BEHAVIOR OF GROUND AND RESPONSE OF EXISTING FOUNDATION DUE TO TUNNELING

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ABSTRACT

An apparatus has been developed to model the excavation of a tunnel in the laboratory. With this apparatus, 2D model tests are carried out to investigate the surface settlement and the earth pressure brought about by the tunneling. Finite element analyses using an elastoplastic subloading t_{ij} model are also conducted. The influence of volume loss on the surface settlement and the earth pressure, due to the shallow tunnelling, is illustrated based on the model tests and the corresponding numerical analyses. It is revealed that the surface settlement troughs and the earth pressure distributions around shallow tunnels depend on both the volume loss and the crown drift of the tunnel. The effect of the interaction between the tunneling and existing nearby foundations is also demonstrated in this paper. For existing foundations, the building loads control the surface settlements and the zone of deformation during the tunnel excavation. The behavior of the foundations depends on the deformation mechanism of the ground during the tunnel excavation. The induced axial force and bending moments in the piles of a piled raft are investigated numerically, and it is shown that the axial force changes due to the stress relaxation of the ground. Bending moments are induced in the piles at a lower value of soil cover due to the differential settlement of the piled raft.

Key words: constitutive equation of soil, earth pressure, excavation, (existing building foundation), finite element method, model test, settlement, tunnel (IGC: E2/E5/E12/E13/E14)

INTRODUCTION

Tunnel excavations induce deformation in the surrounding ground and this may lead to the serious damage of nearby structures. In Japan and many other countries, the earth pressure acting on the tunnel lining and its stability are still being predicted with a rigid plasticity approach, such as Terzaghi's loosening earth pressure theory. The surface settlements due to tunneling are often predicted using empirical approaches, such as Gaussian distribution curve fitting (Peck, 1969), assuming a given volume loss due to the tunneling. In these processes, the influence of the excavation patterns and the effect of the interaction between the ground and existing structures are not taken into account. In the past decade, there have been considerable improvements in the use of numerical methods to simulate tunnel excavations, although many works still use simple constitutive models, such as linear elastic or elastic perfectly plastic Mohr-Coulomb models. Boscarring and Cording (1989), Franzius et al. (2004), Jacobsz et al. (2004) and many other researchers have studied how the soil-structure interaction can be simulated during a tunnel excavation. However, further investigations are necessary using well-controlled tests and

simulations with robust constitutive models.

This paper describes the influence of excavation patterns and soil-structure interaction on the earth pressure and ground movements during a tunnel excavation. For this purpose, 2D model tests and corresponding finite element analyses are conducted. Two-dimensional finite element analyses are carried out with an academic program called FEMtij-2D, using the elastoplastic subloading t_{ij} model (Nakai and Hinokio, 2004). This model can describe the typical stress, deformation and strength characteristics of soils, such as the influence of the intermediate principal stress, the influence of the stress-path dependency of the plastic flow and the influence of the density and/or the confining pressure. Numerical analyses are also performed with a linear elastic theory to compare the results with those of the elastoplastic analyses.

Using the t_{ij} model to simulate the behavior of a mass of aluminium rods, Shahin et al. (2004a) showed that it was possible to adequately reproduce the influence of an excavation sequence in a tunnel construction compared to laboratory results obtained from two-dimensional (2D) physical models using a trapdoor apparatus. This experiment was later improved to investigate the effect of

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the existing building load, simulated by a loaded plate, and the results of the numerical analyses showed that the constitutive model can precisely reproduce the observed patterns of surface settlement and earth pressure under two-dimensional conditions, as demonstrated by Nakai et al. (2005). Sung et al. (2006) showed the capability of the constitutive model to predict the general patterns of ground behavior in model tests with existing piled and non-piled raft foundations.

LAYOUT OF MODEL TESTS

Apparatus of Model Tests

Figure 1(a) shows a schematic diagram of the 2D tunnel apparatus. Figure 1(b) represents the newly developed model tunnel with a circular cross section; it was constructed based on a device described by Adachi et al. (1994). The tunneling device consists of a central shim (a shaft with a variable cross section) surrounded by twelve segments. The segments are strongly tightened all around the shim with rubber bands. One motor is attached to the shim to pull it out in a horizontal direction, thus shrinking the tunnel and causing the soil mass to converge to the tunnel center. The model tunnel is held by a vertical shaft, and it can be moved in a vertical direction with a second motor. Therefore, the device consists of two motors—one for shrinking the tunnel and the other for moving the tunneling device in a vertical direction to fix it at a chosen ground depth. It is possible to make these motors work simultaneously and to control their speed. The

shim is gradually pulled out by the motor, which changes the core cross section. Consequently, the segments move inwardly and the diameter of the tunnel is reduced. By changing the shape of the shim, different kinds of excavation processes, such as the full-face excavation, the top and side drift excavation and the bench cut excavation, can be reproduced with this apparatus. The reduction in tunnel diameter and the amount of radial shrinkage are obtained from a dial gauge reading. The vertical movement of the tunnel, if required, is also measured with another dial gauge. Therefore, the shrinkage of the tunnel can be attained in a controlled manner, which can approximate the relaxation of the ground towards the tunnel that occurs during the tunnel excavation due to the movement of the ground towards the excavation face, the radial movement of the unsupported ground before the installation of the lining and the deformation of the lining.

Twelve load cells are used to measure the earth pressure acting on the tunnel. The load cells are embedded in the blocks which represent the tunnel lining. The blocks measure 2.35 cm in width and 5.0 cm in length; they are tightly fastened with rubber bands. Including the load cell blocks, the total diameter of the model tunnel is 10.0 cm. The circular tunnel device is held in place at a certain height above the ground of an iron table over which the ground is filled, as will be explained in the next section. Therefore, the earth pressure can be obtained at twelve points on the periphery of the tunnel at one time. After setting the physical model, the excavation is simulated by controlling the two motors that move and shrink the tunnel device, as explained above. The resulting surface settlement of the ground is measured using a laser type of displacement transducer with an accuracy of 0.01 mm. Its position in the horizontal direction is monitored with a supersonic wave transducer. Photographs are taken during the experiments and are used later as input data for the determination of ground movements with a program based on the technique of Particle Image Velocimetry (PIV).

To simulate the building loads, a strip footing and a piled raft are used to model the foundations. The strip footing is made of an iron plate, 8 cm in width and 1 cm in thickness. The cap of the piled raft is made of an aluminum plate, 8 cm in width and 2 cm in thickness. The piles are simulated using polyurethane walls, because the model is two-dimensional under plane strain conditions. The Young's modulus of the pile material ($E = 1.06 \times 10^5$ kN/m²) and the distance between two piles are chosen to agree with the similarity ratio of 1:100 used throughout the model. For example, the model tunnel, 10 cm in diameter, is employed to represent a real tunnel, 10 m in diameter. The thickness of the piles is 0.5 cm, the length of the piles (L_p) is 10 cm and the distance between the front and rear piles is 5 cm. To impose the existing load, a constant dead load value of $q_v = 0.32 (\times 9.8 \text{ N/cm})$ is placed in the center of the foundation before performing the tunnel excavation. This load is kept fixed throughout the tests; it was estimated from previous loading tests per-

Fig. 1(a). Schematic diagram of 2D tunnel apparatus

Fig. 1(b). Circular tunnel device

Fig. 2. Observed load-displacement curves for model ground

formed on the model ground, as illustrated in Fig. 2. The applied load of $q_v = 0.32$ ($\times 9.8$ N/cm) is around 1/3 of the ultimate bearing capacity of the ground with a piled raft and 1/2 of the bearing capacity of the strip footing.

Model Ground and Excavation Patterns

Firstly, the tunnel device was set at a height of 10 cm measured from the bottom of the iron table to the tunnel invert. Several experiments were conducted varying the distance between the tunnel invert and the bottom boundary. It was found that this height (10 cm) was enough to avoid the influence of the bottom boundary. After setting the tunnel device, aluminum rods were stacked up to the prescribed height. The rods were 1.6 and 3.0 mm in diameter and mixed at a ratio of 3:2 in weight. The unit weight of the aluminum rod mass was 20.4 kN/m³ and the length was 5.0 cm. The properties of the ground mass consisting of aluminium rods were similar to the properties of dense sand (Shahin et al., 2004a).

Considering the adopted scale of 1:100, the diameters of the rods would correspond to particles with grain sizes of 16 and 30 cm, both of which are rather large for real soil. However, the particle sizes are much smaller than the real tunnel diameter (10.0 m) or the foundation block width (8.0 m) by a factor of 50 and 30 times, respectively. This should be enough to avoid scaling effects, as demonstrated by Roberts (1994), which show that a relation of 1:20 between particle size and the extension of averaging length should be enough to compute quantities, such as density, of particulate media taken as a continuum. The international standard for rocks to be tested in triaxial compression, for instance, suggests that the diameter of a specimen should be related to the size of the largest grain by a factor of at least 10:1, whenever possible (ISRM, 1978).

The bottom of the iron table, over which the mass of aluminium rods was placed, had ten moveable blocks used in the trap door tunnel experiments (Nakai et al.,

Fig. 3. Schematic explanation of excavation patterns

1997; Shahin et al., 2004a; Sung et al., 2006). The reason for using this type of base was to adjust the initial stress conditions of the ground in such a way that the stress distribution would become similar to that of the ground without a tunnel (K_0 stress conditions for the mass of aluminum rods). This was achieved by adjusting the moveable blocks at the bottom of the apparatus until a uniform vertical stress distribution, corresponding to the self-weight of the ground, was obtained. In the model test ground, the value of K_0 was about 0.70 and it was controlled by the moveable blocks at the bottom of the apparatus. Great care was taken to make a uniform ground and not to apply any undesired load on the ground.

In this study, two types of excavation patterns were considered for both the greenfield condition and the ground with existing building loads. These patterns are illustrated in Fig. 3, in which d_r represents the amount of shrinkage in the radial direction towards the center of the tunnel and d_c indicates the downward translation of the tunnel center. Pattern I corresponds to the excavation in which the center of the tunnel was kept fixed ($d_c = 0$) and the diameter of the tunnel was reduced, shrinking $d_r = 4$ mm all around the tunnel, as shown in Fig. 3(a). Pattern II represents the excavation in which the invert was kept fixed (top drift excavation), as shown in Fig. 3(b). This was achieved by lowering the tunnel device itself, $d_c = 4$ mm, during the application of shrinkage. Here, the same amount of shrinkage ($d_r = 4$ mm) was applied, but the overall excavation pattern resulted in a top drift of 8 mm and null invert movement. In real tunnel excavations, however, the deformation mode is in between the above-mentioned excavation patterns. In both excavation patterns, the final volume loss around the tunnel was the same, equal to 15.36%, in order to investigate the deformation behavior of the ground with such a huge volume loss. However, the data on different volume losses, starting from 0.20%, were recorded in the experiments. The model tests were conducted for four overburden ratios, namely, D/B equals 0.5, 1.0, 2.0 and 3.0, where D is the depth from the ground surface to the top of the tunnel and B (10 cm) is the diameter of the tunnel, as shown in Fig. 1(a).

NUMERICAL ANALYSES

Outline of the Subloading t_{ij} Constitutive Model

An elastoplastic constitutive model for soil, called the subloading t_{ij} -model (Nakai and Hinokio 2004), was used

in the finite element analyses. To take into consideration the influence of the intermediate principal stress, the model was formulated using modified stress tensor t_{ij} (Nakai and Mihara, 1984), which is defined as

$$t_{ij} = a_{ik} \sigma_{kj} \quad (1)$$

in which a_{ik} is the non-dimensional symmetric tensor whose principal values are given by the direction cosines of the normal to the spatially mobilized plane (briefly SMP; Matsuoka and Nakai, 1974).

$$a_1 = \sqrt{\frac{I_3}{I_2 \sigma_1}}, \quad a_2 = \sqrt{\frac{I_3}{I_2 \sigma_2}}, \quad a_3 = \sqrt{\frac{I_3}{I_2 \sigma_3}} \quad (2)$$

where σ_i ($i=1, 2, 3$) are the three principal stress levels, and I_2 and I_3 are the second and third invariants of σ_{ij} . The principal axes of t_{ij} coincide with those of σ_{ij} , because the principal axes of a_{ij} and σ_{ij} are identical. The yield function is given by the following equation as a function of mean stress t_N and stress ratio $X \equiv t_s/t_N$ based on the t_{ij} concept:

$$\begin{aligned} f &= \ln t_N + \zeta(X) - \ln t_{N1} \\ &= \ln \frac{t_N}{t_{N0}} + \zeta(X) - \left(\ln \frac{t_{N1e}}{t_{N0}} - \ln \frac{t_{N1e}}{t_{N1}} \right) \\ &= \ln \frac{t_N}{t_{N0}} + \zeta(X) - \left(\frac{1+e_0}{\lambda-\kappa} \varepsilon_v^p - \frac{\rho}{\lambda-\kappa} \right) = 0 \end{aligned} \quad (3)$$

where

$$\zeta(X) = \frac{1}{\beta} \left(\frac{X}{M^*} \right)^\beta \quad (\beta: \text{a material parameter}) \quad (4)$$

As shown in Fig. 4, t_{N1} and t_{N1e} measure the size of the subloading surface and the normal yield surface, respectively, t_{N0} is the value of t_N at the reference state, ε_v^p is related to the plastic volumetric strain and ρ is the state variable which represents the current soil density. The symbols λ and κ denote the compression index and the swelling index, respectively, and e_0 is the void ratio at the

reference state. Here, the yield surface (the subloading surface) not only expands, but also shrinks, so that the current stress point always lies on this surface.

The value of M^* in Eq. (5) is expressed as follows:

$$M^* = (X_{CS} + X_{CS}^{\beta-1} Y_{CS})^{1/\beta}$$

where,

$$X_{CS} = \frac{\sqrt{2}}{3} \left(\sqrt{R_{CS}} - \frac{1}{\sqrt{R_{CS}}} \right)$$

and

$$Y_{CS} = \frac{1 - \sqrt{R_{CS}}}{\sqrt{2} (\sqrt{R_{CS}} + 0.5)} \quad (5)$$

and these ratios, X_{CS} and Y_{CS} , are represented by the principal stress ratio at the critical state in triaxial compression R_{CS} . The plastic strain increment is split into two components, namely, component $d\varepsilon_{ij}^{p(AF)}$, obeying the associated flow in the modified stress t_{ij} space, and isotropic compression component $d\varepsilon_{ij}^{p(IC)}$ (Nakai and Matsuoka, 1986).

$$d\varepsilon_{ij}^p = d\varepsilon_{ij}^{p(AF)} + d\varepsilon_{ij}^{p(IC)} \quad (6)$$

where

$$d\varepsilon_{ij}^{p(AF)} = A \frac{\partial f}{\partial t_{ij}} = \frac{df_\sigma - \frac{1}{t_{N1}} \langle dt_N \rangle}{\frac{1+e_0}{\lambda-\kappa} \left(\frac{\partial f}{\partial t_{ij}} + \frac{G(\rho)}{t_N} \right)} \frac{\partial f}{\partial t_{ij}} \quad (7)$$

$$d\varepsilon_{ij}^{p(IC)} = \frac{a_{ii}}{a_{ii} + G(\rho)} \cdot \frac{\lambda - \kappa}{1 + e_0} \frac{1}{t_{N1}} \langle dt_N \rangle \frac{\delta_{ij}}{3} \quad (8)$$

$G(\rho)$ is a monotonically increasing function, which satisfies the condition $G(0)=0$ and is given in the form of

$$G(\rho) = a \cdot \rho^2 \quad (a: \text{material parameter}) \quad (9)$$

This model requires only a few unified material parameters, but can accurately describe the following typical characteristics of soil: (1) the influence of the intermediate principal stress on the deformation and the strength of soil is considered using the t_{ij} concept; (2) the influence of the stress path on the direction of the plastic flow is considered by splitting the plastic strain increment into two components; (3) the influence of the density and/or the confining pressure is considered by adopting the subloading surface concept by Hashiguchi (1980). On the other hand, the widely used Cam-clay model (both original and modified) overpredicts, for instance, the strength of the soil in the extension condition (parts of feature (1)) and the K_0 value (parts of feature (2)). The model cannot describe positive dilatancy during strain hardening, which is usually observed in overconsolidated clay and sand, nor can it explain the influence of the density and/or the confining pressure on the deformation and the strength of the soil (feature (3)).

The model parameters for the mass of aluminum rods are shown in Table 1. The parameters are fundamentally the same as those of the Cam-clay model (Roscoe and Burland, 1968), except for parameter a , which is respon-

Fig. 4. Shape of subloading surface, normal yield surface and definition of ρ

sible for the influence of the density and the confining pressure. Parameter β controls the shape of the yield surface. These parameters can easily be obtained from traditional laboratory tests. Figure 5 shows the results of biaxial tests for the mass of aluminum rods used in the model tests. The figure shows the positive and the negative dilatancy of the aluminum rod mass; it is clear that the strength and the deformation behavior are very similar to those of dense sand. By the way, the Cam-clay model cannot describe such positive dilatancy during strain hardening, as mentioned in the previous paragraph. The dotted lines represent the numerical results for a confining pressure of 1/100 times the confining pressure in the experiments. From the stress-strain behavior of the element tests simulated with the subloading t_{ij} model, it is noted that this model can express the dependency of stiffness, strength and dilatancy on the density as well as on the confining pressure.

Outline of Numerical Analyses

Figure 6 shows the mesh used in the finite element analyses for the tunnel excavation with the piled raft where the soil cover is $D/B=1.0$. Isoparametric 4-noded elements were used to represent the soil. The mesh was well refined with elements, 1 cm in width in most regions, which was even smaller around the tunnel excavation and below the foundations. The piles were modeled using hybrid elements (Zhang et al., 2000) consisting elastic

beam and solid elements. The frictional behavior (friction angle $\delta=18^\circ$) between the pile and the ground was simulated using elastoplastic joint elements (Nakai, 1985). The frictional angle, $\delta=18^\circ$, was obtained from laboratory model tests. Both vertical sides of the mesh were free in the vertical direction, and the bottom face was kept fixed. The analyses were carried out under plane strain conditions, since the aluminium rods did not deform in the out-of-plane direction. To simulate the tunnel excavation, horizontal and vertical displacements were applied to the nodes around the tunnel periphery. The analyses were carried out under the same conditions as those of the model tests. The initial stress levels of the ground were calculated by applying the body forces due to the self-weight ($\gamma=20.4 \text{ kN/m}^3$), starting from a negligible confining pressure ($p_0=9.8 \times 10^{-6} \text{ kPa}$) and an initial void ratio of $e=0.35$. After self-weight consolidation, the void ratio of the ground was 0.28 at the bottom and 0.30 at the top for $D/B=1.0$. The value of K_0 , derived from the simulation of the self-weight consolidation, was between 0.70 and 0.73, at the top it was 0.70 and at the bottom of the ground it was 0.73. In the case of an existing foundation, the ground is initially formed under geostatic conditions, and then a concentrated load is applied at the middle node of the foundation. The stress levels, void ratios and density parameters of the constitutive model at all integration points were stored and then used as the initial conditions of the ground before the tunnel

Table 1. Material parameters for aluminum rods

λ	0.008	Same parameters as Cam-clay model
κ	0.004	
N (e_{NC} at $p=98 \text{ kPa}$ and $q=0 \text{ kPa}$)	0.3	
$R_{CS}=(\sigma_1/\sigma_3)_{CS}(\text{comp.})$	1.8	
ν_e	0.2	Shape of yield surface (same as Cam-clay at $\beta=1$)
β	1.2	
a	1300	
		Influence of density and confining pressure

Fig. 6. Mesh for finite element analyses ($D/B=1.0$)

Fig. 5. Stress-strain-dilatancy relations for mass of aluminum rods

excavation.

RESULTS AND DISCUSSIONS

Greenfield Model

Surface Settlement

Figure 7(a) shows the observed surface settlement troughs for the tests with the fixed center and the fixed invert excavations for shrinkage $d_r = 4$ mm in the case of $D/B = 1.0$, 2.0 and 3.0. Figure 7(b) represents the computed results corresponding to the observed ones. The abscissa represents the distance from the center of the tunnel, while the vertical axis shows the amount of surface settlement. For both patterns of excavation, the maximum surface settlement occurred in the centerline above the tunnel crown. Surface settlements decreased with the

Fig. 7(a). Observed surface settlement profiles—greenfield

Fig. 7(b). Computed surface settlement profiles—greenfield

increase in tunnel depth, but they extended over a wider region. Similar results were observed in a previous research conducted with a trap door tunnel apparatus (Shahin et al., 2004a). The shape of the surface settlement profiles was the same for all the soil covers in the case of the fixed center excavation. For the same volume loss, the

Fig. 8(a). Observed surface settlements at centerline—greenfield

Fig. 8(b). Computed surface settlements at centerline—greenfield

maximum surface settlement was larger when the invert was fixed than when the center was fixed. Figures 8(a) and (b) illustrate the observed and the computed surface settlements above the centerline of the tunnel against the volume loss during the tunnel excavation, respectively. It can be clearly seen that even for a volume loss of less than 2%, the two different excavation patterns produced different amounts of settlement in the case of shallow tunneling. This is because the applied displacement at the crown was 8 mm for the fixed invert excavation and 4 mm for the fixed center excavation, although the volume loss was the same for both excavation patterns. Surface settlement occurred locally for an applied displacement of 8 mm in the case of the fixed invert excavation; consequently, the shape of the surface settlement profile varied with the soil cover in this case. The tendency of larger surface settlements for the fixed invert excavation was more significant up to $D/B=2.0$. In the case of $D/B=3.0$, however, the difference in surface settlement profiles between the two excavation patterns was less significant. It is revealed from these results that, for the same volume loss, surface settlement profiles vary with the excavation patterns in the case of shallow tunneling. Therefore, the surface settlement may not be properly estimated using the method of volume loss (Mair et al., 1993) for very shallow tunneling. The results of the numerical analyses show the same tendency as the model tests not only in shape, but also in quantity, namely, the maximum values are mainly at the center line. Some discrepancies may be observed in the computed values in the extreme boundaries of the model. This is because the lateral nodes were allowed to move freely in the vertical direction in the numerical analyses, when in fact there was some friction between the aluminium rods and the metal sides of the model. The agreement would have been improved if joint elements had also been used in this region.

Ground Movements and Shear Strain

Figures 9 and 10 represent the observed ground movements and the computed displacement vectors for soil cover $D/B=1.0$ in the case of the fixed center excavation and the fixed invert excavation, respectively. Two photos are superimposed in the figures; the first one was taken after the deposition of the initial ground and before the tunnel excavation, while the second one was taken after shrinking the tunnel by $d_r=4$ mm. The difference between the two photos is darkened using image processing software. The deformed area is bounded with white curves to show the range of the affected area in the ground more clearly in the figures. In the simulations, displacement vectors are used and their directions represent the direction of ground movements. The magnitude of the displacement vectors is proportional to the darkness of the color. It is seen in Fig. 9, for the fixed center excavation, that the deformation of the ground extended diagonally upward from the tunnel invert. It is seen in Fig. 10, however, for the fixed invert excavation, that the onset of the deformation of the ground was not just at the tunnel invert, unlike the case of the fixed tunnel

Fig. 9. Ground movements: center is fixed ($D/B=1.0$)

Fig. 10. Ground movements: invert is fixed ($D/B=1.0$)

Fig. 11. Distributions of shear strain: greenfield ($D/B=1.0$)

center. In this case, the range of the deformed region was narrower compared to the fixed center excavation. The numerical analyses perfectly captured the shapes of the observed deformation zones for different excavation patterns.

The distributions of shear strain in the model tests were obtained using the Particle Image Velocimetry (PIV) technique. PIV was originally developed in the field of fluid mechanics (Adrain, 1991). In this paper, two images were divided into a finite area; the average movement rate of the mass of aluminum rods for each area was extracted as nodal displacements. The strain for one grid was calculated from these displacements using the shape functions

and the B matrix that was used in finite element method to relate displacements and strains. Figure 11 shows the distributions of shear strain for the fixed center and the fixed invert excavations for $d_r = 4$ mm in soil cover $D/B = 1.0$. For the fixed center tunnel excavation, it is seen that the shear band of the ground developed from the tunnel invert, which covers the entire tunnel. For the fixed invert tunnel excavation, however, the shear band developed from the side of the tunnel. The shear band in this case was longer than that in the fixed center excavation. The different patterns of shear strain, due to the different types of the tunnel excavation, led to changes in ground behavior. The two different excavation patterns produced different kinds of shear strain, although the volume loss was the same. The shear strain of the numerical analyses showed a very good agreement with the results of the model tests.

Earth Pressure

Figure 12 shows the observed and the computed earth pressure distributions for $D/B = 1.0$. The plots are drawn in the twelve axes corresponding to the radial direction of the twelve load cells towards the center of the model tunnel. The figures represent the value of earth pressure in Pascal corresponding to the amount of applied displacement (amount of shrinkage). It is seen in the figure that the earth pressure decreased all around the tunnel for the fixed center excavation due to the stress relief. The results appear to be in agreement with the results of the tunnel experiments using the trap door apparatus performed by Murayama and Matsuoka (1971), Adachi et al. (1997) and Shahin et al. (2004a). As the shear band developed around the entire tunnel (Fig. 11), the surrounding ground became looser and reduced the stress levels in this area. It is also noticed that the earth pressure decreased suddenly after applying a little shrinkage of the tunnel with a magnitude between 0.05 and 0.20 mm. The sharp

Fig. 12. Distributions of earth pressure: greenfield

change in earth pressure, with only a very small shrinkage of the tunnel, confirmed what Peck (1969) described about soft ground tunnels. Shrinking the tunnel even further, the earth pressure gradually decreased at a lower rate and became almost constant after applying shrinkage to some extent. Sudden changes in earth pressure were due to the soil arching immediately after disturbing the ground. For the fixed invert excavation the earth pressure distributions were different from those of the fixed center excavation. In this case, the earth pressure decreased all around the tunnel until $d_r = 1$ mm. With further shrinkage of the tunnel, the earth pressure increased in the bottom part of the tunnel, while it remained almost unchanged in the upper part of the tunnel. This can be explained with the shear strain distribution shown in Fig. 11. From the above discussions, it can be said that the distribution of earth pressure is highly dependent on the excavation pattern.

Figures 13 and 14 illustrate the changes in earth pressure at load cells 4 and 9 against the amount of tunnel shrinkage for different soil covers. Load cell 4 is located closer to the tunnel crown, while load cell 9 is in the tun-

Fig. 13. Earth pressure history: center is fixed

Fig. 14. Earth pressure history: invert is fixed

nel invert. These figures confirm the sharp change in earth pressure during the tunnel excavation. In the case of the fixed invert excavation (Fig. 14), the earth pressure levels increased after around $d_t = 1$ mm at the position of load cell 9, which is different from the results of the fixed center excavation (Fig. 13) where the earth pressure remained almost constant after this amount of shrinkage. The phenomenon of the increase in earth pressure after the reduction to some extent can be described by the change in the arching effect due to the non-linear and elastoplastic behavior of the ground materials and to the ground arch which initially began close to the tunnel and then moved away. It cannot be described with an ordinary linear elastic model. The results of the numerical analyses are in good agreement with the results of the model tests. Therefore, it can be said that a proper elastoplastic constitutive model is required to predict the earth pressure around the tunnel for the lining design.

COMPARISONS WITH ELASTIC ANALYSES

Numerical analyses with a linear elastic theory have been carried out to compare the results with the elastoplastic analyses based on the subloading t_{ij} model described in the previous section. The comparisons here have been made by taking some typical results of the analyses. The Young's modulus for the elastic analyses has been chosen from the stress-strain relation (Fig. 15) of the biaxial tests performed in the laboratory on a mass of aluminum rods. The value of $E = 5500$ kPa is obtained from the figure, and the value of Poisson's ratio is assumed to be 0.33 for the ground of a mass of aluminum rods. Fig. 16 shows the surface settlement profiles of the model tests, the elastoplastic analysis and the elastic analysis for soil cover $D/B = 1.0$. It is seen in this figure that the elastic analysis produced a wider surface settlement profile compared to the observed one and that the maximum surface settlement was smaller as well. As there is no yield point in the linear elastic model, it cannot express the deformation that occurred locally. In this analysis, the displacement was applied to simulate the tunnel excavation; therefore, there was no influence of the magnitude of Young's modulus on the shape of the settlement trough except for the value of Poisson's ratio.

Figure 17 shows a comparison of the computed shear strain distributions between the elastic analysis and the elastoplastic analysis (subloading t_{ij} model). The results of the excavation pattern, where the center was kept fixed, are illustrated here. It is seen that the elastic analysis was not able to express the results of the model tests (Fig. 11) of the tunnel excavation. In the elastic analysis, shear strain was concentrated all around the tunnel. To investigate the effect of the boundary, two types of analyses were carried out. In the first type, the distance between the bottom boundary and the invert of the tunnel was 10 cm. In the second type, the tunnel invert was 20 cm from the bottom boundary. In the elastic analyses, it was observed that the distributions of shear strain depended on the distance from the tunnel invert to the bottom bound-

Fig. 15. Stress-strain relation of aluminum rod mass in biaxial tests

Fig. 16. Comparison of surface settlement profiles

Fig. 17. Comparison of shear strain distributions

ary. In contrast, both elastoplastic analyses produced the same shape of shear band and the same distribution of shear strain for the tunnel excavation, as observed in the model tests (Fig. 11). Therefore, it can be said that a proper elastoplastic constitutive model for soil is required to predict the ground deformation in a tunnel excavation.

GROUND WITH BUILDING LOADS

Surface Settlements

Figure 18 shows the observed and the computed surface settlement profiles for $D/B = 2.0$ in the case of a strip foundation at ground level for the fixed invert excava-

Fig. 18. Surface settlement profiles: strip foundation

Fig. 19. Surface settlement profiles: piled raft

tion. Figure 19 represents the same for the piled raft. For the sake of comparison, these figures also present the results of the greenfield condition for shrinkage $d_r = 4$ mm, which is rendered with the solid line. The position of the applied dead load is depicted at the top of the figures. It is seen in these figures that the maximum surface settlement occurred underneath the building load, and it was larger than that of the greenfield condition in the same way as was observed in tests with the trap door apparatus by Shahin et al. (2004a) and Sung et al. (2006). For the applied amount of building load, the strip foundation tilted in the opposite direction of the excavation. In contrast, the piled raft inclined towards the tunnel. The rotation of the foundations is illustrated in Fig. 20 for different soil covers. In this figure, the rotation for the fixed center excavation is included as well. The figure reveals that the direction of the rotation depended on the excavation pattern, the type of foundation and soil cover D/B . In the strip foundation, the maximum absolute rotation was observed when $D/B = 0.5$ for both excavation patterns. The maximum rotation in the piled raft occurred

Fig. 20. Rotation angles of foundations: towards tunnel is positive

when $D/B = 1.0$ for the fixed invert excavation, while it occurred when $D/B = 0.5$ for the fixed center excavation. The numerical simulations were able to explain the results of the model tests well for both foundations. From these results, it can be said that the surface settlement in real field tunneling may not be maximum just above the tunnel axis when superstructures exist nearby the tunnel. Under these circumstances, the proper prediction of the surface settlement is required to prevent excessive damage of nearby existing superstructures. It is also noted that surface settlement troughs for tunnel excavations in grounds disturbed by existing buildings do not follow the usual pattern of a Gaussian distribution curve, as is generally observed for the greenfield condition.

Ground Movements and Shear Strain

Figure 21 represents the observed ground movements and the computed displacement vectors for the strip foundation with a soil cover of $D/B = 1.0$. Figures 22 and 23 illustrate the distribution of shear strain when $D/B = 1.0$ and 2.0 for the strip foundation and the piled raft, respectively. It is demonstrated in the figures that the deformation zone spread towards the foundation from the side of the tunnel, and the disturbed zone for the tunnel excavation spread over the right edge of the strip foundation. The computed displacement vectors showed the same tendency of the ground movements as the model tests. For the induced initial stress in the ground due to the building load, the development of the shear band was unsymmetrical on the left and right sides of the tunnel. For $D/B = 1.0$, shear strain levels were concentrated in the ground to the left and to the right of the strip foundation, while for $D/B = 2.0$ they were mainly concentrated on the right side of the foundation. For the piled raft, a large shear strain due to the tunnel excavation was concentrated around the tip of the farthest (rear) pile in soil covers $D/B = 2.0$ and $D_p/B = 1.0$, while it was concentrated chiefly around the closest (front) pile in soil covers $D/B = 1.0$ and $D_p/B = 0.0$, where D_p is the vertical distance between the pile tip and the tunnel crown. Therefore, it can be said

Fig. 21. Ground movements: strip foundation

Fig. 22. Distributions of shear strain: strip foundation

Fig. 23. Distributions of shear strain: piled raft

that the distance from the tip of the pile to the tunnel crown had a significant effect on the deformation mechanism. The intensity of the shear strain on the left side of the foundations gradually decreased with the increase in soil cover. The computed distributions of shear strain for the numerical analyses showed a very good agreement with the results of the model tests. When ratio D/B was greater than 2.0, the shear band covered the whole region of the foundations, which restricted the differential settlement as well as the rotation of the foundations (Fig. 20). On the other hand, if the foundation fell partially in the large deformation region, the foundation would tilt either towards the tunnel or towards the opposite side of the tunnel, depending on the intensity of the shear strain levels around the foundation. The shape and the development of shear bands control the behavior of foundations in shallow tunneling, which emphasizes the necessity of applying a proper constitutive model for soil in predicting the mechanism of ground deformation when superstructures exist in the vicinity of a tunnel excavation.

Earth Pressure

Figure 24 shows the earth pressure distributions for the strip foundation and the piled raft when $D/B = 1.0$ in the fixed invert excavation. The plots are drawn using twelve axes corresponding to the radial direction of the twelve load cells towards the center of the model tunnel. Here, the dotted curves with black circular marks represent the earth pressure levels before applying the building loads, while the white circular marks represent the pressure levels after applying the building loads, and the solid line represents the earth pressure of the greenfield condition for $d_t = 4.0$ mm. The earth pressure at the foundation side increased after applying the building loads. In the case of the strip foundation, the earth pressure decreased to

Fig. 24. Distributions of earth pressure: $D/B = 1.0$

some extent around the tunnel immediately after performing the tunnel excavation. However, with the advances of the tunnel excavation, the earth pressure at the foundation side increased again in this case. In contrast, the phenomenon of the increase in earth pressure after its reduction to some extent was not observed in the piled raft. For both foundations, an unsymmetrical earth pressure distribution was seen around the shallow tunneling, and the final earth pressure distributions were different from those of the greenfield condition. Therefore, the effect of the soil-structure interaction should be properly contemplated in the earth pressure computation around the tunnel lining. The simulations slightly underestimated the measured earth pressure levels around the tunnel invert. As a whole, the subloading t_{ij} model simulated well the earth pressure distribution of the model tests in the case of the soil-structure interaction problem.

Behavior of Piles

Figure 25 shows the computed distributions of axial force at various stages of the tunnel excavation in the front and rear piles for $D/B=1.0$ and 2.0 . The vertical axis represents the length of the pile starting from the pile tip, and the compressive axial force is taken as positive in the figures. For soil cover $D/B=1.0$, it is revealed that the axial force decreased remarkably in the front pile, while it increased in the rear pile. For soil cover $D/B=2.0$, on the other hand, the axial force in both piles decreased due to the tunnel excavation. The phenomenon of the increase and decrease in axial force can be explained with Fig. 26, which illustrates the shaft friction and the load acting at the tip of the piles and the load applied by the raft. With the tunnel excavation, the shaft friction and the end bearing of the front pile decreased for the stress relaxation due to the tunneling which caused the reduction in axial force. When the soil cover was $D/B=1.0$, the axial force increased in the rear pile, due to the increase in the load

acting at the tip of the piles, since the rear pile was located outside of the large deformation region (Fig. 23). For the deeper ground, however, the induced axial force was less significant as both piles were situated within the area of

Fig. 26. Computed load distributions of piles

Fig. 25. Computed distributions of axial force

Fig. 27. Computed tunneling-induced bending moments

influence, which produced a smaller relative settlement between the ground and the piles. Therefore, the occurrence of induced compressive or tensile axial force depends on the relative position of the pile with respect to the tunnel.

Figure 27 illustrates the induced bending moments due to the tunnel excavation for soil covers $D/B=1.0$ and 2.0 . An induced bending moment developed for the differential settlement of the piled raft due to the tunnel excavation, and its magnitude increased with the volume of excavation. The position of the maximum bending moment varied depending on the location of the pile in the ground. When the soil cover was $D/B=1.0$, the maximum bending moment was observed at the pile head for the front pile, while it occurred in the middle of the pile for the rear pile. Similar to the axial force, the induced bending moment was less significant in the deeper ground as the piles were located in the large deformation region.

SIMULATION OF TUNNELING IN PROTOTYPE SCALE

Numerical simulations have been conducted in prototype scale for both the fixed center and the fixed invert tunnel excavations in the same way as the model scale tunneling. The diameter of the tunnel was 10 m and a soil cover of 10 m was used in the analyses. The same soil

parameters were used in the model scale analyses as were used in the prototype scale analyses. The stress level of the prototype ground corresponded to the solid curve of the stress-strain-dilatancy relations shown in Fig. 5. Figure 28 shows the surface settlement troughs for the fixed center and the fixed invert excavations for different volume losses in the greenfield condition. It is seen in the figures that, for the same volume loss, the maximum surface settlement and the shape of the settlement troughs varied with the excavation patterns in the same way as observed in the model scale shallow tunneling. For the same volume loss, the maximum surface settlement was larger when the invert was fixed than when the center was fixed. The fixed center excavation produced a wider settlement trough compared to the fixed invert excavation for the relatively larger volume loss in the shallow tunneling. Figure 29 represents the surface settlement troughs for the fixed invert tunnel excavation in the case of the strip foundation. It is seen in the figure that the maximum surface settlement occurred underneath the foundation similar to what was observed in the model scale tunneling. From these numerical analyses, the effect of the excavation patterns and the adjacent structures were observed not only in the model scale, but also in the prototype scale of the shallow tunneling. Hence, these effects should be considered for properly predicting the ground deformation during a shallow tunnel excavation.

CONCLUSIONS

To investigate the deformation mechanism and the earth pressure of the ground during a tunnel excavation, 2D model tests and elastoplastic finite element analyses have been carried out for two types of excavation patterns. The effects of the existing building loads with a strip foundation and a piled raft have also been investigated for both types of excavation problems. From the model tests and numerical analyses, the following points can be concluded:

Greenfield Excavation

- (1) The surface settlement and the earth pressure around a tunnel are significantly influenced by the displacement applied at the tunnel crown for the same volume loss and the same overburden.
- (2) The volume loss is less significant compared to the crown drift in predicting surface settlement troughs for shallow tunneling. In the case of a relatively deeper ground, different excavation patterns produce almost the same surface settlement troughs.
- (3) The fixed center excavation produces a wider disturbed region compared to the fixed invert excavation during shallow tunneling. The onset and the shape of shear bands depend on the excavation pattern.
- (4) The deformation mechanisms of the tunnel ground vary with the excavation pattern during shallow tunneling, and this controls the shape of the surface settlement trough and its magnitude.
- (5) The distribution of earth pressure is highly dependent on the excavation pattern, regardless of the depth of

Fig. 28. Surface settlement profiles: prototype scale

Fig. 29. Surface settlement profiles: strip foundation (prototype scale)

the soil covers. For the fixed center excavation, the earth pressure significantly decreases around the tunnel due to the arching effect. In contrast, the reduction in earth pressure is mainly observed at the tunnel crown during the fixed invert excavation.

- (6) The elastic analysis cannot represent the results of the model tests. The results depend on the distance of the boundary of the ground from the tunnel in the elastic analysis.

Ground with Building Loads

- (1) Existing building loads control the surface settlement and the zone of deformation during a tunnel excavation. The maximum surface settlement occurs underneath the existing structure.
- (2) The deformation zone of the ground moves towards the foundations due to the tunnel excavation. The ground deformation depends on the distance between the tunnel and the pile tip. If the closer pile is located in the large deformation region, shear strain is concentrated around it. In contrast, both piles located in the large deformation region produce lesser differential settlements and shear strain reaches the farthest pile.
- (3) For building loads, the unsymmetrical earth pressure distribution was observed during the shallow tunneling. The distribution of earth pressure varies with the type of foundation together with the excavation pattern, depending on the pattern of stress relief.
- (4) The relative position of the piles, with respect to the tunnel, plays an important role in the piled raft behavior in tunneling. During the tunnel excavation, the axial force of the piles changes due to stress relaxation and changes in the shaft friction depending on the deformation mechanism of the ground, i.e., whether the piles are located in the large deformation region or outside this area.

In the elastoplastic analyses, the results showed a good agreement with the results of the model tests in the tunnel excavation, including proper consideration of the existing building loads. The same constitutive model described well the 3D effects on the earth pressure and the displacements (Shahin et al., 2004b) and the influence of the existing foundation on the ground behavior (Sung et al., 2006) due to the tunnel excavation under 3D conditions. Therefore, the finite element analysis, in which the elastoplastic stress-strain behavior of the soil and the construction process are properly taken into consideration, is a powerful tool for the prediction of ground movements and earth pressure in tunneling.

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