First Korea-Japan Joint Geotechnical Seminar
October 18-20, Seoul, Korea

SOIL MECHANICS RELATED TO DEEP EXCAVATIONS AND TUNNELING

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INTRODUCTION

In order to properly design and construct an earth structure such as deep excavation or earth tunnel, it is very important to understand the actual behavior of the adjacent ground during excavation phase and to establish an analytical model to describe this behavior as accurately as possible. Furthermore in recognition of the difficulties and uncertainties associated with the determination of earth design parameters from laboratory experiments and site explorations, it is essential to provide a numerical procedure for the determination of the constitutive parameters of soil by back-analysis according to measurements made in the adjacent ground of the construction. Consequently, this procedure of back-analysis based on the site measurements in early stages of an excavation will permit the fine tuning of the design parameters, thereby improving the design of subsequent stages of construction.

Firstly in this lecture, a reasonable finite element procedure by making use of the stress pass method is shown to predict the behavior of a braced excavation and an application of this procedure to actual case studies of deep excavation in soft ground is discussed in details.

Secondly, it is proposed a numerical procedure to back-analyze the non-linear constitutive parameters for joint elements in simulation of particular discontinuities occurring in sandy ground during tunnel constructions. The proposed procedure is applied to actual case studies and comparisons are made with analytical results based on parameters obtained in laboratory experiments.

APPLICATION OF STRESS PATH METHOD TO A DEEP EXCAVATION

Introduction

For deep braced excavations in soft ground, the movement of the soil surrounding an excavation must be taken into account with considering the soil-retaining wall interaction. Excavations had been first investigated in terms of stability and the forces to be sustained by the retaining walls. However, interest in the deformations of ground around excavations grew at the beginning of the 1960's and many measurements have been made. Although various theoretical works have been done by using finite element method and analytical results show fair agreement with the measurements, there still remains questions in constitutive modeling for soils, that is, non-homogeneity, anisotropy, stress-strain relationship under unloading conditions.

A way to overcome this problem is the use of "Stress Path Method"[1], namely stress-strain relationships used in the F.E. analysis can be experimentally obtained for stress paths similar to actual stress paths. Using this concept, Tominaga et al.[2] developed a F.E. back-analyzing procedure to simulate the behavior of braced excavations. We will discuss their works in detail.

Procedure

Fig.1 shows the stress paths for two soil elements A(behind retaining wall) and B(below the bottom of the cut) during excavation. In soil element A, the total horizontal stress σ_h decreases from K_0-state due to the excavation, but the total

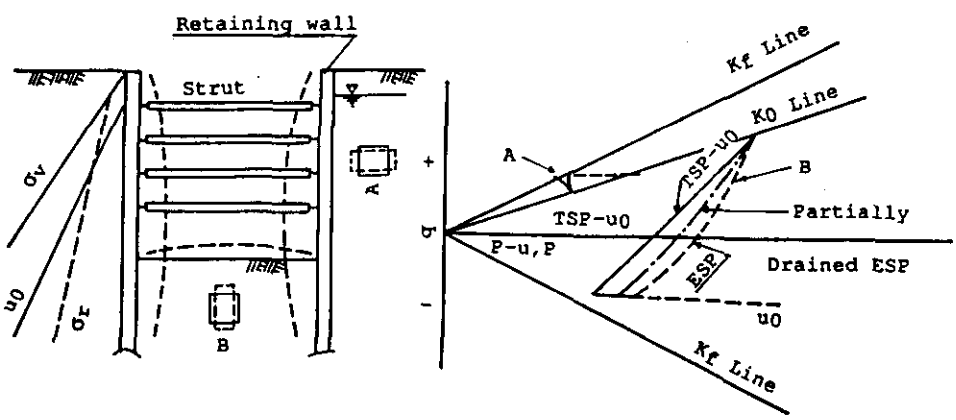


Fig.1 Stress Path for Normally Consolidated Soil.

vertical stress σ_v remains nearly constant. In the soil element B, the total vertical stress σ_v decreases while the change of total horizontal stress σ_h is small. For normally consolidated clays, the effective stresses decrease during undrained shear deformation process due to the increase of pore water pressure. Because the soil mass at the site is under the partial drainage conditions, actual effective stress paths locate between the undrained effective stress path(ESP) and the total stress path(TSP). When the water pressure in the subsoils decreases due to the change of underground water head, the increase of effective stress results in the volumetric contraction.

Based on the above discussions, the stress path for the element A can be simulated by unloading compression test, while unloading extension test is for the element B. From the test results, the required soil parameters, such as Young's modulus E, Poisson's ratio ν and pore water pressure parameter a are obtained as follows;
from the unloading compression test; $\Delta\sigma_a = 0$, $\Delta\sigma_r \rightarrow$ decrease,

$$E_r = \frac{-2\Delta\sigma_r}{\varepsilon_a - 2\varepsilon_r}, \quad \nu_{ar} = \frac{\varepsilon_a}{\varepsilon_a - 2\varepsilon_r} \qquad (1)$$
$$a = (\Delta u - \Delta\sigma_{oct}) / \Delta\tau_{oct}$$

from the unloading extension tests; $\Delta\sigma_a \rightarrow$ decrease, $\Delta\sigma_r = 0$,

$$E_a = \frac{\Delta\sigma_a}{E_a}, \quad \nu_{ra} = \frac{\varepsilon_r}{\varepsilon_a} \qquad (2)$$
$$a = (\Delta u - \Delta\sigma_{oct}) / \Delta\tau_{oct}$$

where $\Delta\sigma_a$ and $\Delta\sigma_r$; axial and radial stress changes, ε_a and ε_r; axial and radial strain components, E_a and E_r; axial and radial Young's moduli, ν_{ar} and ν_{ra}; Poisson's ratios, $\Delta\sigma_{oct}$ and $\Delta\tau_{oct}$; changes of octahedral normal stress and octahedral shear stress.

E_a and E_r obtained from the test results are corresponding to the field Young's moduli E_v(the vertical direction) and E_h (horizontal direction), respectively.

Fig.2 shows the flow of this back-analyzing procedure to predict a braced excavation behavior. This numerical pro-

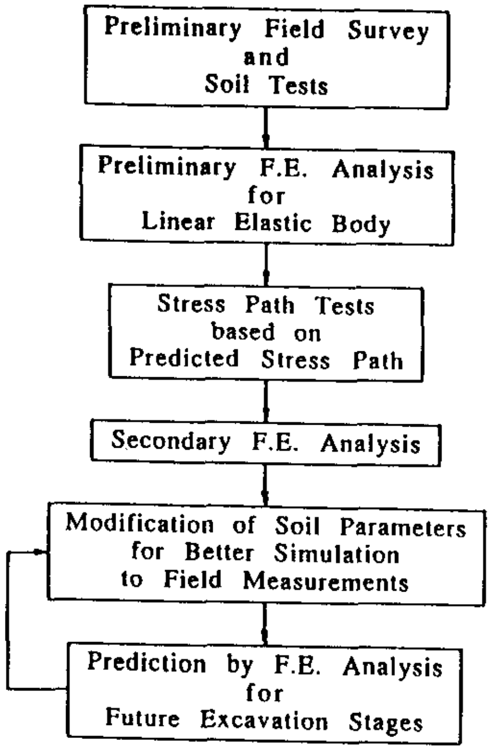


Fig.2 Flow Chart of Predictional Analysis.

gram adopts an elasto-plastic constitutive model with taking into account anisotropy in both strength and elastic behavior, and consolidation process as well. In the analysis, both bar and beam elements can be used to model the retaining structures.

Case Study

(1) Site Conditions

For a construction of sewage treatment facilit... a large and deep braced excavation was made in the north-east district of Tokyo. Fig.3 shows the plane and a typical cross section of the excavation with 54.0m wide x 80.5m long x 26.0m deep. 50m long steel sheet pipe piles braced with steel struts were placed as the temporary retaining structures. Prior to the excavation, reinforced concrete piles (2.0m in diameter) were placed at 8.0m spacing to the depth ranging from -22m to -55m and soil improvement by lime piles was conducted from depth of -6m to -22m in the cutting side.

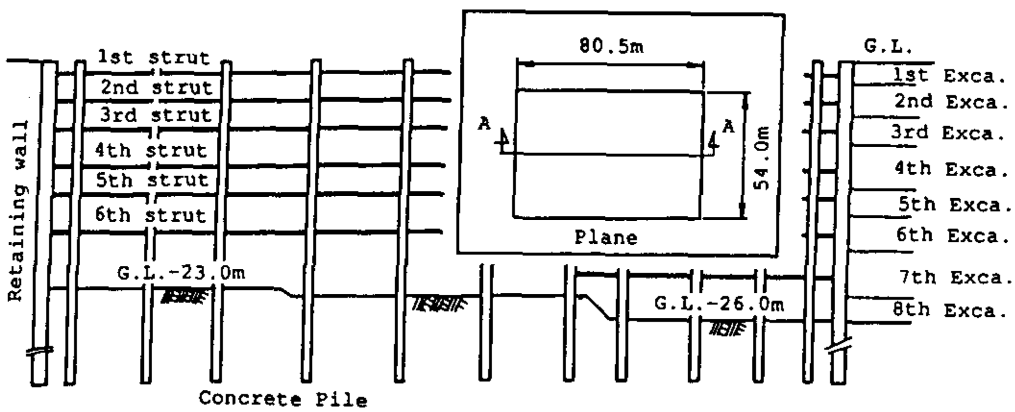


Fig.3 Typical Cross Section of A-A Line.

As shown in Fig.4 the soils from ground surface to a depth of -30m consist of alluvial deposits, such as upper silty sand((AS) 0m ~ -6m) overlying soft silty clay ((AC) -6m ~ -30m). Below the depth of -30m, 15m thick diluvial sandy silt(DC) is overlaying very dense dandy soil(DS). The normally consolidated AC layer has the water content W_n = 55 ~ 60%, the undrained shear strength S_u = 2.9 x depth kPa and the preconsolidation pressure p_c (= the effective overburden pressure(σ_v)). From both the horizontal earth pressure and pore water pressure measurements, the earth pressure coefficient at rest K_0 was obtained as 0.4 for the upper part of AC layer but 0.5 for the lower part of AC layer. As the strength parameters, c' = 0 kPa and ϕ' = 34 ~ 37° for AC layer, c' = 50 kPa and ϕ' = 23° for DC layer were obtained from conventional triaxial compression test results.

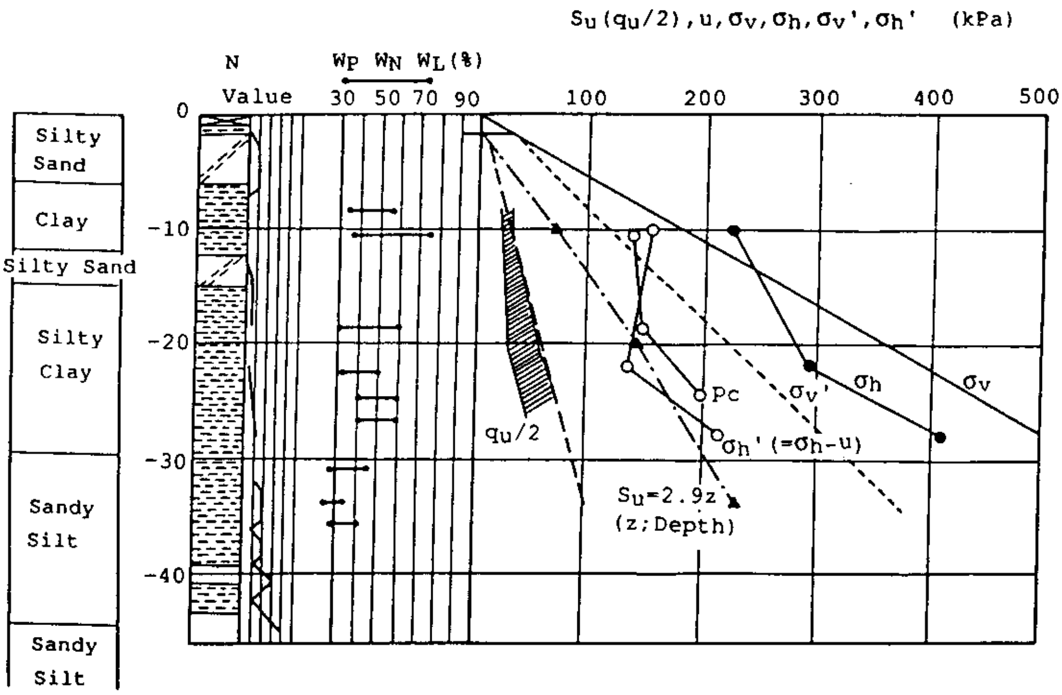


Fig.4 Soil Profile.

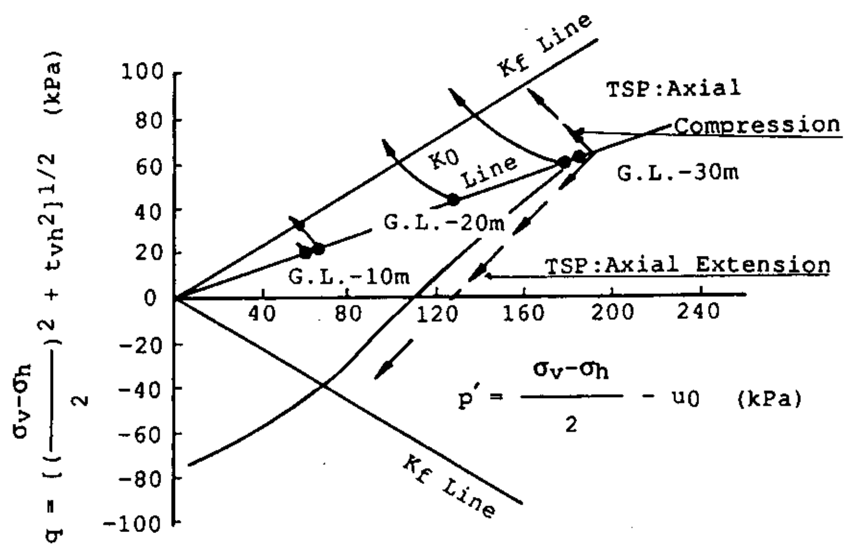
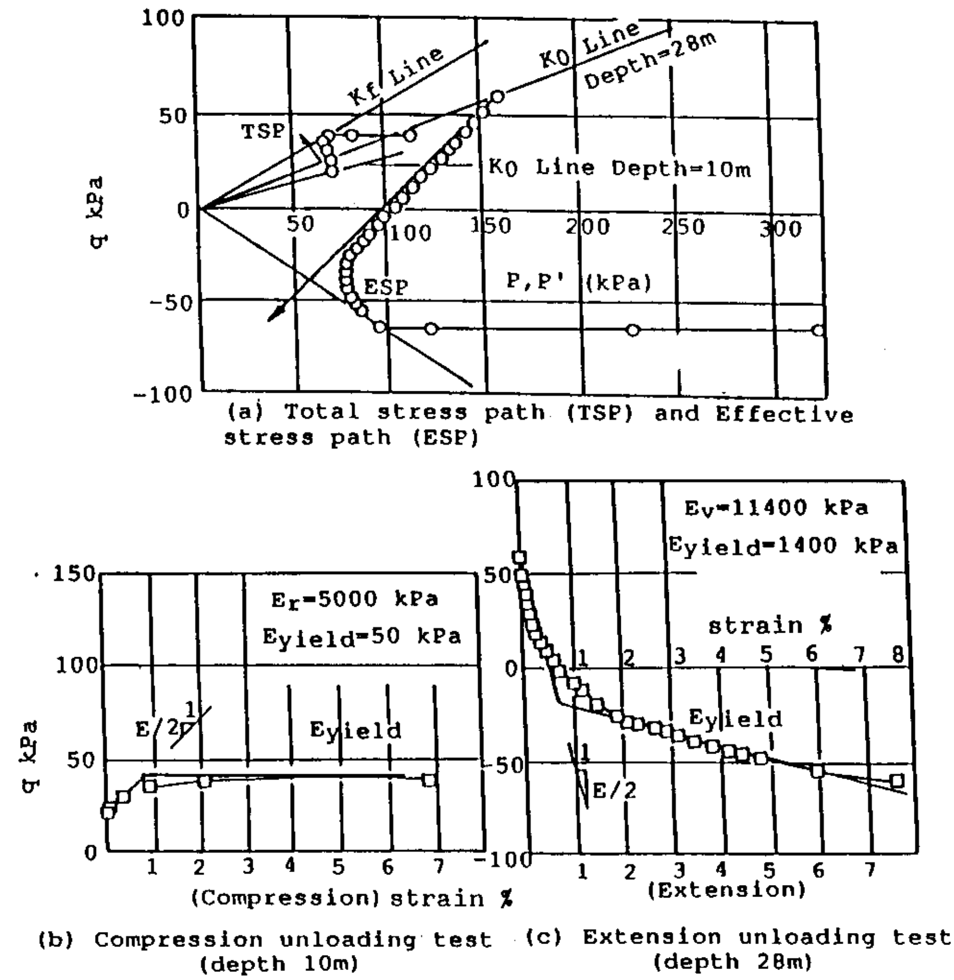


Fig.5 Typical Total Stress Path from Preliminary F.E. Analysis.

(2) Stress Path Test

In order to determine the remaining soil parameters, a series of stress path laboratory tests was conducted by using soil samples taken from the excavation site. The prescribed stress paths used in the tests were calculated from the preliminary F.E. analysis and Fig.5 shows the calculated stress paths for some typical soil elements located at the depth of -10m, -20m, and -30m. From the



(a) Total stress path (TSP) and Effective stress path (ESP)

(b) Compression unloading test (depth 10m)

(c) Extension unloading test (depth 28m)

Fig.6 Results of Stress Path Test(CK_0U).

analytical results, it is seen that the stress state in the soils behind the retaining wall is under unloading compression process but that in the soils below the bottom of the cut is under unloading extension, as previously mentioned. Thus, triaxial unloading compression and extension tests were performed under both drained and undrained conditions. Fig.6 gives some undrained test results, i.e., (a) total and effective stress paths, (b) stress-strain relationships of unloading compression test and (c) stress-strain relationship of unloading extension test.

(3) Determination of Soil Parameters

The soil parameters(Young's moduli, Poisson's ratio and pore water pressure parameter) for the secondary F.E. analysis were determined mainly from the stress path test results. Fig.7 shows the variations of E_a, E_r, ν_{ar}, ν_{ra} against the change of the stress ratio q/p(q; shear stress, p; mean principal stress). On the other hand, Young's modulus E for the diluvial clay was evaluated from the conventional triaxial compression test results as given below;

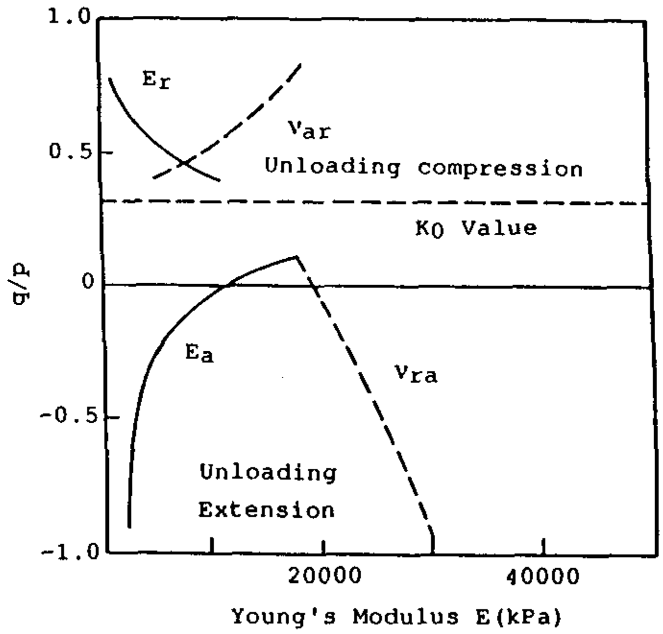


Fig.7 Obtained Elastic Moduli and Poisson's Ratio(CK_0D, Depth = -20m).

$$E = 3000 + 800\,\sigma_0' \ \ (kPa) \tag{3}$$

where σ_0'; the initial mean consolidation pressure.

(4) Deformation Behavior of Braced Excavation

Comprehensive observations of both retaining structures and surrounding ground during the excavation were carried out by using inclinometers and settlement gages. By the first excavation 50mm of the maximum lateral movement of the wall takes place at the ground surface as shown in Fig.8. With the progress of excavation, however, it is evident that a substantial part of the movement took place at a given elevation before the depth of excavation had reached that level. The areal influence of excavation extends more than twice the excavation depth from the wall.

(5) Comparison of the Predictions and Measurements

Prior to the excavation, the behavior at five excavation stages(i.e., first to 5th excavation) had been predicted by the secondary F.E. analysis using the laboratory test results. However, the prediction showed that 150mm of the maximum lateral movement of the wall took place at -3.0m below the ground surface after the first excavation. The predicted maximum lateral movement was three times the observed value and the location was also different.

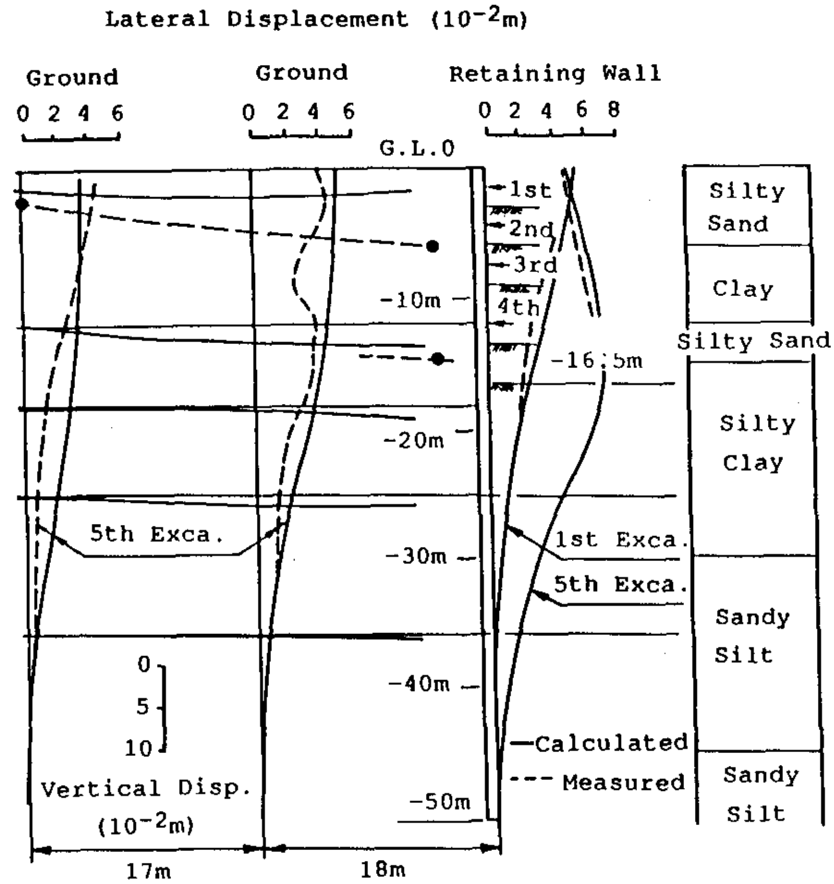


Fig.8 Comparison between Predicted and Actual Field Measurements.

In order to improve the predictions for successive excavation stages, a trial had been done to modify the soil parameters for the cutting ground. Namely, the ground was reinforced by sets of reinforced concrete piles and improved by sets of lime columns. To take into account this effect in the analysis, the following equivalent Young's moduli for the cutting ground were introduced;

for the ground reinforced by concrete piles(at depth from -22m to -55m)

$$E_{eq} = \alpha_1 \, [\, (EA)_{soil} + (EA)_{pile} \,] \, / \, A \tag{4}$$

for the ground improved by lime columns(at depth from -6m to -22m)

$$E_{eq} = \alpha_2 \, E_{soil} \tag{5}$$

where A; the total cross sectional area($A = A_{soil} + A_{pile}$), α_1, α_2; the modification factors. By using thus defined E_{eq} in the back-analyses to simulate the behavior during the first excavation, $\alpha_1 = 6.0$ and $\alpha_2 = 2.0$ were determined as the optimum modification factors. The predictions for the successive excavation stages showed good agreement with the observations as shown in Fig.8.

APPLICATION OF CONVENTIONAL METHOD TO URBAN TUNNELS

Introduction

It has become more and more common to plan and construct new urban railway and highway systems underground for the purpose of environmental protection, especially in commercial and residential areas. From the viewpoint of construction and maintenance costs for drainage, ventilation and traffic safety, as well as for the convenience of the passengers, it is preferable to have as thin an earth cover as possible.

Since the major cities of Japan are located in vast alluvial plains where soft ground and high ground water tables prevail, tunnels must be built safely in order to avoid damage to adjacent or overlaying buildings, streets and/or utilities, and they must be durable ones. In the past, these urban tunnels have generally been constructed by the cut and cover or the shield method. In recent years, however, implementation of the conventional method has increased in lieu of the shield method. This has resulted in a reduction of tunneling costs to a great extent. At first, the present situation of the application of the conventional method to urban tunneling in Japan is discussed.

For shallow tunneling the following problems must be investigated carefully;

1) The stability of the tunnel face and surrounding ground,
2) Problems related to underground water conditions,
3) The effect of tunneling on adjacent or overlaying structures and/or utilities,
4) Evaluation of safety during construction and establishment of observational methods,
5) Long term durability of the tunnel with taking into account all possible items related to future adjacent developments.

In order to solve these problems, it is most important to understand the actual behavior of the surrounding ground during tunnel excavation and the interaction between tunnel structure and surrounding ground, and to establish an analytical method which can simulate the behavior and the interaction as accurately as possible. Therefore, the particular characteristics in the mechanical behavior of shallow sandy ground tunnels is studied and an analytical method, which can simulate its behavior, is proposed. Furthermore, a numerical procedure is derived to back-analyze the non-linear constitutive parameters for joint elements, by simulating particular discontinuities occurring in sandy ground during tunnel constructions. Subsequently this procedure is applied to actual case studies.

The Conventional Method in Urban Tunneling[3]

Although in the past the urban tunnels have been built by the cut and cover or the shield method, today the use of the conventional method has been increased in order to reduce the construction costs. Since the conventional method requires less expenses for machines and equipments, it has economical advantages as shown in Fig.9. Especially when the tunnel length is relatively short and/or the cross-section must be varied, the advantage of the conventional method is obvious.

However, the conventional method should be improved so that it can be applied even to soft ground with a thin cover and high ground water head, because unsolidified alluvial and/or diluvial strata prevail in most of the urban regions of our country. An inquiry was made to determine the applicability of both, the conventional and shield methods, to unsolidified ground with thin cover(less than 50m). After the investigation, it became clear that the conventional method was adopted when the following conditions were met.

1) Although the conventional method has been used even under very severe conditions, such as in crowded residential areas and adjacent to important structures, ground conditions worse than reflected by diluvial formation have been avoided.

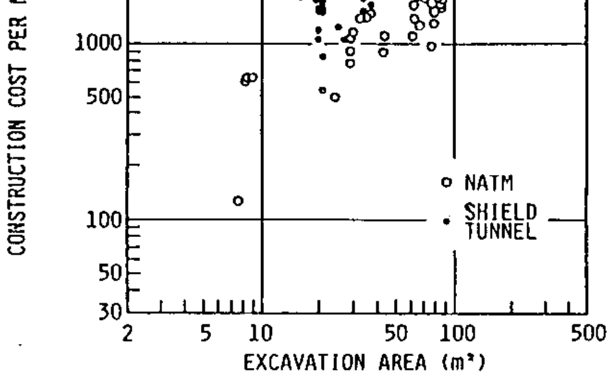


Fig.9 Relation of Construction Costs per Meter to the Tunnel Cross-Sectional Area.

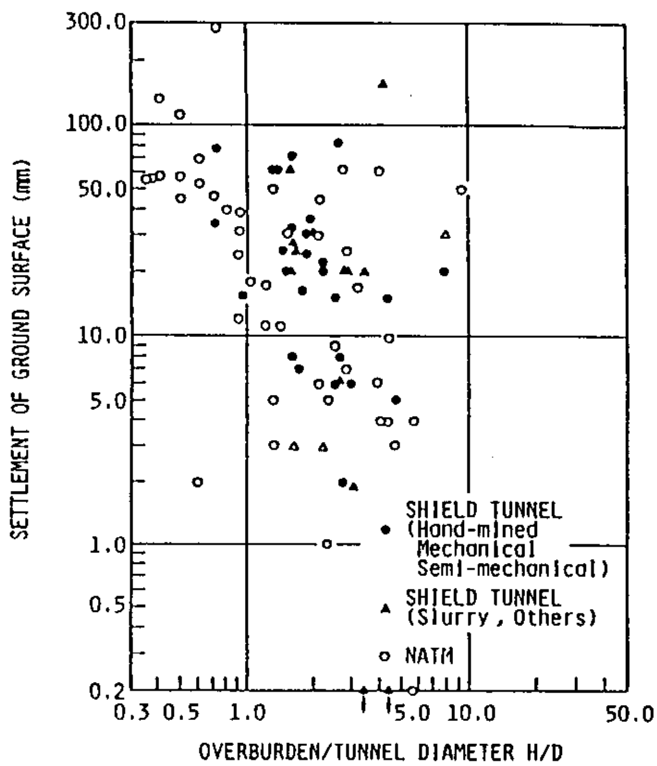


Fig.11 Relation between Surface Subsidence and Overburden Ratio for the Shield Method and the Conventional Method.

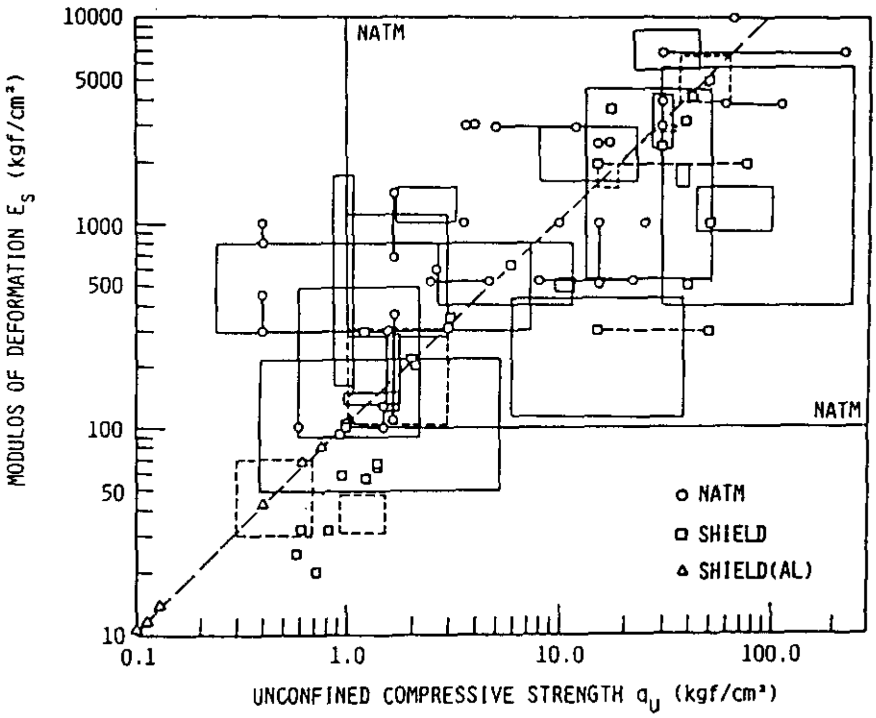


Fig.10 Uniaxial Strength and Modulus of Deformation as Decisive Parameters for the Conventional Method and the Shield Method.

2) Fig.10 shows the applicability of both the shield and the conventional method from the point of view of the mechanical properties of ground. From this figure one understands, that the conventional method has been used when the uniaxial compressive strength q_u is larger then 1kgf/cm2 and the modulus of deformation Es is more than 100kgf/cm2.
3) The most important problem in urban tunnel construction is how to control the ground water condition. Since the tunnel face becomes inevitably unstable when the total amount of water inflow exceeds 100 l / min, a proper countermeasure of dewatering should be accepted.

At any rate, some auxiliary methods are required for dewatering, face stabilization and/or prevention of surface settlements when the conventional method is applied to urban tunneling with thin cover and high ground water head. It is always necessary to improve the technique, including the auxiliary methods.

Fig.11 shows surface settlements due to tunnel excavations for both the shield and conventional methods. It is noteworthy that the difference is very small.

Model Tests and Numerical Analysis of Sandy Ground Tunnels

In order to investigate the mechanical behavior of sandy ground tunnels and to establish a better analytical method, simple laboratory model tests were carried out and subsequently elasto-plastic finite element and joint element analyses were performed to simulate the test results[4].

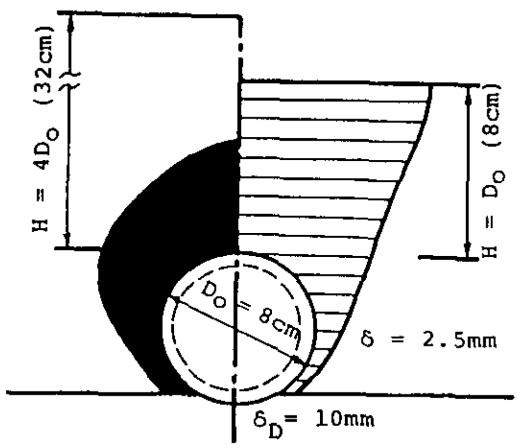


Fig.12 Zone of Displacements Larger than 2.5mm.

The model tests were carried out by using a device with a reducible diameter(its initial diameter D_0: 80mm) installed in a ground which was simulated by aluminum rods of 5cm in length and a diameter of 1.6mm and 3.0mm. Fig.12 shows the test results for two different overburden situations($H = D_0$ & $H = 4D_0$, H: depth of cover). It illustrates the zones where displacements larger than 2.5mm resulted in reducing the tunnel diameter by 10mm. It can be seen that the zone expands with the decrease of the overburden.

The left hand side of Fig.13 shows the change of vertical displacement with depth at four locations. On the right hand side the variation of the displacement isobars and the displacement vectors are given. For a small overburden

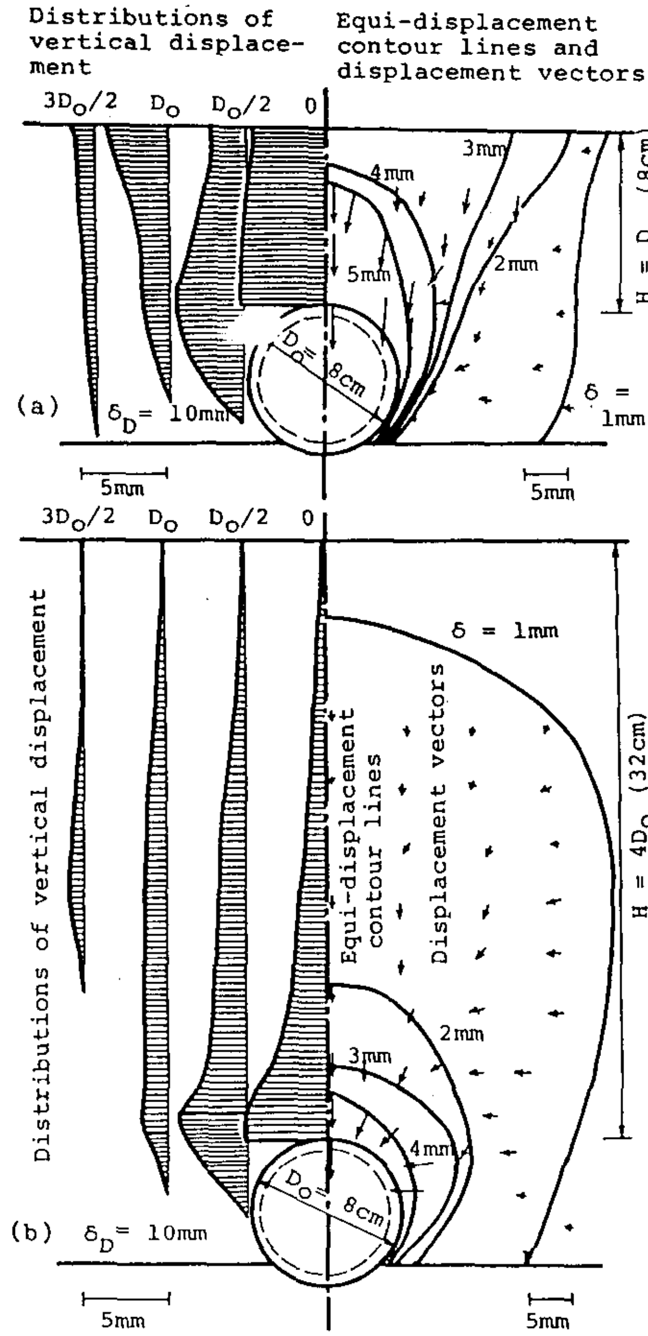


Fig.13 Experimental Results: Vertical Displacement with Depth, Variations of Displacements Isobars and Displacement Vectors (a) Shallow Tunnel (b) Deep Tunnel.

depth, the vertical displacement distribution is very uniform above the tunnel crown. In other words, the surface subsidence is almost the same as the tunnel crown settlement. On the other hand, the surface subsidence diminishes with increase of the overburden depth.

To simulate the test results, the problems were numerically analyzed by using two finite element variation: one with continuous elasto-plastic elements and another one with joint elements for the simulation of discontinuous displacement behavior. Corresponding to Fig.12(test results), Fig.14 shows the analytical results

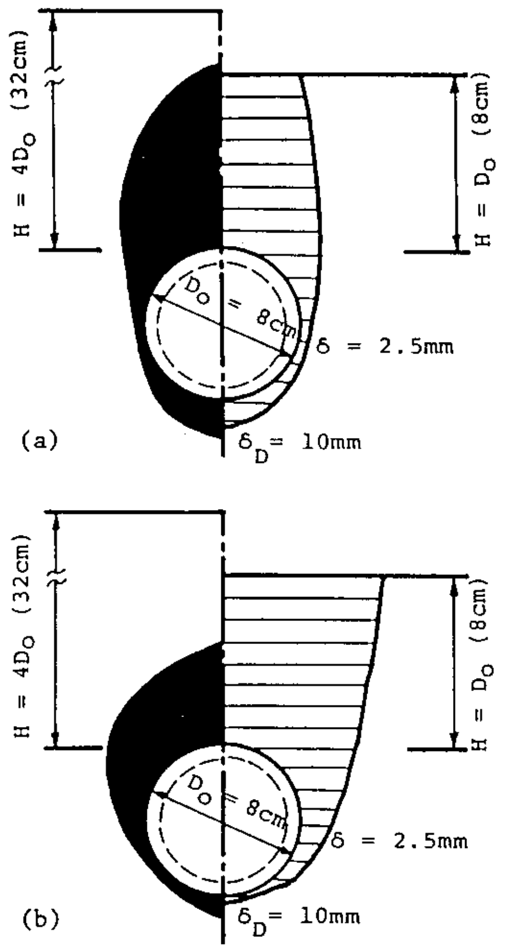


Fig.14 Calculated Results Corresponding to Fig.12, (a) Continuous Elasto-Plastic Elements, (b) Joint Elements.

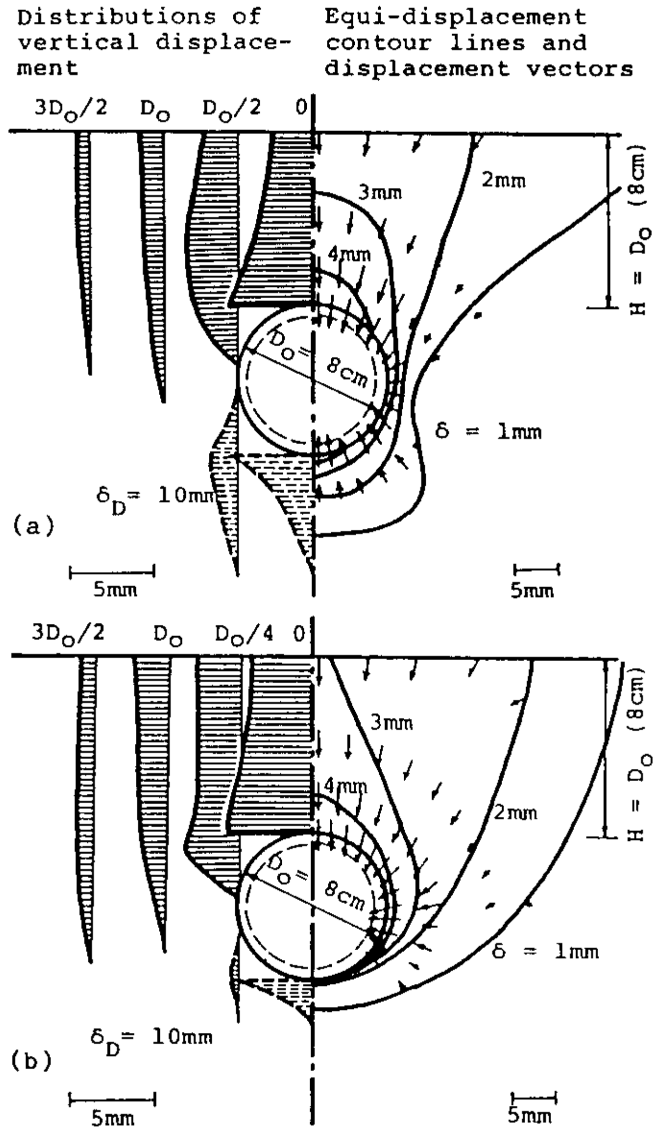


Fig.15 Calculated Results Corresponding to Fig.13 (a), (a) Continuous Elasto-Plastic Elements, (b) Joint Elements.

for zones with larger displacements(>2.5mm). Comparing the analytical results with the experimental ones, the superiority of the analysis with joint elements is obvious. For instance, the zone reduces its size with increasing overburden depth. Fig.15 gives the analytical results corresponding to the test results shown in Fig.13(a). On the basis of this comparison, the analysis with joint elements can simulate well not only the change of the vertical displacement distributions with depth, but also the shape of the displacement isobars. As a conclusion it can be stated that the analysis with joint elements is superior relative to elasto-plastic elements. This study also suggest that a tunnel construction in cohesionless ground causes discontinuous displacements.

Estimation of Design Parameters for Sandy Ground Tunneling

The determination of the soil parameters by back-analysis based on measurements made in the surrounding ground permit the fine tuning of the design parameters, thereby improving the design of subsequent stages of the construction. A numerical procedure for the back-analysis is proposed on the basis of the finite element method with joint elements and the procedure is applied to actual case studies[5].

In order to establish a numerical procedure, the hyperbolic stress-strain relationship given in Fig.16 is adopted in the direction parallel to the joint. The tangent modulus corresponding to any point on the stress-strain curve is expressed as:

$$k_s^t = \frac{k_s^i \cdot s^2}{\left(s + k_s^i |u|\right)^2} \qquad (6)$$

where k_s^t: tangent modulus of rigidity, k_s^i: initial tangent modulus of rigidity, s: shear strength, σ_n: normal stress of the joint element, and u: tangent joint displacement. On the other hand, the constitutive model for the normal direction satisfies the zero tensile strength condition and provides large stiffness in compression.

Elastic continuous-elements and joint-elements were used to model the soil, while beam-elements were applied to the primary lining.

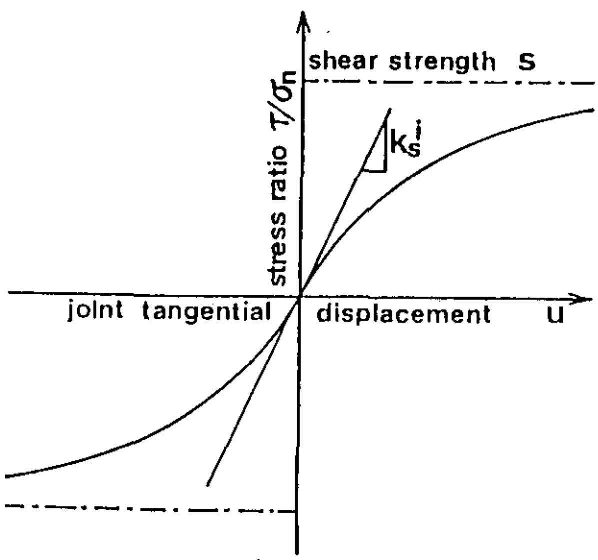


Fig.16 Stress-Displacement Relationship for Join Element.

Hence, if the material parameters for beam-elements are provided, the inverse-problem is to find the initial tangent modulus of rigidity k_s^i and the shear strength s for the joint elements and to find the Young's modulus E and the Poisson's ratio ν for the continuous elements by back-analysis from measured field displacements. The following objective function was used.

$$\text{minimize } J = \sum_{n}^{N_t} \sum_{i}^{N_d} \left(u_i^n - U_i^n \right)^2 \tag{7}$$

where J: objective function, u_i^n: calculated displacement at node i at time step n, U_i^n: measured displacement corresponding to u_i^n, N_t: number of time steps and N_d: number of measured values of displacement.

The actual example given here is a cross-section of the Kuriyama Tunnel of Hokusou line in Chiba. The tunnel was excavated in a soil deposit belonging to the Narita group. During excavation the vicinity of the tunnel was dewatered by deep wells. Thus, the effect of underground water is not taken into account for the analysis.

Fig.17 illustrates the finite element model and the location of measuring points. Joint elements are shown by the bold lines. Fig.18 shows simulation of tunnel excavation. The measured convergence of the cross-section are multiplied by 10/3 to account for the release of initial stresses in the soil mass prior to the arrival of the cutting face.

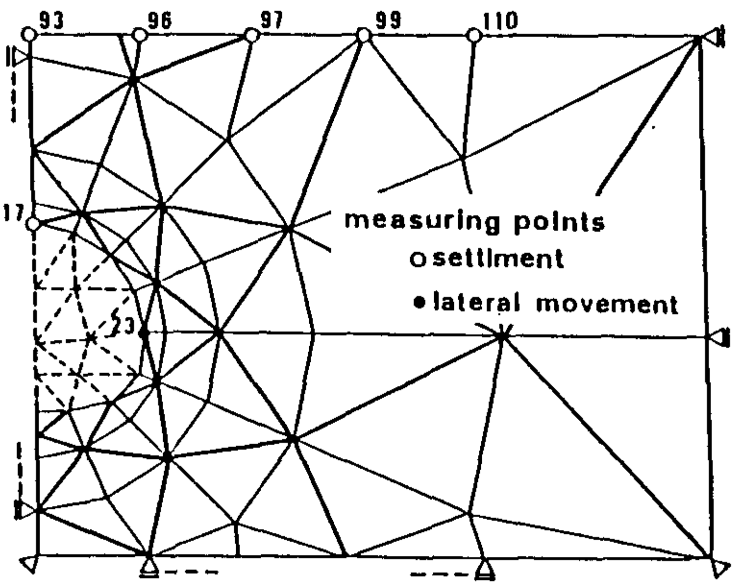


Fig.17 Finite Element Model for Case Study.

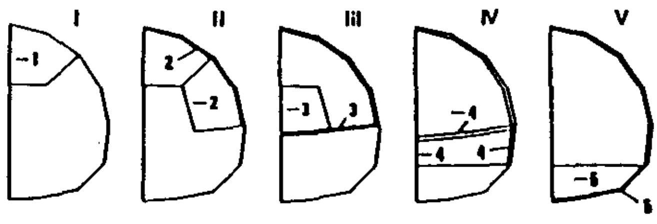


Fig.18 Simulation of Tunnel Excavation.

In the first step, the present procedure uses the data collected after full excavation of the cross-section. Fig.19 shows the iteration behavior of the procedure. Fig.19 also contains the soil parameters, which were obtained from laboratory tests. It is noteworthy that the Young's modulus estimated by back-analysis is larger than the

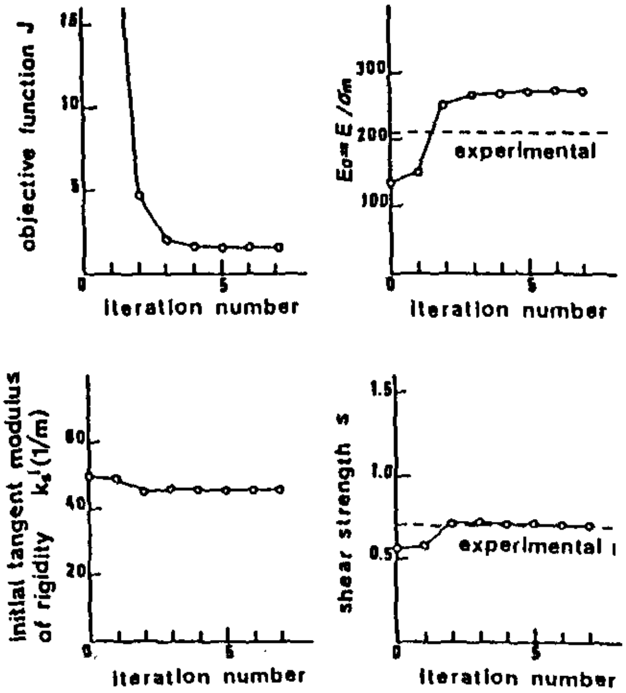


Fig.19 Iteration Behavior Using the Data Collected after Full Excavation.

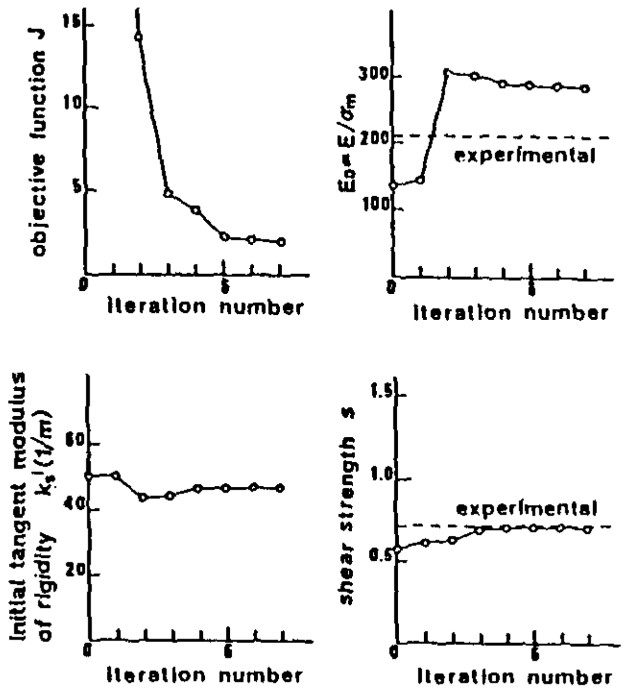


Fig.21 Iteration Behavior Using the Data Collected During the Excavation of the Top Half Portion.

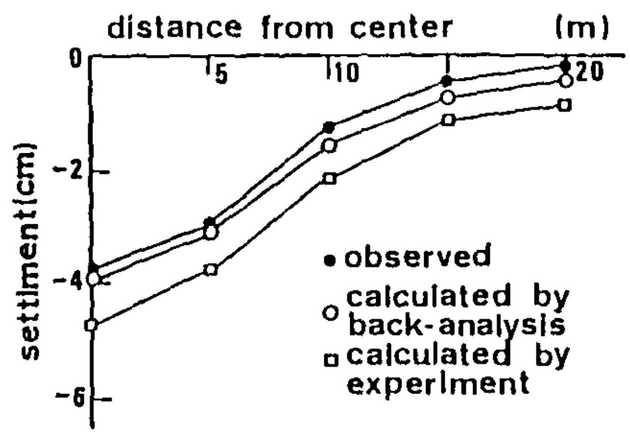


Fig.20 Calculated and Measured Surface Settlements.

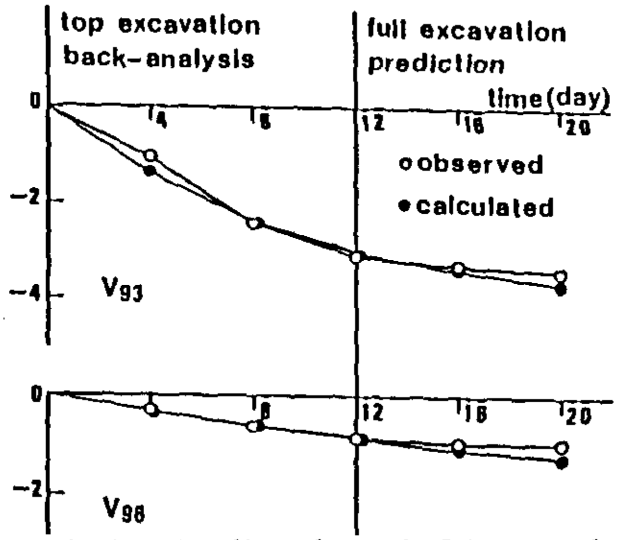


Fig.22 Predicted and Measured Surface Settlements at Two Locations(V93 and V96).

one derived from laboratory tests, although both shear strengths are in good agreement. The calculated surface settlements based on soil parameters obtained from back-analysis are shown in Fig.20, together with those measured in the field and those calculated from laboratory tests. The laboratory tests results overestimates the surface settlement to some extent.

Next, the procedure of back-analysis is applied to the data collected during the excavation of the top portion of the cross-section. Fig.21 illustrates the performance of this procedure. The projected surface settlements at two locations predicted from the back-analyzed material parameters are shown in Fig.22, together with those measured in the field. Thus, with site measurements in the early stages of an earth tunnel excavation, the proposed procedure of back-analysis enables us to permit the fine-tuning of the design parameters and to make predictions of future displacement behavior of the ground.

CONCLUSIONS

1) A reasonable back-analyzing procedure for braced excavations is proposed by using the finite element method and the efficiency of this procedure was proved by a case study.
2) As a lower margin, the conventional method is applicable to diluvial formations with uniaxial compressive strength q_u larger than 1kgf/cm^2(or modulus of deformation E_S higher than 100kgf/cm^2).
3) Tunnel construction in sandy ground causes a particular pattern of discontinuous displacements. For a small overburden depth, the magnitude of the surface settlement is the same as the convergence at the tunnel crown. Taking preventive measures to minimize the tunnel convergence is most important.
4) To find an appropriate analytical method to simulate the discontinuous displacement behavior, the problem was analyzed two ways, that is, on one hand with continuous elasto-plastic elements and on the other hand with joint elements. On the basis of comparison between analytical and test results, the analysis with join elements is superior relative to that with continuous elements in simulating the displacement behavior of the surrounding ground.
5) The determination of soil parameters with back-analysis of site measurements is very useful to improve the design of subsequent stages of the construction. A numerical procedure using joint elements for back-analysis was proposed. This procedure enables us to predict the future displacement behavior quantitatively.

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