BEHAVIOR OF A BRACED EXCAVATION IN SOFT CLAY AND THE UNDRAINED SHEAR STRENGTH FOR PASSIVE EARTH PRESSURE

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ABSTRACT

This report presents the behavior of a braced excavation in soft ground by analyzing data obtained from a construction site of the Tokyo International Airport. The excavation was 35 m wide and 11 m deep. These dimensions are not large in respect of present construction techniques in Japan. However, because the total thickness of soft clay layers at this site was over 50 m, the toe of sheet piles did not extend into a hard stratum, thus the braced-wall structure floated in the soft clay. The large lateral displacements of the sheet piles, as well as heaving of vertical supports, caused the excavation work at some sections to be stopped. At these sections, earth pressures mobilized on the excavated side were considerably smaller than those based on Rankine's passive earth pressure theory, using the unconfined compressive strength as the undrained shear strength. The paper examines effects of adhesion acting between the sheet pile and the ground, strength reduction due to swelling, strain rate and progressive failure, and differences in the confining conditions. As a result it turned out that the mobilized shear strength in the excavated ground is given by the undrained shear strength obtained from the triaxial extension test.

Key words: active earth pressure, case history, excavation, passive earth pressure, shear strength, sheet pile wall (IGC: H2/E5)

INTRODUCTION

The earth pressure acting on a retaining wall in a braced excavation is a very important parameter in calculating bending moments in the retaining wall and strut forces. Up to now, active earth pressure has been studied theoretically and experimentally using a lot of data accumulated from construction sites. Authorities and societies responsible for formulating construction standards have made use of these results to specify earth pressure distributions in their design codes. On the other hand, few studies have been done on the passive earth pressure acting on the excavated side, which is usually calculated from Rankine's theory. This is not because Rankine's theory can precisely evaluate the passive earth pressure mobilized in the excavated ground, but because few cases of the passive earth pressure in soft clays have caused a trouble. Although the scale of braced excavations has become greater and greater even in poor soil conditions, the retaining wall has usually been founded in a firm stratum, such as dense sand or gravel layers. In these cases, the earth pressure caused by excavation is supported in most part by the hard stratum. As a result the resistance of soft clay layers has not been fully mobilized.

However, with increasing utilization of coastal areas, ground conditions in many construction sites are different from those in previous decades. In these areas, where a thick soft clay layer is deposited, the retaining wall cannot be founded in a stiff layer, and braced-wall structures float in a soft clay layer. At the construction site of the Tokyo International Airport, the toe of the retaining wall of the braced excavation was founded in a soft clay layer, and the excavation work was stopped due to large lateral deformations in the retaining wall as well as very large heaving of the vertical supports. Analyses of data obtained from the excavation show that such abnormal behavior is likely to result from low passive earth pressure in the clay layer.

Until now, passive earth pressure has been discussed from two viewpoints; displacements required to fully mobilize the passive state and the undrained shear strength to evaluate the passive earth pressure. It is well known that wall displacements required to develop the passive state are considerably greater than those required to develop the active state (for example, see Lambe and Whitman, 1969). Consequently, some codes propose individual safety factors for the active or passive conditions, as introduced by Simpson (1992) in his Rankine lecture. Among studies on anisotropy of the undrained

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shear strength, Bjerrum et al. (1972) shed light on the strength for use in the excavation problems. They observed earth pressures on the passive side where a wall displacement was large enough to develop the passive state, and pointed out that the measured earth pressure agreed well with the calculated earth pressure using the vane shear strength corrected by the anisotropy factor, μ_a . However, the undrained shear strength is not only affected by anisotropy, but is also affected by many other factors, such as swelling due to excavation, adhesion between the ground and the retaining wall, confining conditions (plane or axial strains) and so on. Based on the analysis of data obtained at the braced excavation in which the excavation work was interrupted before the final stage, this report discusses the undrained shear strength mobilized in the excavated ground.

CONFIGURATION OF THE EXCAVATION WORK

Since 1985 the Ministry of Transport, Japan, has been undertaking a big project to transfer Tokyo International (Haneda) Airport to newly reclaimed land offshore of the present airport. Large scale excavations have been performed in this area to construct buried structures, such as vehicle roads and monorail and train tunnels. Two types of braced excavations have been conducted. Where excavation depths were more than 15 m, excavation sites enclosed by retaining walls have been improved by the Deep Mixing Method to increase passive earth pressure and heave resistance of the vertical supports. The behavior of these structures and their design method

were reported by Tanaka (1993 and 1994). However, soil improvement works were not applied to excavations less than 15 m deep. The braced excavation of Access Road, which is a case study in this report, was a non-improvement type. The location of this excavation is shown in Fig. 1.

Soil profile and soil properties at this site are shown in Figs 2 and 3, respectively. As the reclamation process was reported in detail by Tanaka et al. (1991), properties of each layer are briefly described herein. AC_1 layer, which was reclaimed by soils dredged at Tokyo Bay between 1971 and 1983, consists of clay with high water content and high compressibility. The values of unconfined compressive strength, q_u are less than 60 kPa. BS layer, which overlays the AC_1 layer, consists of material transferred from excavation sites in the Tokyo Metropolitan area, such as sand, gravel and sometimes fragments of concrete. AS layer is a naturally deposited sand layer. Its N values measured by the standard penetration test are in the range of 5 to 20. AC_2 layer is a normally consolidated marine clay that was deposited after the glacial age. The values of q_u in the AC_2 layer are about 100 kPa at the top and gradually increase with depth. The hard stratum whose N value is over 50 can be found at the elevation of -60 m AP (Arakawa Pale).

Because the excavation was over 600 m long, the excavation work was divided into 7 sections as indicated in Fig. 2. Sections from 1 to 4 were firstly excavated, and were heavily instrumented, as listed in Table 1 to observe the behavior for the safe construction and to confirm the design parameters for the other sections. Only

Fig. 1. Plan view of the Tokyo International Airport and location of the braced excavation

Fig. 2. Soil profile of the site and the construction sections

Fig. 3. Soil properties of the site

displacements of the walls and heave of the vertical supports were measured in the other sections.

The excavation profile in section 3 is given in Fig. 4 as a typical example. This excavation is 35 m wide and 11 m deep. U section steel sheet piles were installed as a retaining wall to a depth of 24.25 m, i.e., 2.5 times the excavation depth. The moment of inertia, I , per meter of the sheet pile is $86,000 \text{ cm}^4/\text{m}$, assuming that no shear sliding could not occur at the interlocks located on the neutral axis of the section. However, the strain distribution measured by strain gauges attached to the sheet pile

implies that I may be reduced to $1/4$ of the above figure due to the shear sliding at the interlocks (Tanaka and Suzuki, 1988). Even with such long sheet piles, the toe of the wall did not reach a firm stratum, but the structure floated in a soft clay layer. Four struts comprising double H piles (the first strut was single) were placed. Struts at each level were installed after the excavation of their stage was completed. The excavation depth at each stage was about 1 m deeper than the center of the strut. Preload was applied to each strut to minimize the lateral displacement and to reduce the bending moment of the re-

Table 1. Measurement items of the braced excavation

Item of measurement	Number of measurements /section	Method of measurement
earth pressure	36 points	diaphragm type cell pressure
pore water pressure	18 points	diaphragm type cell pressure with porous metal
stress on sheet piles	48 points	strain guage
strut force	28 points	strain guage
displacement of support	6 points	level
displacement of sheet pile	2 lines	inclinometer

taining wall as indicated in Table 2. These values were nearly a half of the design strut load.

The excavation work at sections 1 to 4 was carried out between March and October, 1986. At sections 1 to 3, the lateral deformation of the sheet piles became large after the 3rd stage. In addition, considerable heaving of the vertical supports was observed. As it was judged that the structure would fail if the excavation work continued, the work was stopped at the 4th stage when the excavation depth reached 9.3 m; a depth of only 1.7 m was left unexcavated.

Figure 5 shows the displacements of the sheet pile at each excavation stage at section 3, where the largest displacement was observed. The shaded areas in the displacement curves indicate the movement while the excavation work was interrupted to install struts. In the figure the horizontal lines show the excavation depths at each stage. As can be seen, the maximum displacement at each stage was yielded a few meters below the excavation

Table 2. Preload on struts

level of strut	1	2	3	4
preload (kN/m)	80	160	240	360

Fig. 5. Displacements yielded in the sheet piles of section 3

bottom. This pattern of the retaining wall displacement is generally observed in soft soil conditions. Fig. 6 shows the maximum displacement of the sheet pile at the same

Fig. 4. Typical cross section of the braced excavation (section 3)

Fig. 6. Maximum displacement of the sheet piles at section 3

section plotted against elapsed time. After the 3rd stage was completed, the magnitude and pattern of sheet pile displacements became unusual and it was decided to carefully monitor the behavior of the structure. In the 4th stage, the sheet piles moved toward the excavated side as much as 100 mm during installation of the struts, and the total displacement reached over 300 mm. Although long sheet piles were installed, large displacements were measured at their toes. Heaving of the vertical supports, as shown in Fig. 7, also caused great anxiety about the stability of the structure. Like the lateral movements of the sheet piles, heaving of the vertical supports became large after the 3rd stage, and at the 4th stage the amount of heave reached as much as 160 mm at the center of the excavation. Thus, it was decided to stop the excavation work and adopt countermeasures. Deepwells were installed at the back of the retaining wall, and water was pumped out to lower the water table. In addition, the ground level at the back was lowered by 1.7 m. This depth was the same as the thickness needed to

reach the final excavation depth from the 4th stage. As a result, lateral movement of the sheet piles ceased as shown in Fig. 6, and the final excavation could be completed. For sections 4 to 8, the maximum deformation of the sheet piles was less than 13 cm, and heave of the vertical supports was so small that the excavation work could proceed smoothly.

EXAMINATION BY THE PRESENT DESIGN METHOD

In Japan, design methods for braced excavations differ slightly depending on the authority and the society. However, procedures in these methods may be divided into three main parts (Tanaka, 1993); 1) equilibrium of earth pressure, 2) examination of base failure, and 3) calculation of strut forces and bending moments in the retaining wall.

Equilibrium of Earth Pressure

In this procedure, the retaining wall is examined to be long enough to sustain the active earth pressure. The examination is conducted by moment balance at the level of the lowest strut. The active and passive earth pressures are generally calculated by Rankine's theory, which neglects friction between the wall and the ground. The safety factor, F_e can be defined by the following equation;

$$F_e = l_p P_p / l_a P_a \quad (1)$$

where P_a and P_p are the resultant forces of the active and passive earth pressures acting on the wall between the depth of the lowest strut to the toe of the wall, respec-

Fig. 7. Heaving of the vertical support at section 1

tively. l_a and l_p are the distances from the depth of the lowest strut to the positions of P_a and P_p , respectively.

Base Failure

This procedure is sometimes called “the stability against heave” examination. Among many methods for examining base failure, Bjerrum and Eide’s method (1956) and the AIJ method, which has been recommended by the Architectural Institute of Japan (1988), are mainly used in practical design in Japan. In the latter method, the circular failure with its center at the lowest strut is assumed.

Bending Moments and Strut Forces

In Japan, there are two methods for calculating strut forces and bending moments in retaining walls. One is to use the earth pressure distribution obtained from a lot of data on strut forces observed in past construction sites. This distribution is called “an apparent distribution”, because it is not directly measured by earth pressure cells attached to the wall, but calculated from strut force data. Until now, a lot of apparent earth pressure distributions, such as Terzaghi and Peck’s (1948) and Tschebotarioff’s (1973) are proposed, and some of them have been incorporated in the design codes by authorities. In Japan, the use of the apparent distribution method is restricted to small-scale excavations. For large-scale excavations in soft soil, the so-called “ground spring method”, proposed by Yamagata et al. (1969) and modified by Nakamura and Nakazawa (1972), is usually used to calculate the bending moments and strut forces. In this method, the ground on the passive side is assumed to be a series of springs that behave elastically when their reaction is smaller than the passive earth pressure, and do perfectly plastically when the reaction exceeds the passive pressure.

The braced excavation introduced herein was designed according to the above design procedures. The calculation results are listed in Table 3. Sections 1 to 3 were calculated at the 4th stage when the excavation work was stopped, and the other sections were calculated at the final stage. It is usual in practical design for a surcharge to be applied on the back to take account of the weights of excavation machinery, and partly because of safety. However, such a surcharge is not considered in the

results listed in Table 3. F_{bb} and F_{bj} are safety factors against base failure according to Bjerrum and Eide’s method and the AIJ method, respectively. N_b is a stability number proposed by Peck (1969). As shown in the table, the calculated results for each section are not likely to suggest that the excavation is unstable except F_e at sections 4 to 7, and no marked differences can be identified at these sections.

The ground spring method was used for determining the size of the sheet piles and struts. The bending moments and strut forces yielded during the excavation were within the allowable stress of each member. It should be noted that the reason for stopping excavation was not excessive bending stresses in the retaining wall, nor excessive axial stresses in the struts.

EARTH PRESSURE DISTRIBUTION

Earth pressures acting on the retaining wall were measured by diaphragm type pressure cells. Figure 8 gives an example of the earth pressure distribution measured in section 2. Before excavation started, the earth pressures on both the excavated and the back sides increased with depth. The coefficient of earth pressure defined by the total stress, $K (=p/\gamma_t z)$, where p is measured earth pressure including water pressure, γ_t is unit weight of soil and z is depth), was around 1.0. As the excavation proceeded, the earth pressure distributions of the earth pressures changed. On the back side, since the stress condition was transferred from at-rest to the active state, the reduction of earth pressure could be observed in the AC_2 layer. At the 4th stage, the earth pressure decreased to the active pressure calculated by Rankine’s theory, where the undrained shear strength is taken as half of q_u . On the other hand, the earth pressure on the upper part became greater than before excavation, because the movements of retaining wall were restricted by the struts.

The change in earth pressures on the excavated side is somewhat more complicated than that on the back side, because the overburden pressure reduces as excavation proceeds. That is, as shown in Fig. 8, the earth pressure itself decreases with increasing excavation depth. The passive earth pressure, p_p is generally evaluated by Rankine’s theory and in cases of cohesive soil p_p is given by Eq. (2).

$$p_p = \gamma_t z + 2s_u \quad (2)$$

where s_u is the undrained shear strength, and $\gamma_t z$ is the overburden pressure calculated from the excavated bottom.

To demonstrate how the shear strength was mobilized on the excavated side at the 4th stage, the earth pressures reduced by $\gamma_t z$, $(p - \gamma_t z)$ are plotted against depth in Fig. 9, where the solid thick line and σ indicate the mean and the standard deviation of pressures measured at 6 locations of sections 1 to 3, respectively. In the same figure, the dotted line indicates the values of $(p - \gamma_t z)$ observed at section 4 where the excavation work was completed without stopping. If the earth pressures on the excavated

Table 3. Calculation results according to the present design method

Item of examination	Section				
	1, 2, 3	4	5	6	7
stability number, N_b	3.36	3.39	3.61	3.17	3.00
equilibrium of earth pressure, F_e	1.51	1.05	1.02	1.12	1.14
base failure by Bjerrum and Eide, F_{bb}	1.64	1.62	1.52	1.74	1.67
base failure by AIJ, F_{bj}	1.62	1.27	1.20	1.23	1.32
interruption	○	×	×	×	×

Fig. 8. Observed earth pressure distribution on the sheet piles at section 3

Fig. 9. Observed earth pressures on the excavated side when excavation work was stopped

side had been fully mobilized to the passive state, which can be expressed by Rankine's theory, the values of $(p - \gamma_t z)$ must coincide with the values of $2s_u$ as indicated by the $2s_u$ line in Fig. 9. As already mentioned, the undrained shear strength is evaluated by $q_u/2$, and $2s_u$ in the

figure is equivalent to q_u . However, the observed values of $(p - \gamma_t z)$ were significantly smaller than those of $2s_u$. This means that the earth pressure on the excavated side could not be mobilized to the passive earth pressure expected from q_u even though the retaining wall might be deformed enough to develop the passive state.

At section 4, the values of $(p - \gamma_t z)$ in the AC_2 layer are far smaller than those at sections 1 to 3. This is because the mobilized passive earth pressure in the sand layer of the AS was so large that the AC_2 layer still did not fully develop its resistance force. By carefully examining the soil profile in Fig. 2, it will be noted that at sections 1 to 3 the AS layer was removed at the 4th stage when the unusual behavior was observed. Conversely, at sections 4 to 8, where the lateral displacements of the sheet piles were less than 130 mm, the AS layer remained below the excavated bottom at the final stage. The existence of the sand layer at the excavated bottom played an important role in whether or not the excavation work could continue to the final depth.

The sand layer also greatly influenced on the earth pressure distribution on the back side, as given in Table 4. In the table, P_a is the resultant earth pressure acting along the full length of the retaining wall on the back side at the 4th stage. $P_{a,u}$ is the same as P_a but on the part above the excavation bottom. P_p is the resultant earth pressure on the excavated side, and ΣS is the sum of the strut forces. In Table 4, it can be seen that in sections 1 to

Table 4. Resultant earth pressures and strut forces

Section	1	2	3	4
P_a (full length on back side) kN/m	2830	3040	2920	3150
$P_{a,u}$ (upper part from the exca. level on back side) kN/m	810	840	640	820
P_p (on excavated side) kN/m	2410	2320	2360	2620
ΣS (sum of strut forces) kN/m	810	790	650	620
$\Sigma S/P_{a,u}$	1.00	0.94	1.01	0.75
$(P_p + \Sigma S)/P_a$	1.14	1.02	1.03	1.03

3, earth pressure on the back side above the excavation bottom was supported by the struts because the ratios of $\Sigma S/P_{a,u}$ are nearly unity. On the other hand, in section 4, where the displacement of the retaining wall was relatively small, the ratio of $\Sigma S/P_{a,u}$ is 0.75, which is much smaller than those in sections 1 to 3. In sections 1 to 3, where the passive earth pressures on the excavated side could not be mobilized enough to sustain the pressures on the back side, the earth pressure on the back side was redistributed. The earth pressure in this case study could be precisely measured because the ratios of $(P_p + \Sigma S)/P_a$ are about unity in every section, which means that the earth pressures on both the back and the excavated side are well balanced.

Figure 10 shows how the earth pressure on the excavated side changed during excavation. As indicated in Fig. 5, the lateral displacements of the retaining wall were not uniform but varied with depths. Therefore, the average values between depths of 12 m and 25 m are plotted on the horizontal axis. The average of the observed earth pressures $(p - \gamma_t z)$ measured at the same depth are plotted on the vertical axis of the figure. The values of $(p - \gamma_t z)$ were negative before excavation started because the vertical overburden pressure was larger than the measured horizontal pressure. However, with the change

Fig. 10. Mobilized earth pressure on the excavated side during excavation

Fig. 11. Undrained shear strength from triaxial extension test

in the direction of the major principal stress during excavation, the values of $(p - \gamma_t z)$ became positive from negative. When the horizontal displacement of the wall exceeded 100 mm, the value of $(p - \gamma_t z)$ reached a constant value of 75 kPa, which is much smaller than expected from the undrained shear strength in the bottom layer.

It is well known that the undrained shear strength is not an inherent value, but is affected by many factors, in particular, anisotropy. It has been suggested that the shear strength used in calculating the passive earth pressure should be obtained by the triaxial extension test (Bjerrum et al., 1972). For the undisturbed samples obtained from the AC_2 layer at this site, the triaxial extension test was conducted. After consolidation under K_0 conditions with the in situ effective overburden stress to the vertical direction, the specimens were sheared under undrained conditions at an axial strain rate of 0.1%/min. As shown in Fig. 11, the values of the undrained shear strength from the extension test, s_{ue} are 0.75 times those of $q_u/2$. If s_{ue} is used instead of $q_u/2$ for calculating the passive earth pressure, the observed earth pressures apparently coincide with the calculated values, as indicated in Fig. 9. However, the above conclusion ignores other important factors, such as shearing rate, swelling, and so on. These factors are carefully examined in the following chapter.

VARIOUS FACTORS AFFECTING PASSIVE EARTH PRESSURE

Influence of Adhesion between the Wall and the Ground, m_1

In most of design codes in Japan, passive earth pressure is calculated by Rankine's theory, which ignores friction between the retaining wall and the ground. If this

friction is taken into consideration, the earth pressure for cohesive soil may be given by Eq. (3) (Lee, I.E., 1968).

$$p_p = \gamma_t z + 2s_u \sqrt{(1 + c_{uw}/s_u)} \quad (3)$$

where c_{uw} is adhesion between the wall and the ground. Although c_{uw} may be affected by the material of the wall, the displacement pattern and so on, it is assumed to be 0.75 times s_u of surrounding soil according to the design method for platforms of the American Petroleum Institute (Poulos, 1988). If this value is substituted into Eq. (3), p_p becomes 1.32 times the value obtained from Eq. (2), which ignores adhesion.

Influence of Swelling, m_2

In practical design, the total stress analysis is usually applied to cohesive ground, because the permeability of the ground can be considered to be small in respect of the construction period. This analysis is conservative in cases where the total stress increases during construction, such as embankments, because the increase in the shear strength gained by consolidation is neglected. However, in excavations where the overburden pressure decreases, the total analysis gives an unsafe answer, because the undrained shear strength is reduced by swelling. Figure 12 shows the strength reduction obtained by the triaxial test where specimens rebounded under K_0 conditions after a normally consolidation state. Using this test result, the strength condition in the bottom ground at the 4th stage is predicted as shown in Fig. 13, assuming that the ground is perfectly swelled. It can be seen that about 20% of the strength reduction occurs in the ground.

Although there have been few reports on the coefficient of consolidation in the swelling process, c_{vr} , Nakase et al. (1973) presented that c_{vr} is 10 or 30 times greater than in the normally consolidated state. The value of c_{vr} is assumed to be 2,000 cm²/day since c_v at normally consolidated state is about 100 cm²/day at this layer. In addition, the drainage length, H is assumed to be 15 m, which is equal to the distance between the surface of the excavated bottom and the toe of the retaining wall. The period of each excavating stage, t was about two weeks. From these

Fig. 12. Strength reduction due to swelling

Fig. 13. Predicted strength reduction in the excavated bottom after swelling is completed

assumptions, time factor, T_v may become 0.012 using Eq. (4).

$$T_v = c_v t / H^2 \quad (4)$$

From the theoretical curve between the degree of consolidation, U and T_v for one dimensional consolidation, U may be 12% at $T_v=0.012$. From the isochrone curve, only 20% of the consolidation occurs near the ground surface at $U=12\%$. Therefore, it can be said that strength reduction due to swelling did not actually occur in the excavated ground. However, the pattern of pore water pressures measured on this site contradicts the above conclusion. Figure 14 shows pore pressure distributions measured at section 3 before excavation at the 4th stage. It can be seen that the pore water pressures are greater than the hydraulic pressure given with datum of the surface of the excavated ground; $p_w = \gamma_w z'$, where p_w is the hydraulic pressure, γ_w is the unit weight of water and z' is the distance from the excavated surface. If the ground is completely swelled and seepage from the back side is ignored, the pore water pressures in the ground must coincide with the hydraulic pressure line of $p_w = \gamma_w z'$. However, as seen in Fig. 14, the measured pore water pressure exceeded the hydraulic pressure. This means that the strength in the bottom reduced more intensively than the strength reduction line suggested by Fig. 13.

The excess pore water pressure, Δu is generated by the shear stress increment, Δq , as well as by the mean stress increment, Δp_m and can be expressed by Eq. (5).

$$\Delta u = \Delta p_m + a \Delta q \quad (5)$$

Fig. 14. Distribution of pore water pressures measured at section 3

where a is a coefficient corresponding to Skempton's A coefficient. Figure 15 shows the relationships between Δq and $(\Delta u - \Delta p_m)$ obtained from triaxial extension tests, where Δq is defined as $\Delta(\sigma_1 - \sigma_3)$. The gradient of the relationships corresponds to the coefficient a of Eq. (5). The values of a in this layer are in the range of 0.25 and 0.5 although they tend to increase with Δq . Figure 16 shows the relationship observed in the excavation, assuming that directions of the principal stresses are in the vertical and horizontal directions and that the stress in the direction of the plane strain holds constant. Then, Δq and Δp_m are given by the following equations;

$$\Delta q = \Delta \gamma_t z - \Delta p_{ob} \quad (6)$$

$$\Delta p_m = (\Delta \gamma_t z + \Delta p_{ob}) / 2 \quad (7)$$

$$\Delta u = u_{init} - u \quad (8)$$

where $\Delta \gamma_t z$ and Δp_{ob} are increments in the overburden pressure and the observed lateral earth pressure on the excavated side, respectively, with the values before excavation as datum. u_{init} and u are the measured pore water pressures before excavation and at each stage, respectively. If these values are plotted on the axes of Fig. 16, the coefficient of a becomes about 0.5. Figs. 15 and 16 cannot be directly compared because there are uncertain-

Fig. 15. The relationships between Δq and $(\Delta u - \Delta p_m)$ obtained from triaxial extension test

Fig. 16. The relationships between Δq and $(\Delta u - \Delta p_m)$ obtained from the braced excavation

ties in calculating the relationships in cases of the excavation. In addition, we have to consider the difference in the confining conditions between the triaxial and plane strain states. However, it can be understood that the relatively high pore water pressure generated in the excavated ground is likely to be caused by the increasing shear stress, not by swelling nor by seepage from the back side.

Stress and Strain Relation, m_3

As many researchers have pointed out, the active state is reached with only small a wall displacement, but large displacements are required to develop the passive state. This is suggested by the stress-strain curves obtained from the compression and extension triaxial tests, as indicated in Fig. 17. The compression test, which corresponds to the active state, gives a stress-strain curve with a clear peak at a small strain. However, in the extension test a very large strain is needed to reach the maximum strength.

To estimate strains generated in the bottom ground, it is assumed that the displacement of the retaining wall monotonously decreases with decreasing depth and there is no displacement at the toe. Then, the shear strain, γ can be calculated by Eq. (9).

$$\gamma = \lambda / H \quad (9)$$

where λ is the displacement of the wall at the excavated ground surface, and H is the length of the buried retaining wall between the toe and the excavated surface. If λ is 30 cm and H is 15 m, then γ is equal to 2%. The axial strain in the triaxial tests, ϵ_a becomes equal to γ under undrained conditions. From these considerations, the mobilized strength in the excavated ground is about 60 to 70% of the maximum strength as shown in Fig. 17, when the excavation work was stopped.

Strain Rate, m_4

It is well known that the strain rate greatly influences

Fig. 17. Stress-strain curves obtained from compression and extension triaxial tests

the shear strength. Tsuchida (1990) reported that the undrained shear strength of Japanese marine clays is reduced by 6% when the strain rate decreases to 1/10, as shown in Fig. 18. Referring to the fact that the rate of wall displacement at the 4th stage is about 20 mm/day, and making the same assumption as for the strain calculation, the strain rate in the excavated ground can be estimated as 0.001%/min. Since the extension triaxial test was conducted at the strain rate of 0.1%/min as previously mentioned, the in situ undrained shear strength may be 0.88 times that obtained from the triaxial test.

Confining Condition, m_5

The stress state of the ground of the braced excavation is a plane strain condition where the strain is confined in the longitudinal direction. Ladd et al. (1977) reported that the average strength ratio of triaxial and plane strain extension tests for four clays is 0.82. That is, the undrained shear strength under plane strain conditions is 1.22 times that obtained from the triaxial extension test.

Relationship between the Undrained Shear Strength Obtained from the Triaxial Extension Test and the Passive Earth Pressure

By integrating all the factors discussed in the previous sections, Eq. (10) can be derived.

$$\begin{aligned} m &= m_1 \times m_2 \times m_3 \times m_4 \times m_5 \\ &= 1.32 \times 1.0 \times (0.6 \sim 0.7) \times 0.88 \times 1.22 \\ &= 0.85 \sim 0.99 \end{aligned} \quad (10)$$

The correction factor for the passive earth pressure to the

Note IC; Isotropic Consolidation
K₀C; K₀-Consolidation

Fig. 18. Strain rate effect on undrained shear strength (after Tsuchida, 1990)

undrained shear strength from the triaxial extension test, m is in the range of 0.85 to 0.99, which is very close to unity. Considering the experimental fact that the stress-strain curve in plane strain is much steeper than that in the triaxial test as reported by Vaid and Campanella (1974), the value of m , may somewhat underestimate the real values, and the value of m may be much closer to unity. This result agrees with the observed fact that the measured earth pressures on the excavated side coincided with the passive earth pressures calculated from Rankine's theory, provided that the undrained shear strength from the triaxial extension tests are used.

CONCLUSIONS

The behavior of braced excavations in soft ground has been presented using construction data from the Tokyo International Airport. In some sections the excavation work was stopped due to large retaining wall displacements and heaving of the vertical supports. From analyses of the observed data and results of laboratory tests, the following conclusions are drawn;

(1) At all sections where the excavation was stopped, the sand layer was removed before the final stage. At these sections the retaining wall displacements and heaving of the vertical supports suddenly became large immediately after the sand layer was excavated.

(2) Even though large displacements were observed, the measured earth pressures on the excavated side were considerably smaller than those calculated by Rankine's passive earth pressure theory if the undrained shear strength was evaluated by the unconfined compressive strength, q_u .

(3) The earth pressure distribution on the back side is strongly affected by the ground conditions below the excavated bottom. At sections where the sand layer was removed, redistribution of earth pressure on the back side was clearly observed.

(4) If the strength from the extension triaxial test was used instead of q_u , the measured earth pressure on the excavated ground agrees well with the calculated passive pressure.

(5) Factors influencing on the passive earth pressure were carefully examined; i.e., adhesion between the retaining wall and the ground, swelling, strain rate, progressive failure, and confining conditions (triaxial or plane strain). By considering all these effects, it was found that the correction factor for passive earth pressure to the undrained shear strength from the triaxial extension test is very close to unity.

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