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GROUND DEFORMATION DURING A BRACED EXCAVATION IN A THICK SOFT CLAY DEPOSIT

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This paper studies the problem of large deformation and plastic flow in the ground during a braced excavation. As a part of the Tokyo International (Haneda) Airport Extension Project (Phase I), a large scale braced excavation (35 m wide, 11 m depth, and 560 m length) was carried out. In the Construction Unit 1 to 3, large lateral deformations of the retaining walls and significant heave of the supporting piles were observed after the 3rd and 4th excavations. Consequently, the 4th excavation was halted and remedial measures were taken. This case history is analyzed by using an elasto-plastic 2D finite element method incorporating non-linear joint elements for soil/retaining wall interfaces. A formula to evaluate the shear strength parameters for the soft clays is proposed. With a reasonable reduction in shear strength of soils, the results of analysis proves quite consistent with those observed in the field.

KEY WORDS: Excavation, Retaining wall, Shear strength, Finite element method

1. Introduction

With the development of urban areas, cases are increasing in recent years where infrastructures such as roads and waste-water treating facilities are constructed underground. It is also usual that large scale basements are constructed for high-rise buildings to effectively utilize very expensive land. To construct these underground structures, large scale excavations are often conducted employing retaining walls and bracing.

In the large scale braced excavations, it is common that the observational construction method is adopted with cost-effective bracing systems for the following reasons:

(1) Increase in durability and accuracy of the instrumentation, and development of efficient observation systems;

(2) Development of back and prediction analysis methods based on the beam theory on the Winkler spring model (Nakamura and Nakazawa, 1972); and

(3) Availability of many prevention and remedial measures (such as dewatering and stiffening bracing) in case for unexpected ground deformations.

Consequently, with plentiful measured data and controlled construction sequences, the large scale braced excavation is becoming one of the most active research topics in the geotechnical engineering. A survey of the literature published in the past 10 years revealed that:

(1) In most cases, retaining walls are embedded into firm soil deposits, and the maximum measured lateral displacements of the walls are less than 100 mm; and

(2) Attention has been paid mainly on the lateral displacements and the bending moments of the retaining wall structures, and on the settlements of the ground surface behind them.

In recent years, however, there have been cases where it was impossible to embed retaining walls into a firm soil stratum since the site was covered with very thick soft clays (Sugimoto 1989; Tanaka et al. 1989). Measured in these cases were not only the deformations and bending moments of retaining walls but also the heave (upward movement) of the supporting piles installed to prevent the bracing from buckling (see Fig. 3). In some cases, the construction work was at halt while appropriate remedial measures were applied to cope with the danger of buckling in horizontal bracing.

In analyzing this problem, it is important to deal with not only the deformation of the retaining walls, but also the deformation and stability of the bottom of the excavated ground. It is therefore apparent that the conventional beam on the Winkler spring model, which deals with only the behaviour of the retaining wall, is not suitable for analyzing the problem. To analyze braced excavation problems in a more realistic manner, finite element methods have been employed by many investigators (Clough and Duncan, 1971; Osaimi and Clough, 1979; Simpson et al., 1979; Honda, 1986). When applying a finite element method, it is important to employ a numerically stable solution method, since numerical difficulties may arise with the formation of large plastic zones in the soil mass (Okamoto and Hayashi, 1991).

In view of the foregoing discussions, this paper studies the applicability and limitations of a finite element method with the pseudo-viscoplastic algorithm (Zienkiewicz and Cormeau, 1974) for analyzing a well documented case history (Tanaka et al., 1989). Choice of the shear strength parameters for the soft clays is discussed based on the comparison between the observed and calculated results.

2. Method of Analyses

To simulate the "plastic flow" anticipated in the soil mass and soil/retaining wall interfaces, a 2D-finite element program NAPG-2D was used (NKK Corporation, 1988). The analyses used a non-linear viscoplastic algorithm (Zienkiewicz and Cormeau, 1974) together with non-linear plane strain elements for the soil, beam elements for the retaining walls (sheet piles) and bracing, and 6-noded non-linear joint elements (Sekiguchi et al., 1990) for the soil/retaining wall interfaces. An optimization method of the pseudo-viscoplastic parameters is also used to accelerate the convergence in calculation (Sekiguchi et al., 1992).

3. A Braced Excavation in Soft Clay

As a part of the Tokyo International (Haneda) Airport Extension Project (Phase I), a large scale braced excavation (35 m width, 11 m depth, and 560 m length) was carried out to construct an access road (Tanaka et al., 1989).

3.1 Soil Conditions

Soil conditions at the construction site are shown in Fig. 1 where the following notations are applicable: BS=stratum composed of reclaimed construction waste soils; AC_1=stratum of reclaimed waste water slag; AS_0=alluvial sand stratum; AC_2=alluvial soft clay stratum; DC=diluvial soft clay stratum; and DS=diluvial firm sand stratum. Engineering properties of these strata are listed in Table 1. The coefficient of earth pressure at rest, Ko, was calculated from Fig.24 of Tanaka et al.(1989). The Poisson's ratio, ν, was estimated assuming undrained condition for clay, and drained condition for sand.

As shown in Fig. 2, the project was divided into 9 construction sections, length of each unit being 60 m. As inferred from the figure, the alluvial sand (AS) stratum was removed during the 4th excavations in the Construction Units 1 to 3.

Table 1 Material Properties of Soils

Soil Type	SPT-N	γ_t (kN/m³)	ν	$q_u/2$ (kPa)	ϕ' (degree)	K_0
BS	2–10	18	0.375	0.0	30.0	0.50
AC_1	–	16	0.475	2.0	0.0	0.90
AC_1	–	16	0.475	5.0	0.0	0.90
AS	5–20	19	0.375	0.0	40.0	0.75
AC_2	0– 4	16	0.475	*	0.0	0.90
DC	2– 4	16	0.475	*	0.0	0.90
DS	> 50	20	0.375	0.0	45.0	0.80

* see Fig. 1

Table 2 Material Properties of Structural Elements

Structural element	Nominal name	Axial Stiffness EA	Bending Stiffness EI
Sheet Pile	V_L Type	56.2	1.32*
1st Brace	1-H350x350	7.30**	0.00
2nd Brace	2-H350x350	14.60**	0.00
3rd Brace	2-H400x400	18.36**	0.00
4th Brace	2-H400x400	18.36**	0.00

Unit: EA ($\times 10^5$ kN/m) ; EI ($\times 10^5$ kNm²/m)
* η=0.4 is to be multiplied.
** α=0.1 is to be multiplied.

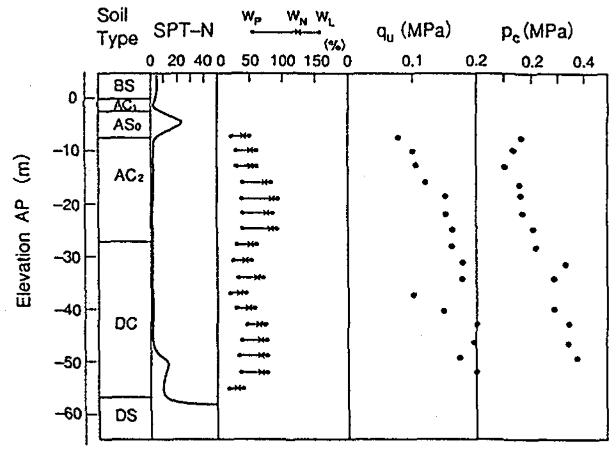


Fig. 1 Soil Conditions at the Construction Site (Tanaka et al., 1989)

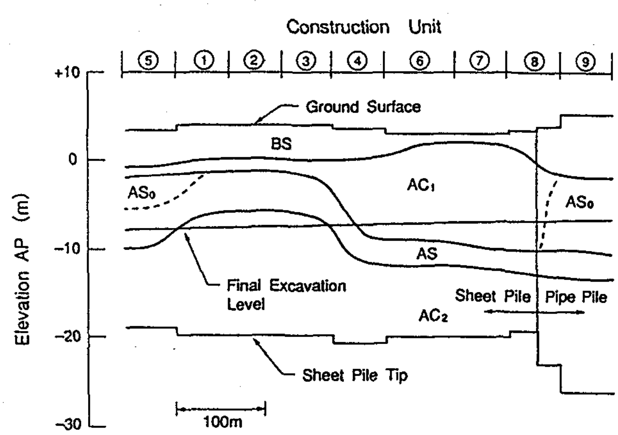


Fig. 2 Soil Profile in Longitudinal Section and Layout of Construction Section (Tanaka et al., 1989)

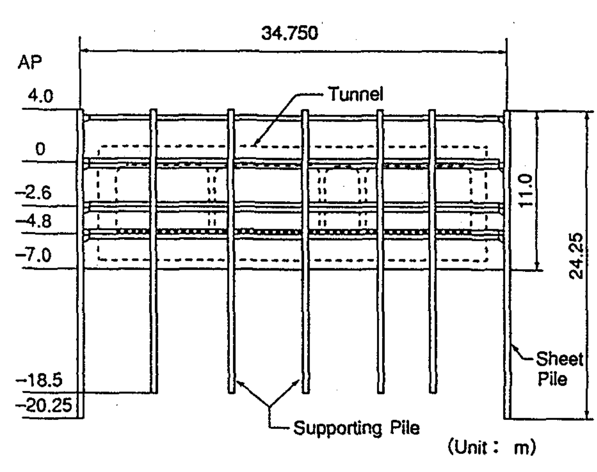


Fig. 3 A Typical Cross Section Showing Bracing Method (Tanaka et al., 1989)

3.2 Bracing Method

Bracing methods were designed based on the conventional design method. A typical cross section is shown in Fig. 3, and the material properties of the structural elements, such as sheet piles and braces, are summarized in Table 2. To form the retaining walls, V_L type steel sheet piles were driven to a depth of approximately 25 m. However they did not reach a firm stratum (such as DS) since the area was covered with soft clays of more than 50 m thickness as shown in Fig. 1. Four sets of braces were installed at a pitch of 5 m along the length of the road. To prevent the braces from buckling, supporting piles were installed transversely with an approximate interval of 6 m as shown in Fig.3.

3.3 Observed Behaviour

In the Construction Units 1 to 4, an extensive measurement program was planned to serve as a safety measure during construction.

In the Construction Units 1 to 3, large lateral deformations of the retaining walls and significant heave of the supporting piles were observed as shown in Figs. 4 and 5. The shaded portion of Fig.4 corresponds to the undrained creep displacement before the installation of the braces. It may be seen in the figure that the maximum horizontal displacement observed after the 4th excavation was 300 mm, and that considerable amount of displacements were measured even at the bottom tip of the sheet pile. Fig. 5 shows the heave of the supporting piles measured at the pile heads in the Construction Unit 1.

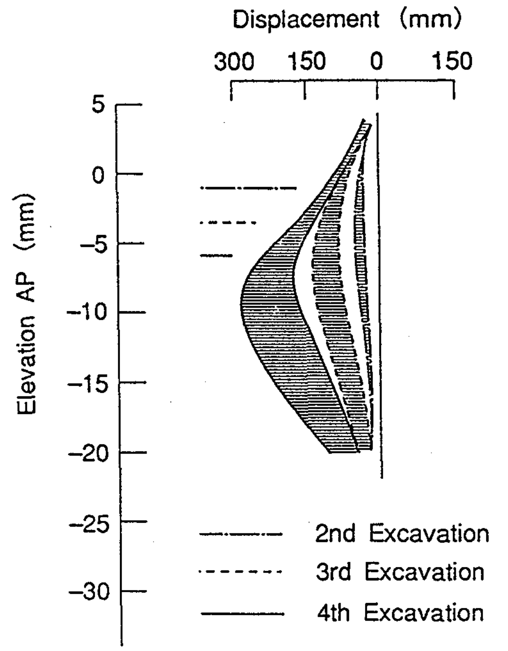


Fig. 4 Measured Horizontal Displacement of Steel Sheet Pile (Tanaka et al., 1989)

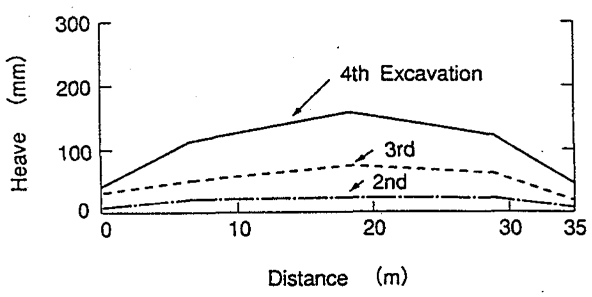


Fig. 5 Measured Heave of Supporting Piles (Tanaka et al., 1989)

With the large ground deformations described above, the 4th excavation was halted and the following remedial measures were taken: 1) removing the soils from the behind of the retaining walls; and 2) lowering the ground water level by the deep well method. As a result, the creep deformations of the sheet pile walls ceased, and the 4th excavation was completed. (The excavations in the Construction Sections 5 to 8 were completed without significant problems, where the maximum lateral displacement of sheet pile walls was less than 130 mm, and the maximum heave of the supporting piles was less than 100 mm.)

4. Numerical Modelling

4.1 Undrained Shear Strength of Clay for Excavation Analysis

Tsuchida et al. (1989) proposed the following formula for determining undrained shear strength of clay, c_u* to be employed in the stability analysis using a circular sliding surface method:

$$c_u^* = c_1 \times c_2 \times c_3 \times c_4 \times c_5 \times (q_u/2) \qquad \text{..... [1]}$$

where

q_u = average value of unconfined compressive strength,

c_1 = correction factor for sampling disturbance ($c_1 > 1.0$),

c_2 = correction factor for strength anisotropy,

c_3 = correction factor for shear strain velocity ($c_3 < 1.0$),

c_4 = correction factor for the assumption that the angle of the slip surface from the vertical plane is 45 degrees ($c_4 < 1.0$), and

c_5 = factor to correct the difference between the axisymmetric and plane-strain conditions ($c_5 > 1.0$).

In Japan, undrained shear strength of soft clay is usually obtained from unconfined compression test (Nakase, 1967). Tanaka and Tanaka (1993) studied the difference between the vane shear strength and q_u at several sites in Japan, and have shown that their ratio is slightly dependent on plasticity index I_p, and that average ratio is around 1.0.

In the excavation analysis with large wall displacements, it is anticipated that the clay layers at the bottom of the excavation deform to such large extent that the strain level at the peak strength is exceeded. Thus, it is probable that the mobilized strength is between the peak and residual strengths as shown in Fig. 6. It is also anticipated that there are other unknown factors such as the effect of vibration caused by the construction work on the undrained shear strength of clay. To incorporate these factors into the analysis using elasto-perfectly plastic models, a new correction factor, c_6 (< 1.0), may be introduced as:

$$c_u^e = c_1 \times c_2 \times c_3 \times c_4 \times c_5 \times c_6 \times (q_u/2) \qquad ..... [2]$$

where c_6 = correction factor to take the effect of strain softening and other unknown factors into account.

4.2 Finite Element Mesh

Fig. 7 shows the finite element mesh used in the analysis. Soils are discretyzed as 8-noded isoparametric quadrilateral elements, whereas sheet piles are modelled by 3-noded isoparametric beam elements. Between the beam and the solid elements, 6-noded joint elements are used to simulate the sliding between the soils and sheet pile walls.

4.3 Simulation of Excavation

Construction sequence was simulated in the following 8 stages :

Stage 1: Set initial stresses in the soil and joint elements (γ_t and K_0 in Table 1 are used);
Stage 2: 1st excavation;
Stage 3: 1st bracing, and pre-loading(100 kN/m);

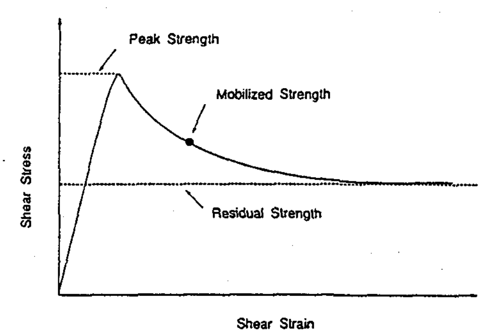


Fig. 6 Shear Stress-Strain Relationship Showing Strain Softening

Stage 4: 2nd excavation;
Stage 5: 2nd bracing, and pre-loading(160 kN/m);
Stage 6: 3rd excavation;
Stage 7: 3rd bracing, and pre-loading(0 kN/m);
Stage 8: 4th excavation.

The pre-loads were originally designed as 80, 160, 240 kN/m in Stages 3, 5, 7, respectively(Tanaka et al., 1989). The pre-loads employed in the present study are estimated values from a number of calculations and measured brace stresses shown in Table 4.

4.4 Material Properties

Material properties of the soils used in the calculation are summarized in Table 1. Soils are modelled as elasto-perfectly plastic material obeying an associated flow rule. Analyses have been performed in total stresses, introducing following equivalent shear strength parameters for a sand layer: $c^{eq} = -u \times \tan\phi'$; $\phi = \phi'$, where u denotes pore water pressure in a sand layer. The following empirical relationships were used to estimate the Young's modulus of soils:

For soft clay:	$E = 210\ c_u$	(Takenaka, 1962)
For sand:	$E = 2800\ N$ (kPa)	(Uto, 1967)

where c_u=undrained shear strength of clay, and N=SPT blow count(Sekiguchi and Nanbu, 1994). For diluvial clay and sand, the values of the Young's modulus were doubled based on the comparison between the observed and calculated ground heave; perhaps, the 3D effect and the strain level effect are included in this adjustment.

Material properties of the beam elements are summarized in Table 2. The efficiency factor of the sheet pile, η, was assumed to be 40% based on Tanaka et al.(1989). The efficiency factor of the brace, α, was set at 0.1 based on parametric studies, and the results reported by Okada and Kurihara (1985) and Nishino and Kukita (1985).

The shear and normal stiffnesses of the joint element were set as 10^5 kN/m^3 and 10^7 kN/m^3, respectively (Sekiguchi et al. 1992). The shear strength of the soil/sheet pile interface was determined by using the following empirical relationships:

For soft clay:	$c_{joint} = c_{clay}$	(API, 1989)
For sand:	$\phi_{joint} = 0.72\ \phi_{sand}$	(Potyondy, 1961)

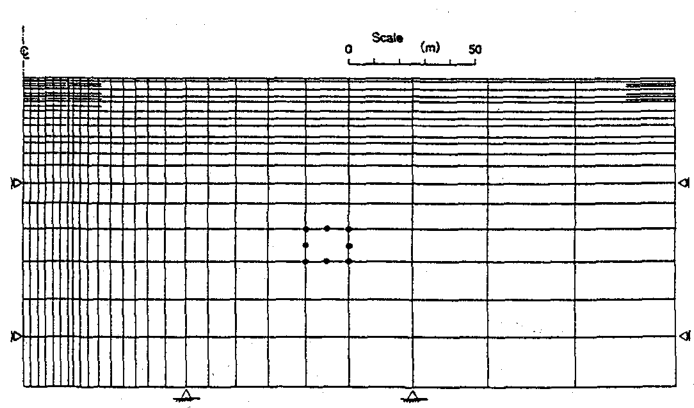


Fig. 7 Finite Element Mesh Used for Calculation

4.5 Cases Analyzed

A number of calculations were performed; where the effect of the following parameters were studied: Young's modulus of diluvial clay and sand; Poisson's ratio of clays; the efficiency factor of the brace, α; pre-loads of braces; and the correction factor, c_6. From the parameter study, it appeared that the correction factor, c_6, plays an important role among various parameters. For space limitations, two representative cases shown in Table 3 are discussed in the present paper. Case Q may be regarded as a conventional approach where the undrained shear strength is equivalent to one half of the unconfined compressive strength, whereas Case S may be regarded as a new approach where the effect of strain softening and other unknown factors are incorporated.

Table 3 Cases Analyzed

Case	c_6	c_u^e	Depth
Q	1.139	$1.00\times(q_u/2)$	from AP. -5.8 m to AP. -20.25 m
S	0.284	$0.25\times(q_u/2)$	at AP. -5.8 m
	0.570	$0.50\times(q_u/2)$	at AP. -20.25 m

$c_u^e = c_1 \times c_2 \times c_3 \times c_4 \times c_5 \times c_6 \times (q_u/2)$
$c_1 = 1.33$ (Tsuchida et al., 1989)
$c_2 = 0.75$ (Tanaka et al., 1989, Fig. 49)
$c_3 = 0.88$ (Tsuchida et al., 1989)
$c_4 \times c_5 = 1.0$ (Hanzawa, 1982; Tsuchida, 1989)

5. Results of Analyses

5.1 Case Q

Fig. 8 shows measured and calculated horizontal displacements of the sheet pile wall after the 3rd and 4th excavations. Though the measured data indicate creep displacements, the present calculation does not consider time-dependent deformation. As shown in the figure, the agreement between the measured and calculated results is poor.

Fig. 9 compares measured heaves of the supporting piles, and calculated heaves of the ground at AP. -7.0 and -20.25 m levels after 3rd and 4th excavations. It may be seen in the figure that the calculated ground heave is in reasonable agreement with the measured results.

Shown in Fig. 10 are measured and calculated soil pressures on the sheet pile walls after the 4th excavation. The figure indicates little difference between the measured and calculated results. The calculated soil pressures however are greater than those measured beneath the bottom of the excavation. On the excavated side, the Rankine's passive pressure is also plotted in the dotted line and is compared with the calculated pressure. It is inferred that the soil/bracing system under consideration (with the assumed soil strength parameters) has a reasonable safety factor against the passive failure at the bottom of the excavation. This is consistent with the distribution of the plastic zones in the soil mass shown in Fig. 11, in which no significant plastic zone can be observed beneath the bottom of the excavation.

Table 4 summarizes the measured and calculated maximum brace stresses. As can be seen in the table, the agreement between the measured and calculated results for Case Q is poor.

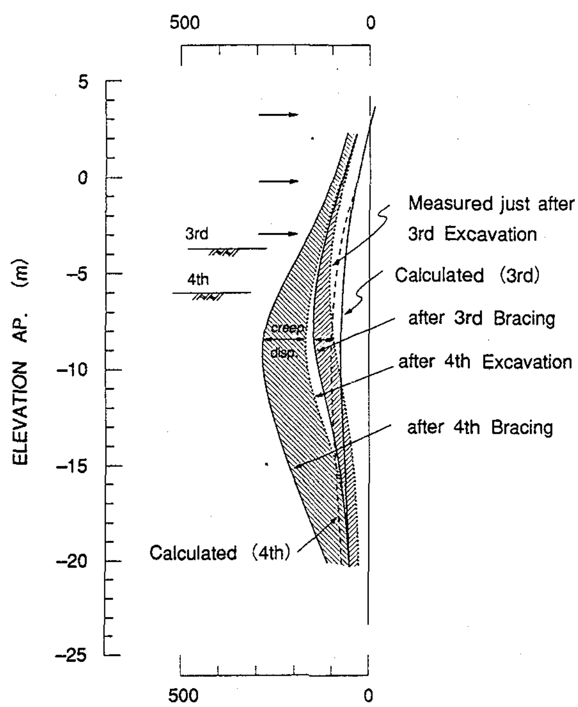


Fig. 8 Measured and Calculated Horizontal Displacements of Steel Sheet Pile (Case Q)

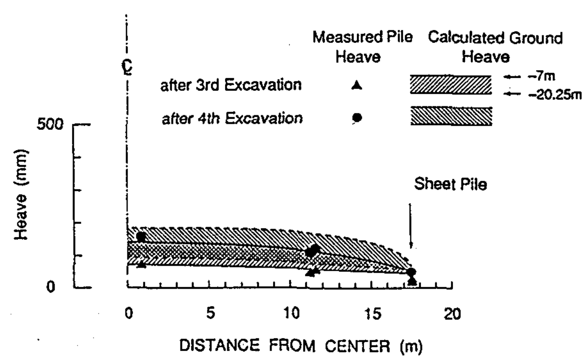


Fig. 9 Measured and Calculated Ground Heave (Case Q)

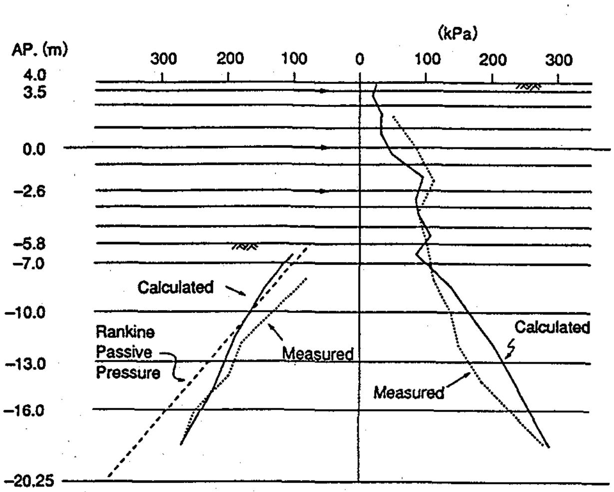


Fig. 10 Measured and Calculated Soil Pressures on The Sheet Pile Wall after 4th Excavation (Case Q)

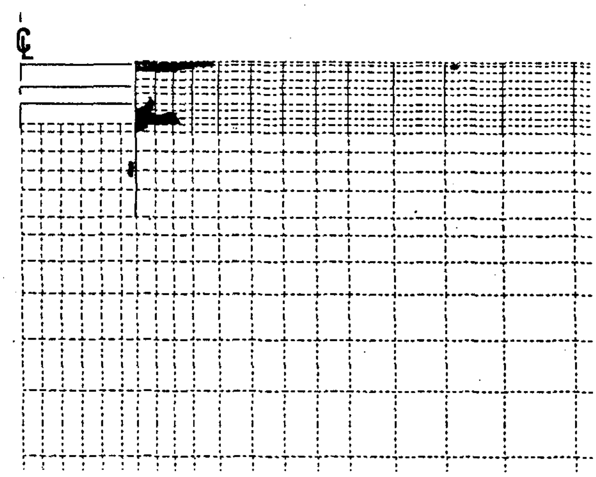


Fig. 11 Distribution of Plastic Zones in The Soil Mass (Case Q)

Table 4 Comparison of Measured and Calculated Maximum Brace Stresses (Unit: x10 kN/m)

	Measured	Calculated	
	(at Unit 3)	Case Q	Case S
1st Brace	17.6	10.3	12.3
2nd Brace	43.9	34.4	45.7
3rd Brace	24.0	10.0	24.3

From the above mentioned comparisons between the measured and calculated results, it may be stated that the assumed soil strength parameters for Case Q are not consistent with the measured results in the field.

5.2 Case S

The measured and calculated horizontal displacements of the sheet pile wall after the 3rd and 4th excavations are compared in Fig. 12. It may be seen in the figure that the agreements between the calculated and measured results are fair to good.

Fig. 13 compares the measured heave of the supporting piles, and the calculated heaves of the ground at AP. -7.0 and -20.25 m levels after 3rd and 4th excavations. The calculated ground heave appears in reasonable agreement with the measured results, as in Case Q. It is interesting to note that the ground heave at a level of AP.-7m has a maximum value at a distance of 6 m from the sheet pile wall. This will also be observed in Fig. 14 in which the calculated ground deformation after the 4th excavation is plotted, and possible reason for that will be given.

Fig. 15 shows measured and calculated soil pressures on the sheet pile walls after the 4th excavation. It may be seen in the figure that the agreement between the measured and calculated results are good to excellent. Since the calculated soil pressures are very close to the Rankine's passive pressures beneath the bottom of the excavation, it is inferred that the soil/bracing system under consideration (with the assumed soil strength parameters) is on the verge of a passive failure at the bottom of the excavation. This is also indicated by the distribution of the plastic zones in the soil mass shown in Fig. 16, in which significant plastic zone can be observed beneath the bottom of the excavation.

Table 4 summarizes the measured and calculated maximum brace stresses. As can be seen in the table, the agreement between the measured and calculated results in Case S is very good.

From the results of the examinations given above, it may be stated that the assumed soil strength parameters for Case S are consistent with the measured results in the field.

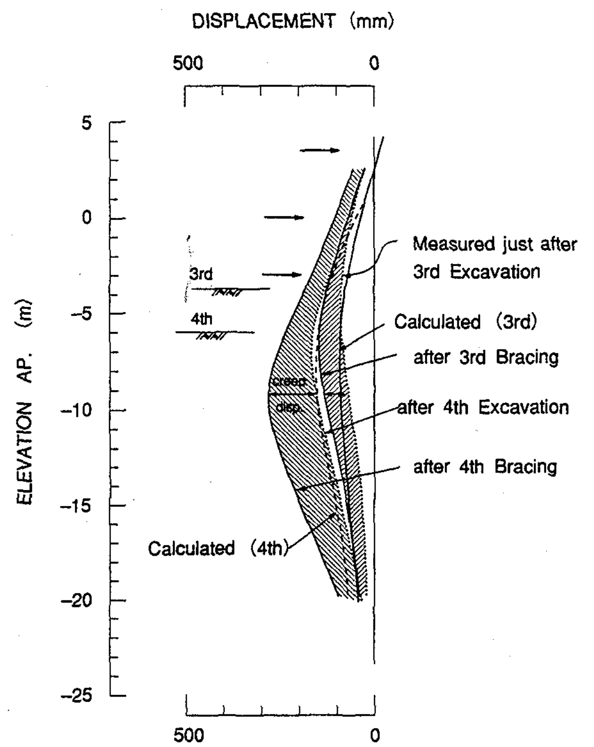


Fig. 12 Measured and Calculated Horizontal Displacement of Steel Sheet Pile (Case S)

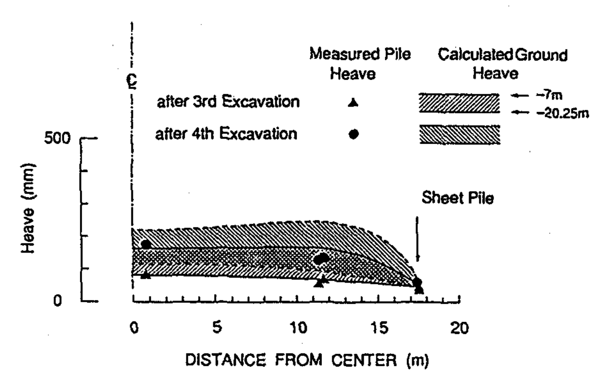


Fig. 13 Measured and Calculated Ground Heave (Case S)

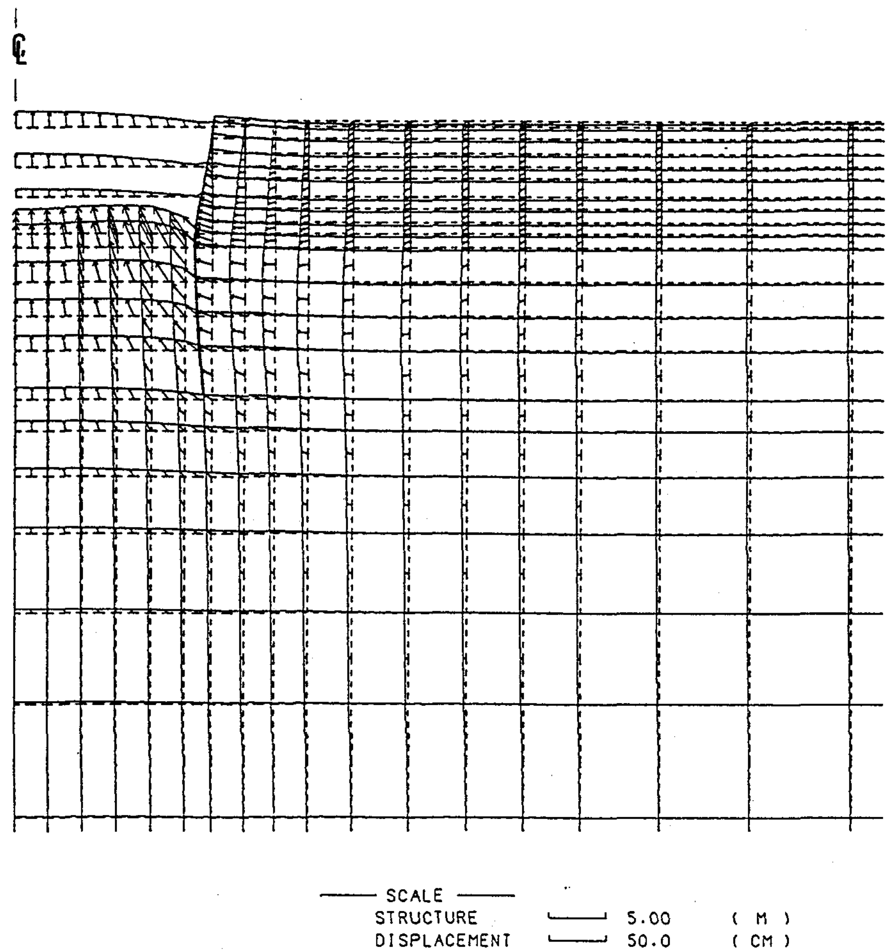
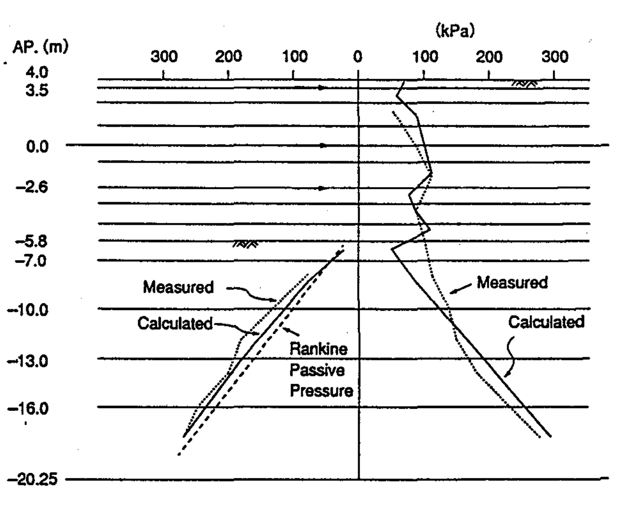


Fig. 14 Calculated Ground Deformation after 4th Excavation (Case S)



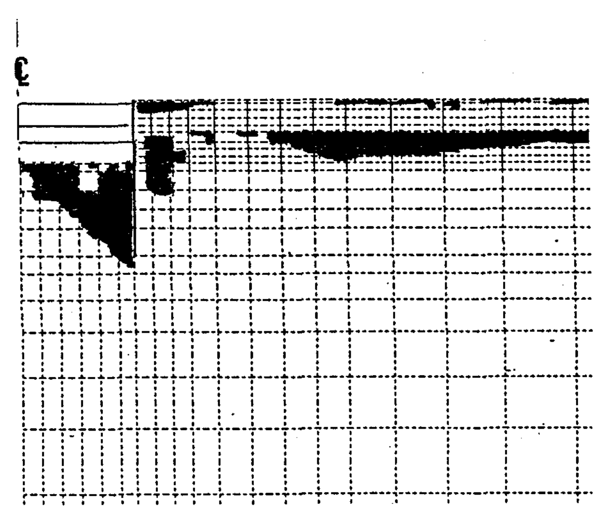


Fig. 15 Measured and Calculated Soil Pressures on The Sheet Pile Wall after 4th Excavation (Case S)

Fig. 16 Distribution of Plastic Zones in The Soil Mass (Case S)

6. Summary and Conclusions

A case history of a braced excavation in a very thick soft clay deposit was analyzed by a finite element method with the viscoplastic algorithm (Zienkiewicz and Cormeau, 1974), incorporating non-linear joint elements for soil/sheet pile interfaces (Sekiguchi et al., 1990, 1992). From the results of the study, the following summary and conclusions may be drawn:

(1) It was found that the analysis method employed was capable of analyzing the "plastic flow" problem without numerical problems where large plastic zones are generated in the soil mass and significant slippage is experienced in the soil/sheet pile interfaces.

(2) It was found that the effect of strain softening of soils has to be taken into account when employing the finite element method with the elasto-perfectly plastic models. With a reasonable reduction in shear strength of soils, the results of the analysis proved quite consistent with those observed in the field. In the present case analyzed, the reduction was of the order of 28 to 57%, in which other unknown factors are also included.

Based on the results of the analyses reported in this paper, it would appear that the analysis method proposed can be a tool for designing and analyzing braced excavation problems in thick soft clay deposits. However, the authors feel that the following subject requires additional investigation to fully understand the case history analyzed in this paper: (1) to separate and quantify the effect of strain softening (Adachi et al., 1991) and other unknown factors; and (2) to study the problem by using a finite strain finite element method incorporating a strain softening constitutive equation of soils, which is capable of simulating strain localization in the soil mass (Oka et al., 1994).

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