LOAD TRANSFER BY SOIL ARCHING IN PILE-SUPPORTED EMBANKMENTS

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ABSTRACT

Piles have been used to support unsymmetrical surcharges due to embankments or backfills on soft grounds. The unsymmetrical surcharges can be transferred by embankment piles to a firm layer below soft grounds according to mobilizing soil arching in pile-supported embankments or backfills. Two kinds of model tests such as the soil arching test and the load transfer test were performed to investigate, respectively, the configuration of the soil arch and the loads transferred on piles in pile-supported embankments. In these model tests, model piles were installed in several rows below sand fills, and the heads of piles in each row were connected with cap beams. The soil arch showed a configuration of a semi hollow cylinder, whose diameter was equal to the space between the outer edges of two cap beams and thickness was equal to the width of the cap beams. Based on the configuration of the soil arch defined by the soil arching test, a theoretical analysis was carried out to predict the loads transferred on the cap beams; the loads depended on space between cap beams, width of cap beams, height and strength parameters of embankment fills, etc. The loads predicted by the presented equation showed good agreement with those measured in not only the presented test but also the previous test. Finally, the presented theoretical analysis was compared with the previous theoretical analyses on soil arching and its differences from the previous theories were discussed.

Key words: backfill, embankment, load transfer, model test, pile, soft ground, soil arching (IGC: E14)

INTRODUCTION

When backfilling is performed behind abutments on soft grounds or embankments are built on soft grounds, unsymmetrical surcharges due to backfills or embankments may generate lateral soil movement or lateral flow, and even sliding failure in soft grounds (Peck et al., 1974). If piles are placed to support infrastructures on soft grounds undergoing lateral flow, the piles will be subjected to undesirable lateral earth pressures due to interaction between soils and piles. Consequently, piles will move laterally, and the displacement of piles may bring about severe damages on infrastructures. The structural damages induced by the lateral flow in soft grounds have been reported (Franx and Boonstra, 1948; Heyman, 1965). Recently, many problems of structures related to lateral soil movements in soft grounds were reported (Hong, 2005).

To prevent these damages or problems, overburdens due to embankments or backfills should be minimized or supported completely. Embankment piles are used to decrease such overburdens on soft grounds according to mobilizing soil arching in pile-supported embankments (Reid and Buchanan, 1984; Jones et al., 1990; Gartung and Verspohl, 1996; Rogbeck et al., 1998). Usually, piles are installed in several rows and cap beams or isolated caps are placed on heads of piles (Hewlett and Randolph, 1988). The unsymmetrical surcharges due to embankments or backfills can be transferred by the piles to a firm layer below soft grounds. Soil arching, which is mobilized in pile-supported embankments during embankment filling, can hardly increase overburdens on soft grounds. Therefore, settlements and lateral movements of soft grounds can be prevented or minimized.

Model tests are performed to investigate the configuration of soil arching and the loads transferred on piles in pile-supported embankments. First, a test, in which model piles are installed in several rows below sand fills and heads of piles in each row are connected with cap beams, is performed to observe the zone of the soil arching in the pile-supported sand fills. Based on the configuration of the soil arch defined by the test, a theoretical analysis is carried out to predict the vertical loads transferred on the piles according to mobilizing soil arching in pile-supported embankments. The reliability of the presented theoretical analysis is investigated by comparing...
son of the analysis results with the results of a series of tests. Furthermore, to ensure that the presented theoretical analysis can work well for various conditions, the results from the presented theoretical analysis are compared with the previous experimental results. Finally, the presented theoretical analysis is compared with the previous theoretical analysis.

PREVIOUS STUDIES

Many researchers have contributed to the study of the soil arching in pile-supported embankment after Terzaghi (1943) defined the soil arching effect. Carlsson (1987) and Guido et al. (1987) suggested that only the soils in an isosceles triangle wedge zone between piles were loaded on soft grounds and the rest of embankment weight was transferred on piles. The base of the wedge zone was assumed to be equal to the clear space between pile cap beams. Carlsson (1987) assumed that the apex angle of the wedge zone was 30 degrees, while Guido et al. (1987) assumed that it was 90 degrees. This approach is very simple to predict the loads transferred on piles by the soil arching. Since this approach defined the wedge zone above soft grounds without considering shear strength of embankment fills, the loads transferred on piles may be inaccurately predicted.

Hewlett and Randolph (1988) assumed the soil arch in the form of a semi hollow cylinder, whose diameter was equal to the center-to-center space between pile cap beams and thickness equal to the width of the cap beams. Low et al. (1994) indicated that body force was not considered in the two-dimensional analysis performed by Hewlett and Randolph (1988). Low et al. (1994) assumed soil arching as was done by Hewlett and Randolph (1988) for a pile-supported embankment system, which used cap beams and introduced body force in their plane strain differential equation. However, Hewlett and Randolph (1988) and Low et al. (1994) both neglected the soil wedge zone above the pile cap beams in their approaches.

BS 8006 (1995) also recommended an equation to predict the loads acting on isolated pile caps due to soil arching, based on the works by Jones et al. (1990). Limitation still exists in this code, since the soil properties of fill materials have not been taken into account in the calculation of loads transferred on pile caps.

Recently, numerical analysis has been employed in researches related to pile-supported embankments (Russell and Pierpoint, 1997; Rogbeck et al., 1998; Han and Gabr, 2002). Horgan and Sarsby (2002) showed that the loads predicted by various approaches under same conditions were considerably different from each other. This means that in spite of the contribution of these researches, both the boundary of soil arching and the mechanism of load transfer in pile-supported embankments have not been clearly defined.

MODEL TEST APPARATUS

Two kinds of model tests were performed. The first test, which is called the soil arching test, focused on the observation of the configuration of soil arching developed in pile-supported embankments. The second test, which is called the load transfer test, focused on the investigation of loads transferred on piles in pile-supported embankments. The apparatuses used for the two kinds of model tests were basically the same except for the loading systems; the strain-control loading system was applied for the soil arching test, and the stress-control loading system was applied for the load transfer test.

Soil Arching Test Apparatus

A model test apparatus was manufactured to observe the soil arching mobilized in sand fills supported by model piles with cap beams, as shown in Fig. 1. The apparatus consisted of a soil container box, model piles with cap beams, and a soil deformation controller, as shown in Fig. 1(a).

The soil container box was made of transparent acrylic plates to observe the deforming behavior of sand fills supported by piles with cap beams. The acrylic plate of 20 mm thickness was chosen to strengthen the stiffness of the
The model piles, which were made of rigid steel, were 2 cm in diameter and 32 cm in length. The cap beams were 3 cm in width, 2 cm in thickness and 30 cm in length. The center-to-center space between two cap beams was 12 cm. A loading plate was placed between two side steel walls, which were used instead of model piles to separate the loading plate and the cylinder from sponge rubber. The thickness of the steel walls was same to the diameter of model piles. The loading plate was made of a rigid steel plate and connected to the piston of the cylinder (60 mm in stroke), which was operated by the soil deformation controller. Lowering down the loading plate could simulate the settlements of soft grounds. The displacement of the loading plate was controlled by the sensors attached on the cylinder, as shown in Fig. 1(b), which was operated by compressed air. These sensors were connected to the soil deformation controller, which moved the loading plate under the strain-control loading system. The displacement rate of the loading plate was approximately 2 mm/min.

Load Transfer Test Apparatus

Figure 2 illustrates the setup of model test apparatus to investigate the loads transferred on piles with cap beams by the soil arching in pile-supported sand fills. The setup of the model test apparatus was divided into three parts: main body, equipment for sand pluviation, and recording device.

The main body of the apparatus consisted of a soil container box and model piles with cap beams, as shown in Fig. 3. The main body of the load transfer test apparatus was basically the same that of the soil arching test apparatus, as shown in Fig. 1.

Several rows of model piles were placed through the simulated soft grounds in the soil container box. Heads of piles in each row were connected to cap beams and pile tips were inserted into holes punched in the base plate at the bottom of the soil container box, as shown in Fig. 3. To investigate the effect of space between cap beams on the load transfer, the space between cap beams was adjusted such that the interval ratio would be from 0.5 to 0.9. Hereupon, the interval ratio represents the ratio of the clear space to the center-to-center space between adjoining two cap beams. To easily set the model piles and to keep the pluviation height constant, the upper part of the soil container box was designed such that it could separate at the height of 50 cm over the model soft grounds, as shown in Fig. 3.

Dry sand rained through the pluviation equipment, which could make the density of sand fills uniform. The outside dimensions of the pluviation equipment were 110 cm in width, 58 cm in depth and 210 cm in height. The pluviation equipment, which covered the soil container box, was designed so as to be movable by four wheels attached below the frame of the equipment. The pluviation box in the equipment could move vertically up to 100 cm from the top of the soil container box. The elevation of the pluviation box was adjusted step by step with increasing the height of the sand fills to keep the height of sand pluviation constant. The bottom plate of the pluviation box consisted of two steel sheets punched with many small holes of 10 mm in diameter. The lower plate of the two sheets could be moved horizontally, while the upper plate was fixed. When the holes in two sheets coincided by the horizontal sliding of the lower plate, dry sand could pass through the holes.
Four load cells were installed both at the bottom of soft grounds and at heads of the piles below the cap beams of the center two rows. Both the vertical loads transferred on piles and the vertical loads acting on soft grounds were measured by these load cells. After the data by the load cells were inputted automatically into the computer through the data logging system, the vertical loads were calculated.

SAND TESTED AND MODEL SOFT GROUND

Embankment fills were simulated by sands. The sand sample used in the model tests was Jumunjin Sand, which was the Korean Standard Sand. The characteristics of this sand are summarized as follows: effective grain size, 0.41 mm; coefficient of uniformity, 1.78; specific gravity of grains, 2.62; maximum and minimum dry unit weight, 16 kN/m$^3$ and 14 kN/m$^3$, respectively. The internal friction angle of sand was $40.2^\circ$ (0.7 radian) for the specimen, whose dry unit weight was 15.4 kN/m$^3$ and relative density was 72.8$, which was equal to the relative density obtained by sand pluviation from 80 cm height.

The sponge rubber was used to simulate soft ground in the model tests. To confirm the applicability of the sponge rubber as a substitution material for soft ground, two preliminary load transfer tests were performed on the pile-supported embankments on both Ansan clay and the sponge rubber. The uniaxial compression strength and initial deformation modulus of Ansan clay were 8.8 kN/m$^2$ and 860 kN/m$^2$, respectively. The unit weight of sponge rubber was 0.15 kN/m$^3$. The vertical loads measured from the preliminary tests for both Ansan clay and the sponge rubber are compared in Fig. 4, in which the test results showed that the sponge rubber can provide a good practical simulation of soft ground.

TESTING PROGRAM

Soil Arching Test

A cylinder was placed between center two cap beams inside the soil container box as shown in Fig. 1(a). Pile tips were inserted in several rows into the bottom of the soil container box, while the pile heads in each row were connected with the cap beams, which were made of acrylic blocks.

The sponge rubber was placed among the piles under the cap beams outside portion A, which was separated by two side steel walls, in which the tops were connected to the cap beams and the tips were inserted into the bottom groove.

After setting up the cylinder and the model piles with the sponge rubber, Jumunjin sand was filled to 60 cm thickness over both pile cap beams and the loading plate by the pluviation equipment, in which height of pluviation was 80 cm.

At about every 4 mm of sand filling, lateral strips of 3 mm thickness of black sand, which was coated by carbon powder, were placed. After the loading plate was lowered, the soil deformation behavior was captured by camera. Consequently, the shapes of the variation of the black lateral strips could be observed.

Load Transfer Test

A series of model tests were performed to investigate the vertical loads transferred on the cap beams by mobilizing soil arching in pile-supported sand fills. The model tests focused on the space between cap beams and embankment height as the major factors affecting on the load transfer in pile-supported embankments.

The space between cap beams could be adjusted according to the interval ratio, which was determined as the ratio of the clear space to the center-to-center space between cap beams. The interval ratio could be adjusted from 0.5 up to 0.9. The piles were installed in several rows below the sand fills, and cap beams were placed on the pile heads perpendicular to the longitudinal axis of sand fills.

Jumunjin sand was uniformly placed on both cap beams and sponge rubber in the soil container box by the pluviation equipment of dry sand, as shown in Fig. 2. After placing dry sand of 10 cm thickness in the pluviation box, dry sand was rained from the height of 80 cm to build a layer of sand fills, whose thickness in the soil container box became approximately 8 cm. Then, the next sand pluviation was performed step by step. At every sand pluviation stage, the height of sand fills was measured, and the vertical loads acting on piles were recorded in a computer through the data logging system. The height of sand fills reached up to 80 cm. These tests were performed for the different interval ratios of the pile cap beams. The interval ratios planned in the model tests were 0.5, 0.6, 0.7, 0.8 and 0.9.

In order to compensate the wall friction between sands and walls inside the soil container box, another preliminary test was performed. The result of the preliminary test showed that the average difference between the actual weight of the sand fills and the measured loads captured by load cells was about 4%. The wall friction was compensated when the weight of sand fills was calculated.
RESULTS OF MODEL TESTS

Observation of Soil Arching

Figure 5 shows the configuration of the soil deformation in the pile-supported sand fills from a small displacement level at an earlier loading state up to a large displacement level which triggers the ultimate failure mode. The soil deformation could be investigated by observing the movement of the lateral black stripes of sands from their initial positions drawn by white lines in Fig. 5.

Figure 5(a) shows the soil deformation for the loading plate displacement of 10 mm. At this loading state, some local shear bands appeared as shown in Fig. 5(a). However, the local shear bands could not produce the full mobilized soil arching. Although the configuration of soil arching is not clear at an earlier loading state, the plastic deformation in sand fills already occurs in a downward direction with the local shear bands. Lateral black stripes of sands moved down within the boundary drawn by a white semi circle, in which the diameter was equal to the space between the outer edges of the two cap beams, as drawn in Fig. 5(a). Sand particles within this boundary tried to move toward the center of the semi circle, while the soils outside this boundary did not deform. Therefore, this boundary could be defined as the boundary of the soil arching zone.

Figure 5(b) shows the soil deformation at ultimate failure state which was triggered by the general shear band when the loading plate was lowered by 60 mm. The location of the general shear band was different from those of the local shear bands. At an earlier loading state, the general shear band did not mobilize sufficiently. Sands and the initial local shear bands within the inner arch between two cap beams subsided totally at ultimate failure state. The boundary of the inner arch is drawn by a white solid arch in Fig. 5(b), which is not a semi circle. However, the white inner solid arch was located originally at the position drawn by the dotted semi circle, whose diameter was equal to the clear space between two cap beams. Sand particles located originally at the position drawn by the white dotted semi circle moved to the white solid arch during the lowering of the loading plate by 60 mm. Therefore, the soil arching zone could be defined by two semi circles with a thickness equal to the width of the cap beams, as shown in Fig. 5(b).

On the other hand, the lateral black stripes of sands above cap beams did not move, when the loading plate was lowered by 60 mm, as shown in Fig. 5(b). Although sands within the inner arch between two cap beams subsided totally, the sands in a zone above the cap beam remained intact. Hereupon, this zone can be defined as the wedge zone, which is assumed to remain intact during the mobilization of the soil arch. This wedge zone is a similar to the one developed in the soil stratum below a
long footing. The surcharge load from sand fills is transmitted on the cap beam through this soil wedge. Consequently, Fig. 5 illustrates the boundary of the soil arching developed in pile-supported embankments. Only soils within the outer semi circle drawn by the white line deformed, while soils within the wedge zones above cap beams remained intact. Especially, the soils within the inner semi circle drawn by a white dotted line deformed outstandingly. Therefore, the configuration of the soil arching in pile-supported embankments can be represented as a semi-hollow cylinder of diameter equal to the space between the outer edges of the adjoining two cap beams and thickness equal to the width of the cap beams.

**Geometric Configuration of Soil Arch**

Based on the result of the soil arching test as shown in Fig. 5, the geometric boundary of soil arching zone can be drawn as shown in Fig. 6(a). The soil arching zone can be represented as a semi-hollow cylinder, whose center axis passes through O, which is in the middle between the two cap beams. The soil arch can be drawn by outer and inner semi circles, in which the diameter of the outer semi circle is equal to the space between the outer edges of the two cap beams and the diameter of the inner semi circle is equal to the clear space between the two cap beams. The soil arching zone can be divided into the outer soil arching zone and the inner soil arching zone as shown in Fig. 6(a). In the outer soil arching zone, the wedge zone exists above the pile cap beams and can be determined by overlapping the adjoining two outer semi circles.

Hewlett and Randolph (1988) also assumed a similar configuration of the soil arching zone as shown in Fig. 6(b). The configuration of the soil arch assumed by Hewlett and Randolph (1988) did not coincide with the result observed in the soil arching test as shown in Fig. 5. In the soil arching test, the soil arching developed at areas wider than those assumed by Hewlett and Randolph (1988). The major differences were that a boundary of the outer soil arching zone shown in Fig. 6(a) was wider than ones assumed in Fig. 6(b) and the wedge zone was defined above cap beams during soil arching.

**Loads Transferred on a Pile Cap Beam**

Figure 7 shows the vertical loads transferred on a cap beam during a series of model tests. The vertical loads plotted in Fig. 7 are the loads measured by load cells used in the model tests. In order to compare with the weight of the sand fills per unit area, the vertical loads transferred on a cap beam was represented as the loads per unit area, in which the vertical loads were divided by the center-to-center space between pile cap beams.

This figure illustrates how much of the weight of sand fills is transferred on a pile cap beam by soil arching in pile-supported sand fills. The vertical loads transferred on a pile cap beam increase linearly with increasing weight of the sand fills. For a given weight of the sand fills, the lower interval ratio shows the higher vertical loads per unit area. Hereupon, a high interval ratio means that space between pile cap beams is wider than that of a low interval ratio.

The diagonal straight line in Fig. 7 represents that all of the weight of the sand fills is transferred on the cap beams, in which the load transfer efficiency on a cap beam is one hundred percent. In this figure, the loads measured in tests under the low interval ratio of lower than 0.7 were plotted close to the diagonal straight line. That is, most of the weight of the sand fills could be trans-
ferred on the cap beams because the cap beams were placed into a narrow space.

On the other hand, when cap beams were placed into very wide space, whose interval ratio became 0.9, only half of the weight of the sand fills was transferred on cap beams, and the rest half was loaded on the soft ground. For example, when the height of the sand fills rose up to 80 cm high, the vertical loads transferred on a pile cap beam was 6.1 kN/m², while the weight of the sand fills was 12.2 kN/m².

**THEORETICAL ANALYSIS**

*Equation to Predict Vertical Loads*

On the basis of the configuration of the soil arching zone shown in Fig. 6(a), an equation can be derived to predict the vertical load transferred on a pile with cap beam in pile-supported embankments. Figure 8 shows an analytical model of semi cylindrical soil arch in pile-supported embankments. The embankment soils supported by pile cap beams would show two-dimensional behavior under the plane strain condition. Therefore, two-dimensional analysis can be applied to analyze the mechanism of soil arching in pile-supported embankments.

Normal stresses acting on an element I near the crown of the soil arch in Fig. 8 are the principle stresses. It is assumed that same normal stress components exist on all elements such as element II, which are located at the same distance from the center O as shown in Fig. 8.

Based on the assumption, the basic differential equation of equilibrium in the radial direction can be expressed as follows (Timoshenko and Goodier, 1970):

\[
\frac{d\sigma_r}{dr} + \frac{\sigma_r - \sigma_\theta}{r} = -\gamma
\]  

(1)

Where \(\sigma_r\) = the radial normal stress; \(\sigma_\theta\) = the tangential normal stress; \(r\) = the radial distance and \(\gamma\) = the unit weight of embankment fills.

Body force is introduced in the plane strain differential equation of equilibrium as expressed in Eq. (1). Body force was not considered by Hewlett and Randolph (1988) in their two-dimensional analysis, but was considered in three-dimensional analysis. However, Low et al. (1994) expressed that body force should also be considered in two-dimensional analysis.

According to Mohr’s circle of stress on the limit state condition of element, tangential normal stress \(\sigma_\theta\) is equal to \(N_0 \cdot \sigma_\theta + 2 \cdot c \cdot N_0^{1/2}\), where \(N_0 = \tan^2 (\pi/4 + \phi/2)\), \(\phi\) = internal friction angle of embankment fills and \(c\) = cohesion of embankment fills. Substituting \(\sigma_\theta\) into Eq. (1), the general solution can be expressed as follows;

\[
\sigma_r = A \cdot r^{(N_0 - 1)} + \gamma \cdot \frac{r}{N_0 - 2} \cdot \frac{2cN_0^{1/2}}{N_0 - 1}
\]  

(2)

Where \(A\) = integral constant.

At \(J\), where the crown of the outer soil arch is located, \(r = r_1\) and \(\sigma = \sigma_{11}\). Using this boundary condition, the integral constant \(A\) can be calculated from Eq. (2). Substituting the integral constant \(A\) again into Eq. (2), \(\sigma_r\) becomes as follows;

\[
\sigma_r = \gamma \left\{H' \cdot \frac{r_1}{N_0 - 2} \cdot \left(\frac{r}{r_1}\right)^{N_0 - 1} \right. \\
+ \left. \frac{r}{N_0 - 2} \cdot \left(1 - \frac{r_2}{r_1}\right)^{N_0 - 1} \cdot \frac{2cN_0^{1/2}}{N_0 - 1} \right\}
\]  

(3)

Where \(H' = H - H_1\), \(H_1\) = height of embankment fills and \(H_1\) = height at the crown of the outer soil arch.

Meanwhile, at \(J'\) where the crown of the inner soil arch is located, \(r = r_2\) and \(\sigma = \sigma_{11}\) becomes \(\sigma_1\), which is expressed as follows;

\[
\sigma_1 = \gamma \left\{H' \cdot \frac{r_2}{N_0 - 2} \cdot \left(\frac{r}{r_2}\right)^{N_0 - 1} \right. \\
+ \left. \frac{r_2}{N_0 - 2} \cdot \left(1 - \frac{r_2}{r_1}\right)^{N_0 - 1} \cdot \frac{2cN_0^{1/2}}{N_0 - 1} \right\}
\]  

(4)

The vertical stress \(\sigma_r\) is acting on the soft ground in the middle between cap beams and can be expressed as Eq. (5). That is, the vertical stress \(\sigma_r\) can be obtained by substituting \(\sigma_1\) calculated by Eq. (4) into Eq. (5).

\[
\sigma_r = \sigma_1 + H_2\gamma
\]  

(5)

Where \(H_2\) = height at the crown of the inner soil arch.

The vertical \(P_r\) load transferred on a pile cap beam can be obtained by subtracting the soil weight acting on the soft ground from the total overburden weight of embankment fills, as expressed by Eq. (6). Hereupon, the vertical load \(P_r\) is the load expressed per unit length along the longitudinal axis of pile cap beams. And the soil weight acting on the soft ground is equal to the summation of the vertical load \(\sigma_r\) loaded on the soft ground over the clear space between the pile cap beams.

\[
P_r = \gamma \cdot D_1 \cdot H - \sigma_r \cdot D_2
\]  

(6)

Where \(D_1\) = center-to-center space between the cap beams and \(D_2\) = clear space between the cap beams.

The efficiency of embankment piles \(E_t\) can be expressed as a ratio of the vertical load \(P_r\) to the total weight of embankment fills as follows;
Equation (7) can be expressed as Eq. (8).

\[ E_i = \frac{P_n}{\gamma \cdot D_1 \cdot H} \times 100(\%) \]  (7)

Equation (8) shows that efficiency can be also expressed as a ratio of the vertical load \( P_n \) per unit area to the weight of embankment fills per unit area. Therefore, the inclination of the values in Fig. 7 represents the efficiency expressed by Eq. (8).

**Characteristics of Theoretical Equation**

The theoretical equation derived above includes many parameters related to piles and soils. Therefore, the characteristics of theoretical equation may need to be clarified by investigating the effects of various parameters on the load transferred on pile cap beams. In a pile-supported embankment system, the factors affecting the vertical loads transferred on piles can be classified into two groups with respect to their relation with piles and soils. The space between cap beams and width of cap beams are major factors related to piles, while height and strength parameters of embankment fills are major factors related to soils.

Figures 9, 10 and 11 show the results of the investigation on the efficiency of the load transfer on a cap beam according to the variation of the influencing parameters. The investigation focused on the four factors related to piles and soils such as the width of the cap beams, the space between the pile cap beams, the internal friction angle of the embankment soils and the height of the embankment fills.

Figure 9 shows that the efficiency is affected by the width of the cap beams. The width of the cap beams is expressed as the area ratio \( b/D_1 \), which represents the ratio of the width of cap beams to the center-to-center space between cap beams. The higher area ratio shows the higher efficiency. Therefore, it is favorable to choose wide cap beams, if the stiffness of the cap beams is sufficiently enough.

Figure 10 shows the effect of the space between the cap beams on efficiency for various internal friction angles of the embankment soils. The effect of the space between the cap beams is represented by the interval ratio \( D_2/D_1 \), which is the ratio of the clear space to the center-to-center space between cap beams. The interval ratio can be chosen between zero and unity, which covers the full range of space between cap beams. A higher interval ratio means a wider space between cap beams. A higher interval ratio yields a lower efficiency. This low efficiency means that the effect of the cap beams to support the embankment fills decreases when the cap beams are placed into a wide space. The efficiency decreases gradually with increasing interval ratio until the interval ratio reaches about 0.7. As the interval ratio increases beyond 0.7, the efficiency decreases rapidly. Generally, it is favorable to choose a lower interval ratio. However, extremely low interval ratio requires narrow spacing and expensive designs. Therefore, a design adopting the interval ratio of about 0.7 is desirable technically and economically.

Figure 10 also shows that the efficiency is affected by the internal friction angle of the embankment soils. The higher internal friction angle shows the higher efficiency.
Therefore, it is favorable to choose filling materials with a high internal friction angle. The water table condition or rain drainage condition may influence the internal friction angle, which is related to the efficiency. However, the influences due to the variation of water table and the rain drainage on the mechanism of soil arching are usually undesirable. Therefore, the soils, which have high permeability, are recommended as filling materials to minimize the influence of water.

The variation of the efficiency with the embankment ratio $H/H_1$ is shown in Fig. 11. The height of embankments is expressed as the embankment ratio, which represents the ratio of embankment height to the height at the crown of the outer soil arch. The efficiency increases initially with increasing height of the embankments until the embankment ratio reaches four. As the embankment ratio increases beyond four, only a small increase in the efficiency is indicated.

Therefore, to provide sufficient efficiency of the cap beams to support embankment fills, embankment filling should be performed so that the height of the filling is over four times the height at the crown of the outer soil arch.

**COMPARISON BETWEEN MODEL TESTS AND THEORETICAL ANALYSIS**

*Effect of Space between Cap Beams*

Figure 12 shows the comparison between the efficiency measured in the model tests and the predicted efficiency according to the space between the cap beams, which is represented in terms of the interval ratio $D_2/D_1$. To investigate also the influence of the pile cap width $b$ on the efficiency, the horizontal axis of Fig. 12 is represented in terms of the area ratio $b/D_1$ together with the interval ratio $D_2/D_1$. The predicted efficiency is in good agreement with the experimental efficiency over a wide range of the interval ratio between the cap beams.

The efficiency decreases gradually with increasing the interval ratio until the interval ratio reaches about 0.7. However, the efficiency decreases rapidly as the interval ratio increases beyond 0.7, especially close to unity. This means that the effect of the load transfer in pile-supported embankments is very low when the cap beams are placed into wide space, in which the interval ratio becomes higher than 0.8. Therefore, to maximize the effect of load transfer in pile-supported embankments, the interval ratio of the cap beams should be decreased on design by reducing the space between the cap beams and/or enlarging the width of the cap beams. If the efficiency must be higher than 80%, it is desirable to design for the interval ratio of lower than 0.8.

When the pile cap width $b$ is outstandingly wider than the clear space between cap beams, the area ratio $b/D_1$ is close to unity and the interval ratio $D_2/D_1$ is close to zero. Under such an exceptional extreme situation, the pile slab method is more suitable than the pile cap beam method, since soil arching is difficult to develop.

Another exceptional extreme situation is presented when the space between cap beams is outstandingly wide. This situation occurs when only one cap beam is placed, where the area ratio $b/D_1$ is close to zero and the interval ratio $D_2/D_1$ is close to unity. The mechanism of the soil arching in embankments can not be expected also under this exceptional extreme situation. Usually, in fields of road and railroad construction, the embankment piles are placed in space with the interval ratio $D_2/D_1$ between 0.4 and 0.8 (Reid and Buchanan, 1984; Jones et al., 1990; Gartung and Verspohl, 1996; Rogbeck et al., 1998).

To ensure that the presented theoretical analysis can work well for various conditions, the results from the presented theoretical analysis were compared with the previous experimental results of Low et al. (1994), as shown in Fig. 13. The density and internal friction angle of the embankment soils were, respectively, 13.8 kN/m$^3$ and 45°. Although Low et al. (1994) performed the model tests at the interval ratio between 0.8 and 0.9, which was an interval ratio that was too high to
sufficiently simulate the field practical situation, the efficiency predicted by the presented equation showed good agreement with the experimental one.

**Effect of Embankment Height**

Figure 14 shows the comparison between the experimental efficiency and the predicted efficiency according to the embankment ratio $H/H_1$, which represents the ratio of embankment height to the height at the crown of the outer soil arch. The predicted efficiency correlates well with the experimental efficiency for the process of sand filling. The efficiency initially increases with the increasing of the embankment ratio, and then, the increasing rate gradually decreases to converge on a constant value.

**Comparison between Experimental Results and Various Theoretical Analyses**

Figure 15 shows the comparison between the experimental vertical loads measured in the model tests and the theoretical vertical loads predicted by various equations. The vertical loads predicted by Eq. (6) shows good agreement with the experimental vertical loads. However, the vertical loads predicted by Carlsson (1987) and Guido et al. (1987) overestimate the experimental vertical loads, while Terzaghi (1943) and Low et al. (1994) slightly underestimate the experimental vertical loads.

Overestimation by Carlsson (1987) and Guido et al. (1987) is induced by the assumption of the triangle wedge zone between piles. Carlsson (1987) and Guido et al. (1987) assumed that only the soils in an isosceles triangle wedge zone between piles were loaded on soft grounds and the rest of embankment weight was transferred on the piles. The triangle wedge zone may fall under the inner arching zone assumed by Carlsson (1987) and Guido et al. (1987) as shown in Fig. 6(a). Therefore, this assumption led to the inaccurate estimation of loads.

On the other hand, underestimation by Terzaghi (1943) and Low et al. (1994) is induced by the assumption on the boundary for the soil arch in pile-supported embankments. Since the soil arch in these approaches are defined without considering the wedge zone above the cap beams, the soil arch assumed in these approaches is smaller than one observed by the model tests as explained in Fig. 6. Terzaghi (1943) described the soil arch through a trap door. However, the soil wedge above cap beams can not be observed in the trap door test. The mechanism of soil arching in pile-supported embankments does not coincide with the mechanism of soil arching above trap door, because the pile cap beams are placed in several rows. Low et al. (1994) used semi-circular timbers for cap beams in their model tests. Therefore, they could not observe the soil wedge above the cap beams, as well. Furthermore, the vertical loads predicted by Low et al. (1994) were dependent on a coefficient which was introduced for the equivalent uniform stress acting on the soft ground between cap beams. Maximum vertical loads predicted by Low et al. (1994) show good agreement with the experimental vertical loads.

**CONCLUSIONS**

A model test was performed to investigate soil arching in pile-supported embankments. Piles were installed in several rows below the embankments and cap beams were placed on pile heads perpendicular to the longitudinal axis of the embankments. The soil arching developed in the pile-supported embankments as a semi hollow cylinder of diameter equal to the space between the outer edges of two cap beams and thickness equal to the width of the cap beams.

Based on the configuration of the soil arch observed in the model test, a theoretical equation was presented to predict loads transferred on piles with cap beams in pile-supported embankments. The equation could consider the effect of various factors affecting on the loads transferred on pile cap beams; the loads depended on the space between cap beams, width of cap beams, height of embankment fills, and strength parameters of embankment soils.
Also, laboratory investigations were performed to measure the loads transferred on piles by soil arching in pile-supported sand fills and to verify the reliability of the presented equations. When the height of the sand fills was higher than the crown of the outer soil arch of the soil arching zone, soil arching developed well and the predicted vertical loads showed good agreement with the loads measured in the model tests. Furthermore, the results from the presented theoretical analysis showed good agreement with the previous experimental results of Low et al. (1994).

However, the vertical loads predicted by Carlsson (1987) and Guido et al. (1987) overestimated the experimental vertical loads, while Terzaghi (1943) and Low (1987) and Guido et al. (1987) overestimated the expected vertical loads, while Terzaghi (1943) and Low et al. (1994) slightly underestimated.

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NOTATION

\[ A: \text{ integral constant} \]
\[ b: \text{ width of cap beam} \]
\[ c: \text{ cohesion of embankment fills} \]
\[ D_1: \text{ center-to-center space between cap beams} \]
\[ D_2: \text{ clear interval space between cap beams} \]
\[ E_t: \text{ efficiency} \]
\[ H: \text{ height of sand fills} \]
\[ H_s(=r_1): \text{ height of the outer soil arch at peak.} \]
\[ H_i(=r_2): \text{ height of the inner soil arch at peak} \]
\[ H': H - H_i \]
\[ J: \text{ top of the outer arch} \]
\[ J_i: \text{ top of the internal arch} \]
\[ N_c: \text{ coefficient } (= \tan^2 (\pi/4 + \phi/2)) \]
\[ P: \text{ vertical load acting on cap beam} \]
\[ r: \text{ radial distance of element in arching zone} \]
\[ y: \text{ unit weight of embankment fills} \]
\[ \sigma_r: \text{ radial stress} \]
\[ \sigma_i: \text{ vertical stress at peak of internal arch} \]
\[ \sigma_s: \text{ tangential stress} \]
\[ \phi: \text{ internal friction angle of embankment fills} \]

REFERENCES