MODEL TESTS AND NUMERICAL SIMULATION OF BRACED EXCAVATION IN SANDY GROUND: INFLUENCES OF CONSTRUCTION HISTORY, WALL FRICTION, WALL STIFFNESS, STRUT POSITION AND STRUT STIFFNESS

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ABSTRACT

Two-dimensional model tests and elastoplastic finite element analyses for excavation problems with a braced wall are performed to investigate the influence of construction history, wall friction, wall stiffness and the struts on the ground movements and the earth pressures on the wall. Test results show that the surface settlements of the backfill and earth pressure on the wall depend on the deflection process of the wall, even if the final wall deflections are the same. The wall friction, wall stiffness and the position and stiffness of strut significantly affect the earth pressure on the wall and the ground movement including the surface settlements. The computed results, in which mechanical behavior of soil and structure, behavior of interface between soil and wall and the process of excavation were properly taken into consideration, describe well the results obtained in the model tests. Such influence on the deflection of the wall, the earth pressure on the wall and the ground movements has not been evaluated properly in the present design practice.

Key words: earth pressure, excavation, finite element method, model test, retaining wall, settlement (IGC: E2/E5/H2)

INTRODUCTION

Surface settlement of the ground caused by open excavation becomes a serious problem particularly in urban areas. The question of how to evaluate the earth pressure on the braced wall is also a problem in practical design. It is then necessary to properly take into consideration not only the mechanical behavior of the soil but also the influence of the wall friction, the wall stiffness and the excavation process, in order to predict the ground movements and the earth pressure under excavation. However, in the present design method, the earth pressure is usually predicted by a conventional method such as a frame model together with Rankine's earth pressure theory. There is no appropriate method to predict the surface settlement of the ground. Furthermore, we do not pay much attention to the influence of the wall friction and the excavation process when predicting the ground movements and the earth pressure due to excavation.

One of the authors has developed elastoplastic constitutive models for sand (Nakai, 1989; Nakai, Fujii and Taki, 1989; Nakai, Taki and Funada, 1993) and an elastoplastic interface model between structure and soil (Nakai, 1985), which are capable of describing various soil and slip behaviors properly. We have carried out not only numerical analyses of fundamental excavation problems using these constitutive models, but also the corresponding model tests (Nakai, Kohno and Hashimoto, 1996). In the present study, whilst considering the braced excavation problem of ground to be a soil-structure interaction problem, we performed two-dimensional model tests and corresponding elastoplastic finite element analyses. The behavior of the excavation, including the deformation of the retaining wall, earth pressures, ground movements and others are discussed, through the experimental and analytical results.

Throughout this paper, the term "stress", "pressure" and "unit weight" are interpreted as effective values as

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well as total values, because every model test and analysis was carried out in a dry condition (drained condition).

DESCRIPTION OF MODEL TESTS

Model Tests to Investigate the Influence of Wall Deflection Process (Series A)

Figure 1(a) shows the two-dimensional apparatus for simulation of excavation problems (Nakai, Kohno and Hashimoto, 1996). The size of the model ground is 450 mm in height and 480 mm in width. The wall on the right hand side consists of 13 slide blocks (30 mm in height) which may move independently in the horizontal direction. The movements of two blocks beneath these slide blocks are always fixed. Every side block is individually supported by roller bearings to move without frictional resistance. Any mode and procedure of wall deflection, therefore, can be applied. A mass of aluminum rods (unit weight $\gamma = 21.1 \text{ kN/m}^3$), in which two kinds of rods having diameters of 1.6 and 3.0 mm and a length of 5 cm are mixed in the weight ratio of 3:2 is used as a two-dimensional model of the backfill soil. Figure 2 shows the observed stress-strain-dilatancy relations of biaxial tests on the mass of aluminum rods under constant minor principal stress ($\sigma_2 = 78 \text{ kPa}$) and constant major principal stress ($\sigma_1 = 78 \text{ kPa}$). The data are arranged in terms of the relation between major minor principal stress ratio (σ_1/σ_2), major principal strain (ϵ_1) and volumetric strain (ϵ_v). We can see that the mass of aluminum rods behaves like a medium or dense sandy soil with negative and then positive dilatancy. Wall deflection is controlled by micrometers behind these movable slide blocks, and earth pressure on the wall is measured by load cell set up between each slide block and micrometer. The initial horizontal stress distribution also can be found with these load cells. Surface settlements are measured continuously with a laser type displacement transducer (accuracy of 0.01 mm). The laser type displacement transducer is mounted on a slide shaft over the aluminum rod mass and is movable on the slide shaft. The position of

the laser type transducer is measured by a supersonic wave type transducer. All the data of surface settlements, positions and earth pressures are automatically collected by a data logger connected to a personal computer. Furthermore, the movements of the ground as a whole can be found by taking photographs of the mass of aluminum rods. The following three types of wall deflection are employed in the present experiments to investigate the influence of the wall deflection process (displacements are imposed to the upper ten movable slide blocks in Fig. 1):

- (1) type I: The same displacement is applied simultaneously to all of the slide blocks.
 - (2) type II: The same displacement is applied one by one from upper slide block to lower slide block.
 - (3) type III: The same displacement is applied one by one from lower slide block to upper slide block.
- Therefore, at the completion of wall movement, the mode and magnitude of wall deflection in types I, II and III is the same.

Model Tests to Investigate the Influence of Wall Friction and Wall Stiffness and Strut (Series B)

In this series, the model wall is installed in the mass

Fig. 2. Observed stress-strain-dilatancy relation of aluminum rods mass under biaxial compression

Fig. 1. Apparatus for simulation of excavations: (a) test to investigate influence of wall deflection process, (b) test to investigate influence of wall friction, wall stiffness and strut

Photo. 1. Test of braced excavation with a strut

200 mm away from the right-hand side. The excavation process is simulated in such a way that the mass of aluminum rods on the right side of the wall is then removed step by step from the surface to the bottom (see Fig. 1(b)). Three types of model wall (aluminum plate), with thicknesses of 0.5 mm, 1.0 mm and 2.0 mm, a width of 60 mm and a height of 300 mm, are used. The initial horizontal stress in the ground and strut load are measured by a load cell set up at the back of each block on the right-hand side. Surface settlement can be measured continuously with a laser type displacement transducer in the same way as series A. Strain gages are glued at both side of a model wall so that the bending strain, from which the earth pressure and the deflection of wall can be evaluated, can be measured. Furthermore, to investigate the influence of wall friction, both a wall with a flat surface and a wall on which aluminum rods are glued are used. Here, aluminum rods of 1.6 mm in diameter are glued at an equal spacing of 20 mm. Between every two 1.6 mm

rods, there are intermediate rods of 3.2 mm in diameter. The flat surface is called "smooth", and the surface on which aluminum rods are glued "rough". It has been checked that the glued aluminum rods do not affect on the stiffness of the wall. Two kinds of struts are provided—one, very stiff (spring stiffness $k \approx \infty$) and the other, less stiff ($k \approx 0.14 (\times 0.98 \text{ kN/m})$)—in case of excavation with struts. The constant k represents the stiffness estimated per 10 mm in depth. The "stiff" strut is an aluminum square bar (6 mm $\times$ 6 mm $\times$ 200 mm), and the other is one in which a coil spring is placed between two aluminum bars as shown in Fig. 1(b). Photograph 1 shows a test of a braced excavation with a strut.

METHODS OF ANALYSES

Plane strain finite element analyses are carried out using the mesh in Figs. 3. Here, Fig. 3(a) is for series A, Fig. 3(b) for series B. As the stress level of the model tests is very small and the material parameters are not easy to determine, the soil in the numerical analysis is assumed to be medium dense Toyoura sand and the size of the analysed mesh is 100 times as large as the model tests. The ground material is simulated with kinematic t_{ij} -sand model (Nakai, Fujii and Taki, 1989; Nakai, Taki and Funada, 1993). The initial stresses in the ground are calculated from the unit weight of sand ($\gamma = 15.5 \text{ kN/m}^3$) and the coefficient of earth pressure at rest ($K_0 = 0.57$) which can be determined theoretically from the constitutive model used in the analysis. For the numerical analysis of series A, the same model geometry and same deflection process of the wall as the model tests are applied to the right-hand side boundary, and the wall is assumed to be smooth. In series B, the retaining wall, which is introduced between A and B in Fig. 3(b), is modelled using beam elements, and elastoplastic joint elements (Nakai, 1985) are introduced between the wall and the ground to

Fig. 3. Finite element meshes: (a) mesh to investigate influence of wall deflection process, (b) mesh to investigate influence of wall friction, wall stiffness and strut

investigate the influence of wall friction. The friction angle δ between the wall and the ground is assumed to be 10° in case of “smooth” surface, and 40° in case of “rough” surface. As extreme cases, analyses of $\delta=0^\circ$ (perfectly smooth) and $\delta=90^\circ$ (no slip along the wall) are also performed. The stiffness of the strut is simulated with an elastic support. Excavation is simulated by taking out the solid elements at the right of the wall from top to bottom. The bending stiffnesses of the wall are assumed to be $EI=1.0 \times 10^4$, 1.0×10^5 and 1.0×10^6 ($\times 9.8$ kNm²). Since it is difficult in reality to quantify the friction angle of the wall and the stiffness contrast between ground and wall, the above values of wall friction and wall stiffness are assumed in the analysis.

ELASTOPLASTIC MODELS FOR SOIL AND INTERFACE USED IN THE ANALYSES

Constitutive Model for Sand

The model for sand used in the analyses is the so-called kinematic t_{ij} -sand model which has been fully described elsewhere (for example, Nakai, Fujii and Taki, 1989; Nakai, Taki and Funada, 1993). This model is an extension of an isotropic hardening model called t_{ij} -sand model (Nakai, 1989) to a kinematic hardening model. So, only an outline of the kinematic t_{ij} -sand model is given here.

This model is capable of describing of the following typical characteristics of sand:

- ① Influence of intermediate principal stress on the deformation and strength of sand (unified description of stress-strain curve and failure condition)
- ② Influence of stress path on the direction of plastic flow (stress path dependency of stress-dilatancy relation)
- ③ Positive and negative dilatancy
- ④ Induced anisotropy by stress history (stress-strain behavior under cyclic loading and rotation of principal stress axes)

In this model, “① Influence of intermediate principal stress” can be taken into consideration by formulating the yield function and others based on the modified stress tensor t_{ij} instead of the usual stress tensor σ_{ij} and by assuming the flow rule in t_{ij} space rather than in σ_{ij} space. “② Influence of stress path” is considered by assuming that the total plastic strain increment $d\epsilon_{ij}^p$ consists of the component satisfying associated flow rule $d\epsilon_{ij}^{p(AF)}$ and the component of isotropic compression $d\epsilon_{ij}^{p(IC)}$. For describing “③ Positive and negative dilatancy”, the plastic work based on t_{ij} -concept ($W^p = \int t_{ij} d\epsilon_{ij}^p$) is employed as a strain hardening parameter. “④ Stress induced anisotropy” can be considered in the model by assuming the kinematic hardening rule in stress ratio space.

The values of soil parameters for the kinematic t_{ij} -sand model for medium dense Toyoura sand are listed in Table 1. These can be determined from a drained conventional triaxial compression test after isotropic compression. Figure 4 shows the results (triangles) of triaxial compression and extension tests under constant mean principal stress together with the calculated curves, ar-

Table 1. Values of soil parameters for medium dense Toyoura sand

C_i	0.0084
C_e	0.0060
m	0.3
$R_f \equiv (\sigma_1 / \sigma_3)_{f(comp)}$	4.7
$D_f \equiv (d\epsilon_v / d\epsilon_1)_{f(comp)}$	-0.6
α	0.85
ξ	0.3

Fig. 4. Observed and calculated stress-strain relations in triaxial compression and extension tests on Toyoura sand

ranged with respect to principal stress ratio (σ_1 / σ_3), principal strains (ϵ_1 and ϵ_3) and volumetric strain (ϵ_v). The model describes well the observed stress-strain behavior using three-dimensional stresses including dilatancy characteristics. Figure 5 shows the calculated curves of plane strain tests on medium dense Toyoura sand under constant minor principal stress ($\sigma_3 = 196$ kPa) and constant major principal stress ($\sigma_1 = 196$ kPa). Figure 5(a) shows the relations similar to those of the observed results of aluminum rods mass in Fig. 2, and Fig. 5(b) shows the variations of relative magnitude of intermediate principal stress with principal strain. It can be seen from Figs. 2 and Fig. 5(a) that the mechanical behavior of the material (aluminum rods mass) used in the model tests is well described qualitatively, by the present constitutive model for sand. Furthermore, the calculated stress ratio at failure $((\sigma_1 / \sigma_3)_{f(plane strain)} \approx 5.8)$ under the plane strain condition is a little larger than that $((\sigma_1 / \sigma_3)_{f(comp.)} = (\sigma_1 / \sigma_3)_{f(ext.)} = 4.7)$ under triaxial compression and extension conditions as shown in Figs. 4 and 5, and the parameter $b = (\sigma_2 - \sigma_3) / (\sigma_1 - \sigma_3)$ under the plane strain condition converges to a value between 0.3 and 0.4. These calculated characteristics of sand under plane strain correspond to the observed results reported before (e.g., Green and Read, 1975) and can be described by introducing the t_{ij} -concept into constitutive modeling.

Interface Model between Wall and Soil

On the interface between wall and soil, slip does not oc-

Fig. 5. Calculated results: stress-strain-dilatancy curves of Toyoura sand under plane strain condition: (a) stress-strain-dilatancy curves, (b) variations of relative magnitude of intermediate principal stress

Fig. 6. Schematic diagram of stress and displacement at interface between wall and soil

cur until the shear-normal stress ratio on the interface (p_s/p_n) reaches the coefficient of friction $\tan \delta$ (δ = friction angle on the interface). On the other hand, after the stress ratio (p_s/p_n) reaches $\tan \delta$, slip occurs on the inter-

face, keeping the condition of $p_s/p_n = \tan \delta$ (see Fig. 6). To model such behavior on the interface, elastoplastic property matrix between the stress increments (dp_s, dp_n) and relative displacement increments (dw_s, dw_n) has been developed (Nakai, 1985). Here, the increments dw_s and dw_n denote the shear and normal components of the relative displacement increment on the joint element.

By introducing this property matrix into joint elements (e.g. Goodman, Taylor and Brekke, 1968), the slip behavior on the interface shown in Fig. 6 can be analyzed.

RESULTS AND DISCUSSIONS

Influence of Wall Deflection Process (Series A)

Figures 7 compare the observed profiles of surface settlements of the backfill at the completion of wall movement (wall displacement $d = 10$ mm) of types II and III with that of type I. Here, the distance x from the wall is normalized with respect to the wall height H_0 ($= 300$ mm), and the settlement s with respect to the wall displacement d . The settlements in type II and III are much larger than those of type I just in the vicinity of the wall, but occur locally. On the other hand, the settlements in type I extend to a wider range. Figures 8 are the computed results (wall displacement $d = 50$ mm) corresponding to the observed results in Figs. 7. The computed settlements describe well the observed difference of the wall deflection process.

Figures 9 show the comparison of the observed distributions of horizontal earth pressure on the wall at the completion of wall movement. The elevation h from the bottom of the wall and the horizontal earth pressure p_x are normalized with respect to H_0 and γH_0 , respectively (γH_0 corresponds to the initial vertical stress at the bottom of the wall). Though the profiles of surface settlements of type III and type II resemble those shown in Figs. 7, the shape of earth pressure distribution in type III is similar not to that of type II but to that of type I. The shapes of the earth pressure distribution in type I and III are rather triangular (similar to Rankine's distribution), but the shape in type II is barrel like. The computed earth pressure distributions in Figs. 10 agree well with those observed in Figs. 9. It can be seen from Figs. 7 to 10 that surface settlements and earth pressures depend greatly on the mode of wall deflection process, even though the final deflections of the wall are similar. Although the material in the ground, stress levels, the stress condition and the magnitude of displacement assumed in the analyses are different from those in the model tests, the analytical results show well the influence of the mode of wall deflection process on the settlements and earth pressures. Since actual open excavation is done from ground surface, the actual mechanism is more likely to be type II rather than type I. The present study suggests that actual settlements are larger and more local than those estimated, irrespective of the consideration of construction procedure, and that actual earth pressures are also different from those estimated in actual design. It is also noted that elastic analysis cannot describe the observed differences due to construction procedure.

Fig. 7. Observed profile of surface settlements: (a) type I and type II, (b) type I and type III

Fig. 8. Computed profile of surface settlements: (a) type I and type II, (b) type I and type III

Influence of Wall Friction and Wall Stiffness (Series B)

Figure 11 shows the observed deflection of the retaining wall without struts for different wall roughness. Here, the depth of excavation d from the surface is half that of the wall height H_0 ($d/H_0 = 0.5$). In this figure, the vertical axis (h/H_0) and the horizontal axis (u/H_0) represent the distance from the bottom of the wall and the deflection of the wall, normalized by the wall height. It can be seen from this figure that wall friction has the effect of reducing the deflection. Figure 12 shows the experimental results for different stiffnesses of wall. The deflection does not decrease in inverse proportion to the increase in bending stiffness of the wall. Figures 13 and

14 are the analytical results corresponding to those observed in Figs. 11 and 12, respectively. The results of $\delta = 40^\circ$ and $\delta = 90^\circ$ in Fig. 13 are the same. On the other hand, there is considerable difference between $\delta = 0^\circ$ and $\delta = 10^\circ$ indicating that wall friction contributes very much to the reduction in wall deflection even if the friction is small. Although the material and the scale are different from those of the model tests, these analytical results describe well the influence of wall friction and stiffness. In practical design, though not much attention is paid to wall friction, it is important to consider it for adequate prediction. The results in Figs. 12 and 14 mean that the distribution and magnitude of earth pressures depend

Fig. 9. Observed distribution of earth pressure on wall: (a) type I and type II, (b) type I and type III

Fig. 10. Computed distribution of earth pressure on wall: (a) type I and type II, (b) type I and type III

Fig. 11. Observed deflection of retaining wall with different wall friction

Fig. 12. Observed deflection of retaining wall with different wall stiffness

Fig. 13. Computed deflection of retaining wall with different wall friction

Fig. 14. Computed deflection of retaining wall with different wall stiffness

on the wall stiffness.

Figure 15(a) shows the observed profile of surface settlements of the backfill at different excavation depths in the case of a smooth wall with thickness, $t=1.0$ mm. The distance x from the wall and the ground settlement s are normalized by the wall height H_0 . The observed surface settlement is not proportional to the excavation depth but increases sharply with the depth of excavation. Figure 15(b) shows the results in the case of rough wall with thickness, $t=1.0$ mm. The wall friction inhibits the surface settlement. Figure 15(c) shows the results in which the wall thickness is twice the value of that in Fig. 15(a). The comparison between Figs. 15(a) and (c) indicates that the profile, as well as the magnitude of the surface settlement, depends on the wall stiffness. Numerical results corresponding to these tests are shown in Figs. 16(a) to (c). It can be seen that the computed surface settlements describe well the observed tendency of settlements influenced by the wall friction and the wall stiffness.

Figures 17(a) to (c) show the observed earth pressure acting on the retaining wall at the excavation depth $d/H_0=0.2$ and 0.4 , corresponding to the test conditions in

Fig. 15. Observed surface settlements

Fig. 16. Computed surface settlements

Figs. 15(a) to (c). The vertical axis represents the normalized distance from the bottom of the wall. Earth pressure evaluated from the observed bending strain is indicated by an equivalent nodal force equally spaced at 30 mm. Though the actual earth pressure on the wall changes continuously with depth, the earth pressure in these figures is

Fig. 17. Observed distribution of earth pressure on wall

Fig. 18. Computed distribution of earth pressure on wall

indicated as the discontinuous nodal forces calculated from the bending moments observed with a spacing of 30 mm in depth. Therefore, the nodal forces on the right and left in these figures are not exactly balanced. Here, the nodal forces above the excavation level shown as broken lines represent the active earth pressure at backfill side, but those below the excavation level represent the difference between the earth pressures at backfill side and excavation side. Although the earth pressure at excavation side (passive side) is larger than that at backfill side below the excavation level, the earth pressure acting on the flexible wall at backfill side (active side) becomes larger than that at excavation side near the bottom of the

Fig. 19. Observed deflection of retaining wall (with strut)

Fig. 20. Observed surface settlements (with strut)

wall. It is also found from the comparison between Figs. 17(a) and (c) that the earth pressure on the excavation side increases with increasing bending stiffness of the wall. The computed results in Figs. 18(a) to (c) also show similar tendency of the observed earth pressure. The computed earth pressures acting on the wall at backfill side and excavation side are indicated individually in the figures. In the present design practice, a triangular or trapezoidal shape of earth pressure is always assumed, and the influence of deflection mode of wall, wall friction and wall stiffness on the earth pressure is not taken into account property.

Influence of Stiffness and Position of Struts (Series B)

In the model tests of an excavation with bracing, aluminum plate with a thickness of 0.5 mm and a smooth surface is used as the wall. Figures 19 to 21 show the experimental results of the wall deflection, the surface settlement and the earth pressure on the wall, in case of a stiff strut being set up at the position of $d_s/H_0 = 0.15$ (d_s :

Fig. 21. Observed distribution of earth pressure on wall (with strut)

Fig. 22. Computed deflection of retaining wall (with strut)

depth of the strut from the ground surface). In every model test and analysis, the strut is set up when the depth of excavation d/H_0 becomes $d_s/H_0 + 0.05$. The computed results shown in Figs. 22 to 23 have a good qualitative agreement with the experimental results shown in Figs. 19 to 21, respectively. These experimental and analytical studies indicates that not only the wall deflection but also the profile of surface settlement and the earth pressure distribution on the wall depend on whether the wall is braced or not.

The effect of the position of strut was also investigated in this study. Figures 25 show the observed variations of the axial force in the strut with the depth of excavation. Here, Fig. 25(a) shows the results with different position

Fig. 23. Computed surface settlements (with strut)

Fig. 24. Computed distribution of earth pressure on wall (with strut)

Fig. 25. Observed variations of axial force of bracing with increasing excavation depth

Fig. 26. Computed variations of axial force of bracing with increasing excavation depth

of the strut and the same stiffness of strut ($k \approx \infty$), whereas Fig. 25(b) shows the results with the same position ($d_s/H_0=0.15$) but different stiffnesses of the strut. Figures 26 are the computed results corresponding to these experiments. The axial force of the strut increases almost proportionally to the depth of excavation in every case. It is seen from Fig. 26(a) that the deeper the position of strut is, the larger the increase of axial force with excavation becomes. Furthermore, Fig. 26(b) shows that the stiffness of the strut does not influence the axial force in the bracing very much except in the case of extremely soft bracing.

CONCLUSIONS

Main results of the present experimental and numerical study on braced excavation are summarized as follows:

(1) Using the results of the 2-D model tests in which the wall deflection process can be changed, it is shown that the settlements of ground and earth pressure on the retaining wall are much influenced by the construction procedure, even though the final wall deflection is the same. For example, the surface settlements of the ground when the wall moves sequentially from upper to lower (similar to actual excavation process) is larger in the vicinity of wall and more local than those when the wall moves simultaneously in both upper and lower parts. The earth pressure distribution in the case of sequential

wall movement from upper to lower is not similar to Rankine's triangular shape but to barrel shape.

(2) It is shown through the 2-D model tests that the wall friction, the stiffness of the wall and the strut significantly influence the behavior of the wall and the ground—the wall deflection, the earth pressure and the surface settlement during excavation. For example, the wall deflection and the surface settlement of the ground decrease with increasing the wall friction even if the wall stiffness is the same. The magnitudes of wall deflection and surface settlement do not necessarily vary inversely with the magnitude of bending stiffness of wall. The axial load in the strut increases almost linearly with the depth of excavation, and the deeper is the strut, the larger is the increase. However, the stiffness of the strut has little effect on the axial load in the strut.

(3) Numerical analyses in which the mechanical behavior of soil and structure, the frictional behavior of the interface between soil and structure and the excavation process are properly taken into consideration may predict well the excavation behavior observed in model tests.

In order to predict precisely the behavior of ground under excavation, it is necessary to carry out the analysis in which the construction procedure, elastoplastic characteristics of the soil including soil dilatancy and the frictional characteristics between soil and wall are properly taken into account.

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