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Strain softening analysis for improved prediction of deformational mechanism of a shallow tunnel

Jaeho LEE[1], Yasuhiro YOKOTA[1], Shinichi AKUTAGAWA[2], Takashi KITAGAWA[3], and Takeshi MATSUNAGA[4]

[1]Member of JSCE, Graduate student, Kobe University
(Rokkodai 1-1, Nada, Kobe, 657-8501, Japan)
E-mail : 006d804n@y02.kobe-u.ac.jp
[2]Member of JSCE, Associate Professor, Dept. of Architecture and Civil Engineering, Kobe University
(Rokkodai 1-1, Nada-ku, Kobe, 657-8501, Japan)
[3]Member of JSCE, Japan Railway Construction, Transport and Technology Agency, Tokyo, Japan
(Honmachi 6-50-1, Naka-ku, Yokohama, 231-8315, Japan)
[4]Member of JSCE, Pacific Consultants CO., LTD., Tokyo, Japan
(Nishi Shinjuku 2-7-1, Shinjuku-ku, 163-0730, Tokyo)

A finite element analysis procedure considering strain softening behavior is applied for identification and prediction of deformational behavior around a shallow NATM tunnel. A determination of the unknown model parameters is proposed by means of artificial neural network. The network is trained to approximate the results of FE simulations. Based on the identified material parameter, numerical results produced a strain distribution, deformational mechanism and surface settlement profile, which are compared with the results of the field measurement conducted at cross section. This paper discussed that the performance of the proposed method is demonstrated by prediction problem of actual NATM tunnel.

Key Words : *NATM tunnel, strain softening analysis, shear band, prediction, artificial neural network*

1. INTRODUCTION

Currently an increasing number of urban tunnels with small overburden are excavated according to the principle of the New Austrian Tunneling Method (NATM). In urban area, constructions of soft ground tunnels are usually important in terms of prediction and control of surface settlement and gradient[1)]. Several approaches[2)-5)] are readily used for prediction of the ground deformations associated with tunneling. In recent years, numerical methods for design purposes are often used to predict deformational behavior around tunnels. Finite element procedures have been applied not only to the ground movement prediction but also to the whole tunnel design problem, which includes simulation of the construction method, analysis of the extent and development of failed zones, design of the support system, and effects on nearby tunnels, etc. In the approach of numerical modeling, those results are strongly dependent on the construction stages modeled, the constitutive law selected, and the appropriate assessment of the corresponding soil parameters. However, it is still difficult under the present state of the art to predict deterministically the behavior of the ground and tunnel system in the planning and design state, thereby resulting in an extensive difference between the predicted values and the actual behavior after excavation. In this paper, finite element simulation has been applied to predict ground movement caused by tunneling of a shallow NATM tunnel in unconsolidated soil. The method used here incorporates reduction of shear stiffness, as well as strain softening effects of given material strength parameters[4)]. A determination of the unknown model parameters is proposed by means of artificial neural network. The network is trained to approximate the results of FE simulations. The parameters identified were then used to predict displacements at the final stage. Thus, it is stressed in this paper that the proper consideration of nonlinear nature of ground materials is necessary not only for identification of deformational mechanism which has already happened, but also for predicting what to expect with reasonably high accuracy.

2. NONLINEAR DEFORMATION IN SHALLOW TUNNELS

Deformational behavior around a shallow tunnel is often characterized by formation of shear bands developing from tunnel shoulder reaching, sometimes, to the ground surface. **Fig.1** shows a strain distribution derived from the results of displacement measurements taken from a subway tunnel in Washington D.C.[6]

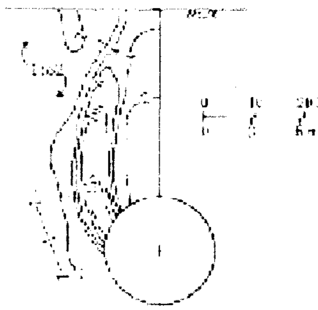


Fig.1 Strain distribution around a subway tunnel (after Hansmire and Cording, 1985[6]).

One possible explanation of this deformational behavior may be best stated with a help of an illustration given in **Fig.2**.

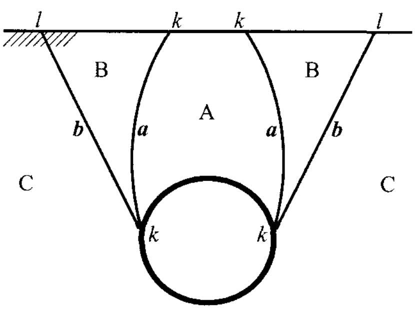


Fig.2 Typical deformational mechanism around a shallow tunnel.

Region-A, surrounded by slip plane *k-k*, is regarded as a potentially unstable zone that may displace downward at the lack of frictional support along *k-k* planes. What is separating region-A from the surrounding is shear band ***a*** formed along *k-k* line with some thickness, as region A slides downward. The adjacent region-B follows the movement of region-A, leading to the formation of another shear band ***b***. The direction of shear band ***b*** is related to $45^\circ + \phi/2$ (ϕ : friction angle) and often coincides with what is called a boundary line of zone influenced by excavation.

Regions A and B correspond to the primary and secondary zones of deformational behavior pointed out earlier by Murayama et al.[2), 3)] in the series of trap door experiments.

Confirming the presence of these zones is equivalent to acknowledging formation of shear bands ***a*** and ***b***, which may not be a desirable practice in view of minimizing deformation during construction of shallow tunnels. However, it is regarded very important that a reliable method be established in order to reveal non-linear deformational mechanism and identify the state of deformation with reference to an ultimate state, which is of current interest in the new design practice.

3. STRAIN SOFTENING ANALYSIS

In the framework of applying general numerical analysis tools, such as finite element methods, there have been series of approaches taken for simulation of tunnel excavation. Adachi et al.[4)] made use of classical slip line theory to define geometrical distribution of joint elements for modeling shallow tunnel excavation. Okuda et al.[5)] applied a back analysis procedure to identify the deformational mechanism, in which anisotropic damage parameter m was employed. Sterpi[7)] conducted strain softening analysis in which strength parameters (cohesion and friction angle) were lowered immediately after the initiation of plastic yielding. This approach was applied for the interpretation of field measurements by Gioda and Locatelli[8)] who succeeded to simulate the actual excavation procedure with accuracy. These attempts incorporate some of the key factors that must be taken into consideration for modeling shallow tunnel excavation. However, there still is shortage in modeling capability, which is expected to cope with development of shear bands, formation of primary and secondary zones, etc.

By reviewing the previous works, the authors concluded that the essential features to be taken into the numerical procedure would be reduction of shear stiffness and strength parameters after yielding (namely, strain softening)[9)]. Following is a brief summary of the procedure employed in this work. A fundamental constitutive relation between stress σ and strain ε is defined by an elasticity matrix D

$$D = \frac{E}{1-\nu-2\nu^2}\begin{bmatrix} 1-\nu & \nu & 0 \\ \nu & 1-\nu & 0 \\ 0 & 0 & m(1-\nu-2\nu^2) \end{bmatrix} \tag{1}$$

where $\sigma = D\varepsilon$ holds. E and ν stands for Young's modulus and Poisson's ratio, respectively. The anisotropy parameter m is defined as

$$m = m_e - (m_e - m_r)[1 - Exp\{-100\alpha(\gamma - \gamma_c)\}] \quad (2)$$

where m_e is the initial value of m, m_r is the residual value, α is a constant, γ is shear strain, γ_c is the shear strain at the onset of yielding.

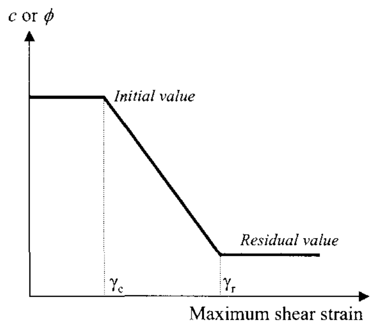


Fig.3 Reduction of strength parameters.

The constitutive relationship is defined for conjugate slip plane direction (45° $\pm\phi/2$) and transformed back to the global coordinate system. Strength parameters, namely cohesion c and friction angle ϕ are reduced from the moment of initiation of yielding to residual values, as indicated in **Fig.3**. This implies that the admissible space for stress is gradually shrunk as strain-softening process takes place. Any excess stress, which is computed on the transformed coordinate system based on slip plane direction, outside an updated failure envelop is converted into unbalanced forces that are compensated for in an iterative algorithm.

4. THIS TUNNEL FOR CASE STUDY

(1) Construction site for Rokunohe tunnel

The Rokunohe tunnel, 3810m long, is located at the northern end of the Honshu, between Hachinohe and Shin-Aomori as shown in **Fig.4**. The excavation was conducted by top heading method. Excavation of the lower section excavation followed approximately 40m behind the face of the upper section excavation. Reinforcement of supports has been put by using rockbolt, shotctete and steel support as shown in **Fig.5**. Auxiliary method is applied by face shotcrete, face bolt, deep well, well point, and so on, for face stabilization and water inflow control.

The geological profile of the ground consists of unconsolidated sand layer (Layer 3) in excess of 30m which is lying beneath two layers (Layers 1 and 2) of volcanic ash. During the tunnel construction, various measurements on tunnel and ground were carried out to confirm the stability of the tunnel and the adequateness of the excavation method. Crown, convergence, surface settlement, subsurface settlement and horizontal displacement were measured as shown in **Fig.6**.

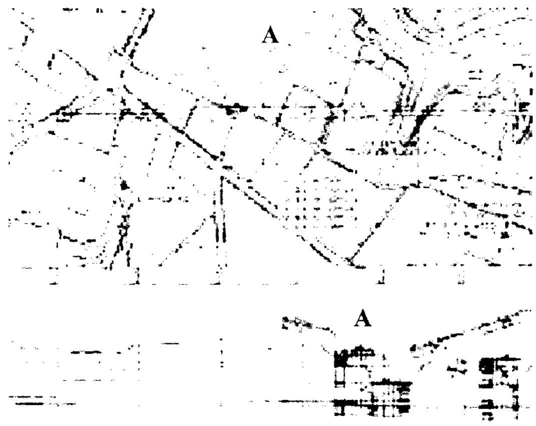


Fig.4 Plan and vertical view of the site.

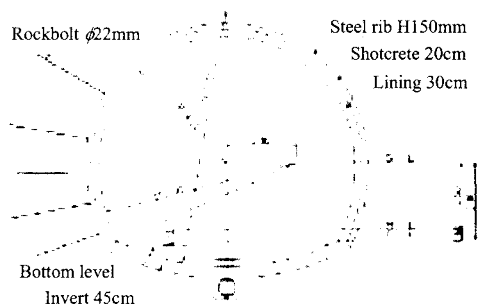


Fig.5 Tunnel cross section.

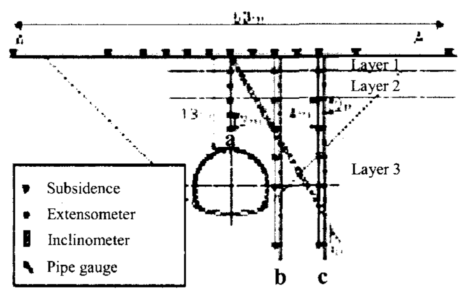


Fig.6 Field instrumentation

(2) Outline of numerical simulation

Numerical simulations were conducted for two cross sections with slightly different geometric configuration. Locations for the sections A are shown in **Fig.4**. Geometry and boundary conditions of the finite element meshes are shown in **Fig.7** for the case of Section A. The ground behavior was modeled with the strain softening model proposed in this paper. Shotcrete and steel support were modeled as elastic elements. The material properties obtained for each layer are shown in **Tab.1**. The construction sequence is to excavate the top heading (upper section) in advance followed by bench (lower section) and invert excavation. Simulation has been performed in several computational steps for excavation of the tunnel top heading.

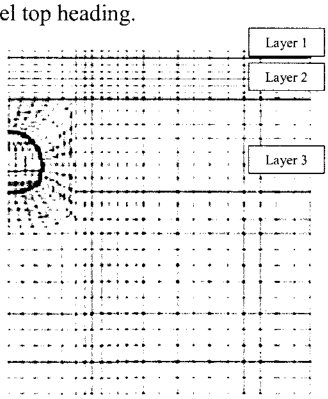


Fig.7 Finite element mesh used for simulation.

In the first step, 40% stress release ratio with excavation of the top heading (upper section) have been applied. This step relates to the timing when an upper section arrives at a tunnel face. In the second step, the support has been put in place and, at the same time, the remaining 60% of the excavation forces has been released. As for strain softening analysis, parametric study was performed in which Δ γ (increment of maximum strain during which strength drops from peak to residual value, see **Fig.3**) and the ratio of residual to original strength were varied, resulting in the total of 9 cases as shown in **Tab.2**.

Tab.1 Material properties for 3 layers.

	Layer 1 *Takadate volcanic ash layer*	**Layer 2** *Tengutai volcanic ash layer*	**Layer 3** *Noheji sandy layer*
γ(kN/m^3)	14.0	18.0	20.0
E (MPa)	5.0	5.0	80.0
ν	0.286	0.286	0.286
ϕ (degrees)	30	45	30
c (MPa)	0	0	35

Tab.2 Scheme for strain softening analyses.

		Residual strength/Original Strength, η		
		80%	60%	40%
Δγ	0.04	Case 1	Case 4	Case 7
	0.02	Case 2	Case 5	Case 8
	0.01	Case 3	Case 6	Case 9

(3) Subsidence profile and Pipe gauge

The surface settlements measured at various stages of the advancing of excavation are shown in **Fig.8**.

The reading obtained from the pipe gauge embedded in section A showed an interesting behavior shown in **Fig.9**. Though the magnitude of deformation is small, the bending mode of the pipe which was installed at an inclined position suggests that the shear band formation was already in progress to some extent. This is not an easy phenomenon to identify from standard pattern of displacement monitoring, but definitely a very important behavior not to be missed.

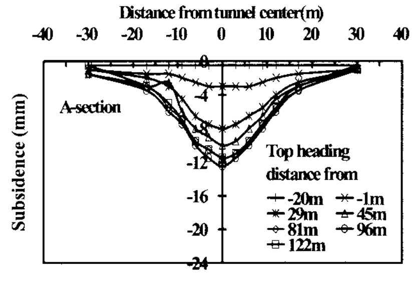


Fig.8 Subsidence

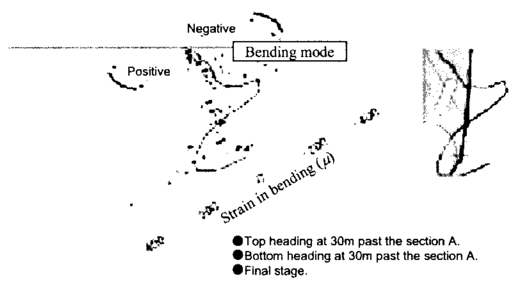


Fig.9 Bending of pipe gauge embedded in section A.

5. IDENTIFACATION AND PREDICTION

(1) Parameter identification

As pointed out earlier, it is ideal if deformation at final stage could be predicted before construction using lab test results and etc. However, it is well known that direct use of lab test results would not lead to realistic estimate of the final state of deformation around tunnels. Generally, back analysis is used for parameter identification. The use of field data from in situ as input data for back analysis was proposed in References[10].

Based on these measurements, values of unknown model parameters characterizing mechnanical properties of the soil/rock mass can be determined. The obtained model parameters are optimal in the sense that the best possible agreement between respective results and in situ measurements will be achieved, if these parameters are used as input for the numerical analysis. Hence, based on the identified model parameters, an appropriate numerical model provides a good numerical approximation of the monitored structural behavior. A determination of the unknown model parameters is proposed by means of The Back Propagation Neural Network[11]. The network is trained to approximate the results of FE simulations. Its basic principle is in learning from known examples: it is a procedure of iterative computation and comparison of computed results with the correct input data. Before the parameter identification, a teaching process of the neural network mesh had taken place on 54 learning cases from numerical study. The back-propagation network consists of an input line, one or more hidden line, and an outline. The back-propagation algorithm is repeated until the difference between the desired and the computed result converges to a minimum. The hidden neurons are linked with input and output neurons. The construction is presented in **Fig. 10**. From neural network, four parameters, E, K_0, η and $\Delta\gamma$ are assumed to be unknown. They are collected in the data space p, reading

$P=[E_{layer3},\ K_0,\ \eta_{Layer3},\ \Delta\gamma_{Layer3}]$.

The intervals for the unknown parameters are chosen as

$E_{layer3}, \in \{80MPa, 300Mpa\}$

$K_0 \in \{0.4, 0.97\}$

$\eta_{Layer3} \in \{20\%, 80\%\}$

$\Delta\gamma_{Layer3} \in \{0.01, 0.04\}$

These results were obtained by computations with the Neural Back Propagation Mesh, with two hidden layers, having in the first layer 32 hidden neurons and 32 in the second one.

Parameter identification is assumed to have converged if computation and comparison of computed results is smaller than or equal to 0.1% or after 300000 iteration steps. Based on the settings, optimal parameter set were computed, giving p

P=[265MPa, 0.73, 30%, 0.037].

The correlation coefficient between the measured and calculated value was 0.95.

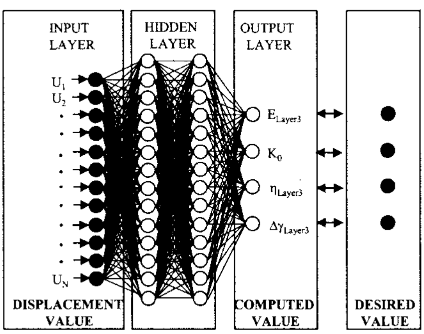


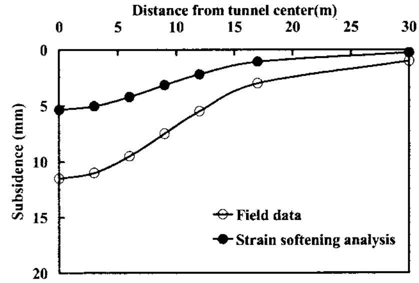
Fig.10 Neural network structure

(2) Prediction of surface settlement and displacement around tunnel

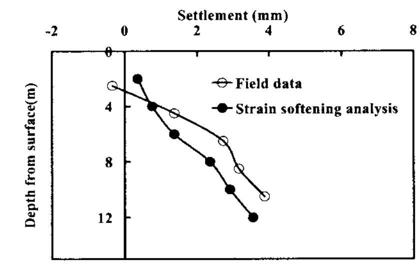
Based on the identified material parameter, numerical results produced a strain distribution, deformational mechanism and surface settlement profile, which are compared with the results of the field measurement conducted at cross section.

The vertical displacements corresponding to the 'optimal' model parameters of the Neural network results at final stage of cross-section A are shown **Fig.11** and compared with the in situ measurements. The diagrams (a)-(c) represent the vertical measurements along two vertical lines and the surface settlements, respectively.

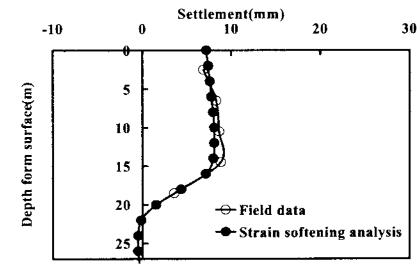
As for the results of strain softening analysis, the one which gave the closest results to the measurement is shown for cross sections A.



(a)Subsidence



(b) Extensometer above the tunnel

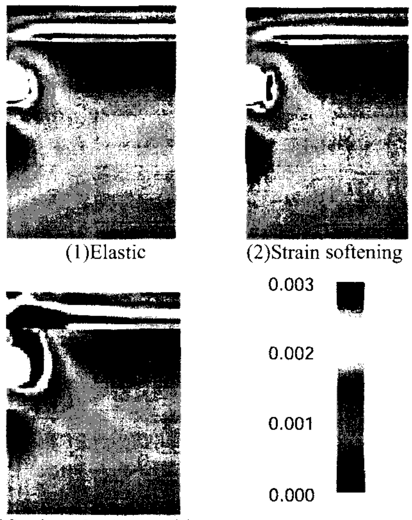


(c) Extensometer at near side

Fig.11 Displacement at the final stage

(2) Distribution of maximum shear strain

Finally, **Fig.12** shows the maximum shear strain distribution at the final stage of analysis for sections A. Elastic and strain softening models resulted in similar images since the magnitude of displacement here was constrained to fairly low level. The case which showed the best results in comparison with the measurement, namely the result from the softening analysis, shows the development of shear band from tunnel shoulder. The band is believed to be of a fair size, although it has not reached the surface of the ground. If no-support, this development of the shear band is regarded as shown in **Fig.12(3)** and the cause of large displacement occurred for this section.



(3)Strain softening with no-support

Fig.12 Maximum shear strain distribution at the final stage

6. CONCLUSION

A finite element analysis procedure considering strain softening behavior is applied for identification and prediction of deformational behavior around a shallow NATM tunnel. A determination of the unknown model parameters is proposed by means of artificial neural network. An objective was to point out the importance of modeling a non-linear nature of the deformational mechanism for obtaining a better understanding of design load on tunnel linings and its relation to kinematics of the surrounding ground. The proposed approach produced strain distribution, deformational mechanism, surface settlement profile, which were in agreement with the results of the field measurement results by case study.

Furthermore, the method offers a practical way for predicting final displacement of tunnel at earlier stages of construction, enabling rational safety management scheme to be employed. This makes a good starting point for optimizing ground support for reducing surface settlement, considering a particular nature of the deformational mechanism of shallow tunnels.

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