
2) The settlement for the PFS structure was greater than that when all the sheet-piles were end-bearing and less than that when no countermeasures were used, except for one of the cases considered with the PFS method.
3) For one of the cases with a large interval between the end-bearing sheet-piles, the settlement was found to be a little greater than that for the case without any countermeasures for a short while. The installation of the end-bearing sheet-piles with too wide an interval partially increased the amount of soil deformation, compared with the case without any countermeasures.
4) As the volume subjected to lateral flow was almost the same as that due to the embankment settlement, regardless of the particular case, the lateral flow was considered to be the main factor determining the settlement of an embankment under a large earthquake.
5) The end-bearing and floating sheet-piles prevented the liquefied soil from flowing laterally, which reduced the settlement of an embankment.
6) The consideration of a large number of different cases revealed that the intervals between the end-bearing sheet-piles had a greater effect on the amount of settlement than that of the lengths of the floating sheet-piles.
7) Calculation of the shear stress of end-bearing sheet-piles revealed that an end-bearing sheet-pile installed with a certain interval between them could support a more comprehensive range of soils than an installation without any intervals.
8) In terms of material cost, the most effective outcome required that the end-bearing sheet-piles and the floating sheet-piles were installed alternately and the floating sheet-pile were as short as possible.
9) Some conditions, such as vertical load, input motion, yield of the sheet-piles, and so on, were simplified or ignored in the numerical analyses. These conditions should be improved to achieve results close to the actual behaviour.

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