Experimental findings of 3D seepage failure of soil within a cofferdam

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ABSTRACT

For deep and/or large excavation of soil with a high ground water level, three-dimensional seepage flow through soil within a cofferdam is a problem. Three-dimensionally concentrated flow lowers the stability against seepage failure more than in the two-dimensional condition. To clarify the three-dimensional seepage failure mechanism, experiments were conducted under three-dimensional flow conditions for various cases, and analyses of FEM seepage flow and stability against seepage failure of soil were carried out. Theoretical critical hydraulic head differences based on the Prismatic failure concept are defined as $H_{PF}$. From experiments, the hydraulic head differences at an abrupt change of discharge $H_d$, at the onset of soil deformation $H_y$, and at failure $H_f$ are considered. The self-stabilizing effects and the method for estimating $H_f$ using the value $H_{PF}$ are discussed.

Keywords: 3D seepage failure of soil, cofferdam, critical hydraulic head difference, self-stabilizing effect

1 INTRODUCTION

For the excavation of soil with a high ground water level, sheet piles or diaphragm walls are often used for retaining soil and water. Under such conditions, seepage flow occurs through soil, and seepage failure is often a problem. For an excavation involving a large area, seepage failure is a problem in two dimensions. On the other hand, the more the region of a cofferdam is restricted, the greater the seepage flow concentrates three-dimensionally within the cofferdam. The three-dimensionally concentrated flow lowers the safety factor for seepage failure more than in the two-dimensional condition (Nikkei construction, 2001).

In this paper, 3D seepage failure experiments were conducted in 20 cases involving various thicknesses of soil, with or without excavation. Analyses of the FEM seepage flow and stability against the seepage failure of soil based on the prismatic failure concept 3D were also carried out. The characteristics of seepage failure of soil in three dimensions are discussed based on experiments and numerical analyses. Here we discuss the hydraulic head differences at an abrupt change of the $H-Q_{15}$ curve $H_d$, at the onset of deformation $H_y$, and at failure $H_f$, where $H$ is the hydraulic head difference between up- and downstream sides, and $Q_{15}$ is the discharge translated to the value at 15 degrees centigrade. The margin of the differences $H_f - H_y$, i.e., the self-stabilizing effect, is presented. We also prove that $H_f$ can be estimated using the theoretical critical hydraulic head difference based on the prismatic failure concept 3D, $H_c$.

Fig. 1. 3D seepage failure apparatus

2 THREE DIMENSIONAL EXPERIMENTS

A test apparatus was designed for studying 3D seepage failure of soil within a cofferdam as shown in Fig.1. The main apparatus consists of a seepage tank.
and water tank on the left/rear side as shown in Fig.1. The seepage tank is made of stainless steel, 1,000mm wide, 1,300mm high and 1,000mm deep. The front of the tank is made of transparent glass for observation of the behavior of soil particles inside and the right side of the tank is equipped with 283 piezometer holes for the measurements of pore water pressures. The twenty-nine piezometer holes are also installed at the bottom of the seepage tank. The left and rear sides of the tank are fitted with partitions of 700mm high. There is water seepage tank. The left and rear sides of the tank are partitioned into five sides. A cofferdam is mounted on the right/front side with a surface shape of 200mm×400mm. Outside of the cofferdam is referred to as upstream, and inside as downstream.

Lake Biwa Sand 3 was used as the test sand. It is classified as a uniform and fine sand under 850μm. The specific gravity $G_s$ is 2.668, the uniformity index $U_r$ is 1.404, the 50 % grain size $D_{50}$ is 0.283 mm, minimum and maximum void ratios $e_{\min}$ and $e_{\max}$ are 0.761 and 1.115, and the coefficient of permeability at 15 °C, $k_{15}$, is $7.263\times10^{-4}$ m/s ($D_r=50\%$).

The test procedure for making sand models and setting up the test equipment are as follows: test sand, which is put into several containers and weighed, is soaked in water for about one week to fully saturate and deaerate it. The sand saturated with water is poured by hand, little by little, into the upstream part from the inside of the water tank, and into the downstream part from the outside of the seepage tank. The sand is placed in 10 to 20 layers and compacted with an aluminum rod 7mm in diameter, 1,000mm in length and 100g in weight for upstream soil, and 5mm in diameter, 1,000mm in length and 55g in weight for downstream soil. For each layer, the compacting rod is dropped from about 100mm height and by a given number of drops: 50 for upstream and 8 for downstream soil.

The series of tests for sand models was numbered from E0301 to E0320. Notation is as follows referring to Fig.2: $T_1$ and $D_1$ are the total depth of soil and penetration depth of sheet piles on the upstream side, $T$ and $D$ are those on the downstream side, $d (=D_1−D)$ is the excavation depth for an excavation model, and $D_r(=48.1−52.2\%)$ is the relative density of soil. The length between the lower edge of the sheet pile wall and the bottom of apparatus $T−D=T_1−D_1=20.0$ cm.

3 PRISMATIC FAILURE CONCEPT 3D (PFC 3D)

The Prismatic failure concept 3D presented by Tanaka et al. (2012) is used for estimating the stability against seepage failure of soil. In the Prismatic failure concept 3D, we assume that the body of soil lifted by seepage water has the shape of a prism with a certain height and width adjoining the sheet pile wall. The rise of the prism is resisted by the submerged weight, $W'$, and frictions $F_RL$ and $F_{RBR}$ on the left and right sides, and $F_{RF}$ and $F_{RCB}$ on the front and back sides. The safety factor $F_s$ with respect to the rise of the prism, which is subjected to the excess pore water pressure on its base, $U_e$, is given as:

$$F_s = \frac{W' + F_{RL} + F_{RBR} + F_{RF} + F_{RCB}}{U_e} \quad (1)$$

For the hydraulic head difference $H$ between up- and downstream sides, safety factors, $F_s$, are calculated for all of the prisms within a cofferdam. The safety factor $F_s$ takes the minimum $F_{s_{\min}}$ for a certain prism among all of the prisms. The calculation is iterated for another hydraulic head difference, $H$, until the condition whereby $F_{s_{\min}}$ becomes nearly equal to 1.0 is found. $H=H_c$ at which the condition $F_{s_{\min}}=1.0$ is applied is defined as the critical hydraulic head difference. The prism with a value of $F_{s_{\min}}=1.0$ among all of the prisms for $H=H_c$ is defined as the critical prism. We could say that the critical prism is separated from the underlying soil at its base when $H$ exceeds $H_c$. Safety factors using the Prismatic failure concept when considering frictions are discussed below.

Analyses were conducted using a Fortran program FEMSEE6F for FEM seepage flow analyses with iso-parametric elements composed of 27 nodes, and a Fortran program SEEPFL67 for the stability analyses based on the pfc 3D. These two programs, not coupled, were coded in the authors’ laboratory.
4 EXPERIMENTAL RESULTS

An upstream water head was raised stepwise until the sand model deformed and collapsed. The increase in hydraulic head difference, $\Delta H$, was 20% of the critical hydraulic head difference of the sand model $H_c$ at early steps ($H < 0.8 H_c$). $\Delta H$ was then gradually reduced (by half and by half), and finally 2% of $H_c$ in the vicinity of $H = H_c$.

4.1 $H$-$Q_{15}$ curve and the hydraulic head difference at an abrupt change of the curve $H_d$

Fig.3 shows the $H$-$Q_{15}$ curve for the test E0320. It is observed from Fig.3 that $Q_{15}$ increases linearly with increasing $H$ until a certain value $H_d$. $H_d$ value is referred to as the hydraulic head difference at which the $H$-$Q_{15}$ curve diverts from linearity. The linear relationship does not hold and the discharge by experiment increases abruptly when $H$ goes beyond $H_d$.

As stated below, at almost the same point as $H_d$, the soil surface begins to settle on the upstream side and to rise on the downstream side. This is because, at this point, soil loosens on the downstream side, void spaces enlarge, permeability of the soil grows larger, and discharge increases non-linearly with $H$. As $H$ increases beyond $H_d$, $Q_{15}$ becomes larger with increasing $H$ more steeply than before, and the ground finally collapses at the hydraulic head difference at failure $H_f$.

4.2 Change in shapes of soil surface

The heights of the soil surface were measured at several chosen points along the measurement line shown in Fig.4. The measurement line is a bisector of the right angle of the inside corner of the rectangular diaphragm wall. With respect to seepage failure, soil is most critical on the line. Let us consider the test E0320 ($H_d = 29.25$ cm, $H_f = 38.06$ m). Figs. 5(a)-(d) show changes in shape of the surface of the sand model along the surface height measurement line with increase in $H$, from $H = 4.34$ cm at the first step to $H = 37.70$ cm at one step before failure. Configurations of soil surfaces are plotted by series of dots in Figs. 5(a)-(d).

The model sand is in a stable state for early steps of $H$ (Fig. 5(a)). When $H$ increases beyond a certain value $H_f$, the model sand changes in shape near the sheet pile wall. The surface of the soil in the vicinity of the sheet pile wall subsides on the upstream side and rises on the downstream side (Fig. 5(b)). The value of $H_f$ is referred...
to as the hydraulic head difference at onset of deformation. Some soil particles are observed to move around from the upstream to downstream sides near the bottom tip of the sheet pile wall. Subsidence of the upstream soil surface and rise of the downstream soil proceed with steps of increasing $H$ (Fig. 5(c)). The upstream soil surface is an inverse conical shape centered at the outer corner of the rectangular diaphragm wall. A close-up photo of the upstream inverse conical shape is given in Fig. 6 at $H=37.70$ cm (E0320). The slope angle increases with an increase in the hydraulic head difference, $H$, from $45^\circ$ immediately after the deformation of soil to $52^\circ$ just before failure for the test E0320. The rise in the downstream soil surface occurs uniformly within the cofferdam, especially when the penetration depth of sheet piles is large. As $H$ increases and approaches $H_f$, the upstream subsidence shows a clear inverse conical shape, and sand particles are observed to roll down on the slope of the upstream soil surface (Fig. 5(d)). Fine grains of soil boil at the downstream soil surface and the water becomes dirty. Immediately after $H$ reaches $H_f$, a mass of soil moves slowly from up- to downstream, and after a few seconds the downstream soil is spouted out in the water. As if it were transmitted, the upstream sand moves from up- to downstream around the bottom tip of the sheet pile and the sand model collapses with a loud sound.

4.3 Relationship between hydraulic head differences $H_c$ (by theory) and $H_f$ (by experiment)

From a precise discussion on seepage failure of soil in front of sheet piles, it is concluded that the deformation of soil represents unrecoverable damage and should be avoided in practice. The hydraulic head difference at the onset of deformation $H_f$ is so important for designing excavations of soil with a high ground water level that we discuss the value of $H_f (H_f = H_d)$. Fig. 9 shows the relationships between $D/T$ and $H_{\gamma}/(T+d)$ for no-excavation model, and Figs. 10 (a) and (b) show the relationships between $d/(D+d)$ and $H_{\gamma}/((T+d)\gamma')$ for excavation models, $D_1=35$ cm and 40 cm, respectively. The experimental results are also plotted in Figs. 9, 10 (a) and (b). It is observed from Fig. 9 and Figs. 10 (a) and (b) that the calculated critical hydraulic head differences $H_{PF}$ is very close to the measured $H_d$. The Prismatic failure concept 3D is proved to be a useful method for calculating the critical hydraulic head difference at onset of deformation of soil within a cofferdam. The greater the $d/(D+d)$ grows, the smaller the $H_f$ becomes. $H_c$ depends on the downstream penetration depth of sheet piles, $D$.

Fig. 7 shows the hydraulic head difference $H$ and the height of the soil on the surface height measurement line $Y$. It is found from the $H-Y$ curve that the height of the soil surface is constant with increasing $H$ until a certain value $H_f$. When $H$ increases beyond $H_f$, the soil surface begins to settle on the upstream side and rise on the downstream side. $H_f$ values are obtained on up- and downstream sides, $H_{fu}$ and $H_{fd}$, respectively. The smaller value of these two is taken as $H_f$, which is referred to as the hydraulic head difference at onset of deformation. Fig. 8 shows the relationship between $H_{fu}$ and $H_f$ obtained by a series of experiments. From Fig. 8, it is obvious that the experimental results lead to the interesting conclusion $H_{fu}=H_d$.  

![Fig. 6. A close-up photo of the upstream inverse conical shape at $H=37.70$ cm (E0320V)](image)

![Fig. 7. $H-Y$ curve (E0320)](image)

![Fig. 8. Relationship between $H_f$ and $H_{fu}$](image)
4.4 Self-stabilizing effect
As the hydraulic head difference $H$ increases, the shape of soil ground changes in the following way:

- Upstream subsidence and downstream rise
- Movement of soil particles from the up- to downstream side
- Progress of soil deformation
- Failure

After deformation, soil particles move successively from the up- to downstream side with increasing $H$. A new equilibrium of forces is reached in the downstream soil. $H_f$ depends on the penetration depth of the cutoff wall in the downstream, and $H_f - H_d$ depends on that in the upstream soil. The value of $H_f - H_d$ indicates the self-stabilizing effect. In axisymmetric and three-dimensional conditions, the self-stabilizing effect is larger than that in the two-dimensional condition and depends on the penetration depth of sheet piles on the upstream side $D/T$. The behavior of soil from deformation to failure continues until the potential supply of upstream sand is exhausted. So, the difference $H_f - H_d$ largely depends on the penetration depth of sheet piles on the upstream side.

The difference between $H_f$ and $H_d$ increases with increase in $D/T$ for the no-excavation model, whereas it is constant with increasing $d/(D+d)$ for excavation models. As presented by Tanaka et al. (2003), the $H_f - H_d$ value depends on the soil depth on the upstream side, and the phenomenon of soil from deformation to failure is referred to as self-stabilizing. In two dimensions, the equation $H_f = 1.106H_d$ is applied irrespective of $D/T$ values, which means that soil has self-stabilizing effect of about 11%. On the other hand, in an axisymmetric and three-dimensional conditions, the self-stabilizing effect depends on $D/T$, and increases from 29% to 75%, and from 19% to 53% with increasing $D/T (= 0.27$ to $0.60)$, respectively. Thus, in axisymmetric and three-dimensional conditions, the self-stabilizing effect is larger than in two dimensions, and becomes large with increases in $D/T$.

5 ESTIMATION OF $H_F$

5.1 No-excavation model
For the no-excavation model, the relationship between $H_f$ and $H_c$ is given as using Terzaghi’s theory (1943):

$$
\frac{H_f}{H_c} = f\left(\frac{D}{T}\right) + 1.0
$$

where $f(D/T)$ is a function of $D/T$. The relationship between $H_f/H_c$ and $D/T$ is given by the equation from the experimental results using the least-squares method:

$$
H_f = H_c \left(0.82 \frac{D}{T} + 1.0\right)
$$

The dashed line in Fig.9 shows Eq.(3), whose difference with $H_f$ by experiment is small (relative error, RE $\approx 0.027$).

5.2 Excavation models
For the excavation model, it is considered that the difference between $H_f$ and $H_d$ is almost constant regardless of the value $d/(D+d)$, and depends on $T_1$.
The value of $H_f$ for the two excavation models ($T_1 = 35$ cm and 40 cm) are given by averaging the values of $(H_f - H_y) / T_1$ as follows, respectively:

$$\frac{H_f}{T_1} = \frac{H_f}{T_1} + 0.24 \quad \text{(for } T_1 = 35 \text{ cm}) \quad (4)$$

$$\frac{H_f}{T_1} = \frac{H_f}{T_1} + 0.31 \quad \text{(for } T_1 = 40 \text{ cm}) \quad (5)$$

The dashed lines in Figs.10(a) and (b) show Eqs.(4) and (5), whose differences from $H_f$ by experiment are small ($RE \approx 0.045$ and 0.024), respectively.

The larger the $T_1$ (or $D_1$) grows, the larger the $H_f - H_y$ becomes. The behavior of soil from deformation to failure continues until the potential supply of upstream sands is exhausted. So, the difference $H_f - H_y$ largely depends on the penetration depth of sheet piles on the upstream side $D + d$.

6 CONCLUSIONS

For deep and/or large excavations of soil with a high ground water level, three-dimensional seepage flow through soil within a cofferdam is a problem. Three-dimensionally concentrated flow lowers the stability against seepage failure more than in the two-dimensional condition. To clarify the three-dimensional seepage failure mechanism, experiments were conducted in 20 cases of E0301–E0320 involving various thicknesses of soil, with or without excavation. Analyses of the FEM seepage flow and stability against the seepage failure of soil based on the prismatic failure concept 3D were also carried out, and the following results were obtained:

1. With respect to seepage failure, soil is most critical at a bisector of the right angle of the inside corner of the rectangular diaphragm wall.
2. The hydraulic head difference at an abrupt change of the $H - Q_{15}$ curve, $H_o$, is nearly equal to the hydraulic head difference at the onset of deformation, $H_y$.
3. The theoretical critical hydraulic head difference based on the prismatic failure concept 3D, $H_c$, is nearly equal to $H_o (\approx H_y)$.
4. The difference between $H_o$ and $H_y$, i.e., the margin from deformation to failure of soil, which means the self-stabilizing effect, increases with an increase in the downstream penetration depth of sheet piles, $D$, in the case of no excavation, while it is constant in the case of excavation for a constant value of the upstream penetration depth of sheet piles, $D_1$.
5. The upstream soil surface is an inverse conical shape centered at the outer corner of the rectangular wall. The slope angle increases with an increase in the hydraulic head difference, $H$, from $45^\circ$ immediately after the deformation of soil to $52^\circ$ just before failure for the test E0320.
6. In the downstream soil, the critical prism withstands the increasing head difference due to sand particles supplied from upstream.

7. $H_c$ depends on the downstream penetration depth of sheet piles, $D$, and the self-stabilizing effect depends on that upstream, $D_1$.

8. $H_f$ can be estimated using the theoretical critical hydraulic head difference, $H_c$.

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