Numerical simulation of a strain softening behavior of a shallow tunnel for a bullet train

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ABSTRACT: A new finite element analysis procedure is proposed for simulation of deformational behavior around a shallow tunnel. The method incorporates reduction of shear stiffness, as well as strain softening effects of a given material. The proposed approach produced a strain distribution, deformational mechanism and surface settlement profile, which are in good agreement with the results of the field measurement conducted at two cross sections with different deformational characteristics.

1 INTRODUCTION

Deformational mechanism of an urban NATM tunnel at shallow depth is often characterized by a set of unique kinematic movements in subsidence profile and crown settlement, etc. Computational tools available, so far, for design purposes are used to predict deformational behavior around tunnel, interaction between support structures and ground. However, results from most attempts might not be satisfactory in that those unique deformational behaviors of non-linear nature are not properly represented.

This paper proposes an improved computational scheme by which characteristic deformational behavior of a shallow tunnel is properly modeled. The new computational procedure, incorporating strain-induced anisotropy and strain softening, was applied to simulate Shinkansen tunnel excavations for two cross sections with varying soil properties. The results obtained showed that the new computational scheme could represent (1) surface subsidence and crown settlement, (2) shear bands, if they do exist, developing from tunnel shoulder, which were in good agreement with field measurement results.

2 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. Figure 1 shows a strain distribution derived from the results of displacement measurements taken from a subway tunnel in Washington D.C. (Hansmire and Cording, 1985).

One possible explanation of this deformational behavior may be best stated with a help of an illustration given in Figure 2. Region-A, surrounded by slip plane k-k, is regarded as a potentially unstable zone which 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 $45° + \phi/2$ (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

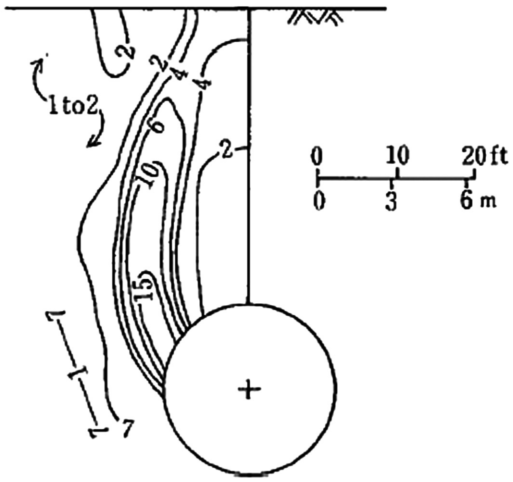

Figure 1. Strain distribution around a subway tunnel (after Hansmire and Cording, 1985).

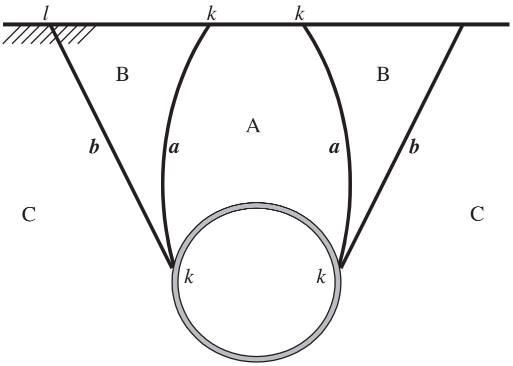

Figure 2. Typical deformational mechanism around a shallow tunnel.

pointed out earlier by Murayama et al. (1969, 1971) 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 NUMERICAL PROCEDURE

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. (1985) made use of classical slip line theory to define geometrical distribution of joint elements for modeling shallow tunnel excavation. Okuda et al. (1999) applied a back analysis procedure to identify the deformational mechanism, in which anisotropic damage parameter m was employed. A strain softening analysis was conducted by Sterpi (1999) 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 (1999) 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). Following is a brief summary of the procedure employed in this work (Matsumoto, 2000). A fundamental constitutive relation between stress σ and strain ε is defined by an elasticity matrix D

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

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

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

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.

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 Figure 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.

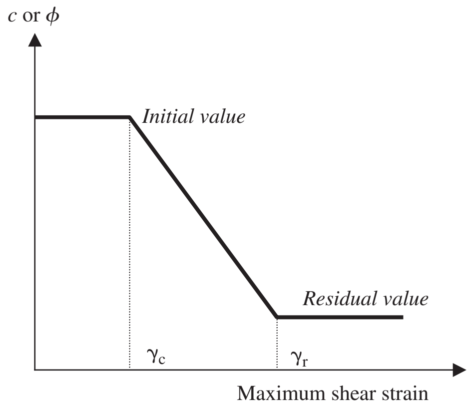

Figure 3. Reduction of strength parameters.

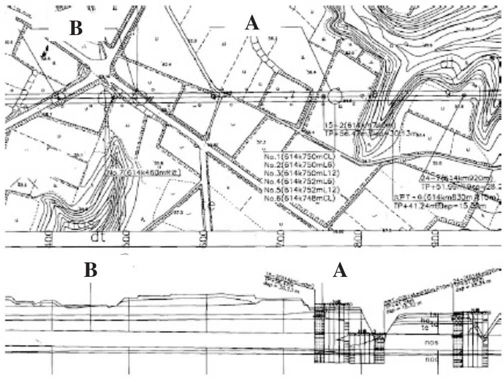

Figure 4. Plan and vertical view of the site.

4 APPLICATION EXAMPLE

4.1 *Construction site for Rokunohe tunnel*

The Rokunohe tunnel, 3810 m long, is located at the northern end of the Honshu, between Hachinohe and Shin-Aomori as shown in Figure 4. The excavation was conducted by top heading method. Excavation of the lower section excavation followed approximately 40 m behind the face of the upper section excavation. Reinforcement of supports has been put by using rockbolt, shotcrete and steel support as shown in Figure 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 30 m which is lying beneath two layers (Layers 1 and 2) of volcanic ash. The material properties obtained for each layer are shown in Table 1.

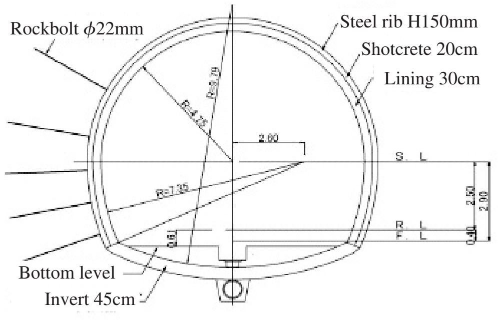

Figure 5. Tunnel cross section.

Table 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

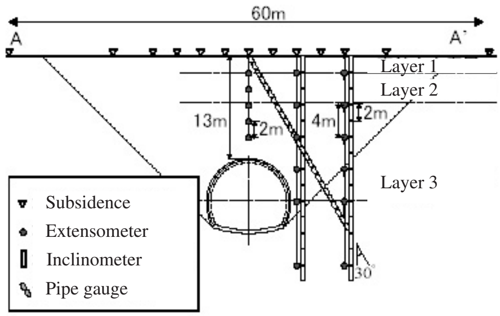

Figure 6. Field instrumentation.

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 Figure 6.

4.2 *Numerical simulation*

Numerical simulations were conducted for two cross sections with slightly different geometric configuration. Locations for the two sections A and B are shown in Figure 5. Geometry and boundary conditions of the finite element meshes are shown in Figure 7 for the case of Section A. The ground behavior was modeled with three different constitutive laws; namely (1) an elastic model, (2) elastic-plastic material model with a Mohr-Coulomb failure criterion and (3) the strain softening model proposed in this paper. Shotcrete and steel support were modeled as elastic elements. 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. 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 $\Delta\gamma$ (increment of maximum strain during which strength drops from peak to residual value, see Figure 3) and the ratio of residual to original strength were varied, resulting in the total of 9 cases as shown in Table 2.

4.3 *Resutls of numerical simulation and discussion*

Figure 8 shows surface subsidence from 3 different material models and the measurement. As for the results of strain softening analysis, the one which gave the closest results to the measurement is shown for both cross sections A and B. For section A, where the maximum subsidence was around 10 mm, the results from different models show insignificant differences. On the contrary, those for section B, where the displacement in excess of 50 mm was measured, the superiority of the softening model is seen as compared to elastic or elasto-plastic analysis. The similar trend is easily recognized in Figure 9, where surface gradient profiles are shown.

A clear advantage of the strain softening analysis is seen here for section B, where the shear band development might have occurred to produce this particular profile of surface gradient.

Table 2. Scheme for strain softening analyses.

		Residual strength/Original strength		
		80%	60%	40%
$\Delta\gamma$	0.04	Case 1	Case 4	Case 7
	0.02	Case 2	Case 5	Case 8
	0.01	Case 3	Case 6	Case 9

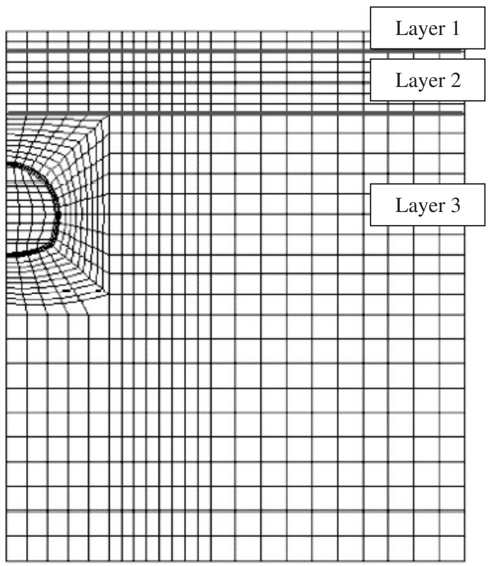

Figure 7. Finite element mesh used for simulation.

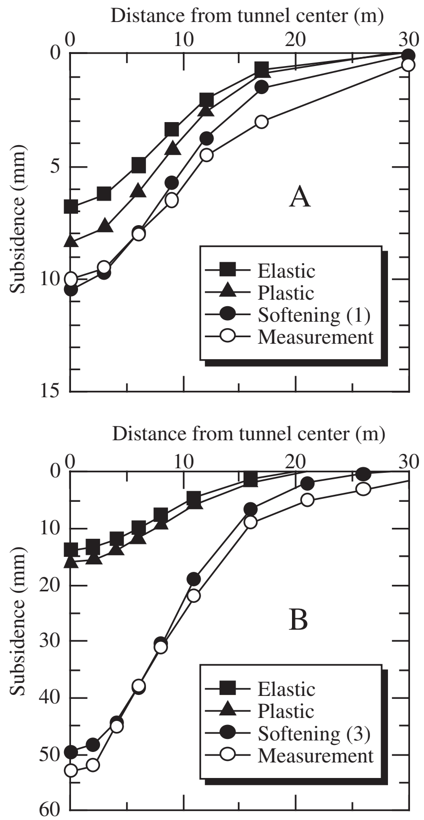

Figure 8. Surface subsidence.

Finally, Figure 10 shows the maximum shear strain distribution at the final stage of analysis for sections A and B. It is seen for section A, all material models resulted in similar images since the magnitude of displacement here was constrained to fairly low level. However, for section B, 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. However, this development of the shear band is regarded as the cause of large displacement that occurred for this section.

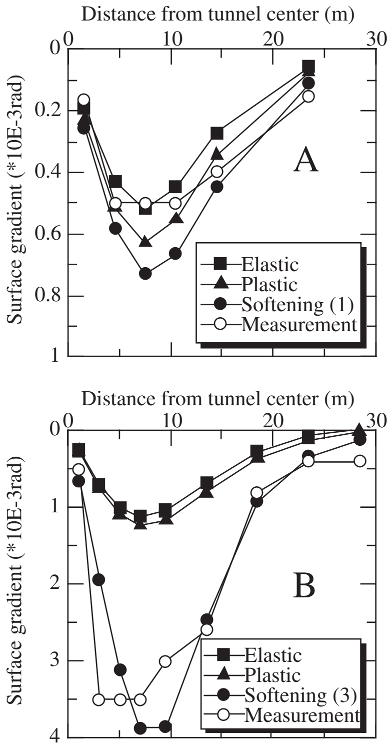

Figure 9. Surface gradient profiles.

5 CONCLUSION

A non-linear finite element analysis procedure was proposed for modeling a deformational behavior, which is unique to tunnels with shallow depth. 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 results obtained showed that modeling of a ground behavior, which is essentially of non-linear nature, by an elastic or elastic-perfectly plastic approach, leads to incorrect understanding of the deformational mechanism. The proposed approach produced strain distribution, deformational mechanism, surface settlement profile, which were in good agreement with the results of the field measurement results for two cross sections that showed different deformational behaviors.

Use of the proposed approach would enable a better understanding of deformational characteristics of the ground medium not only in identifying local plastic zones, but also in revealing kinematics of movement of blocks formed between slip planes, or shear bands.

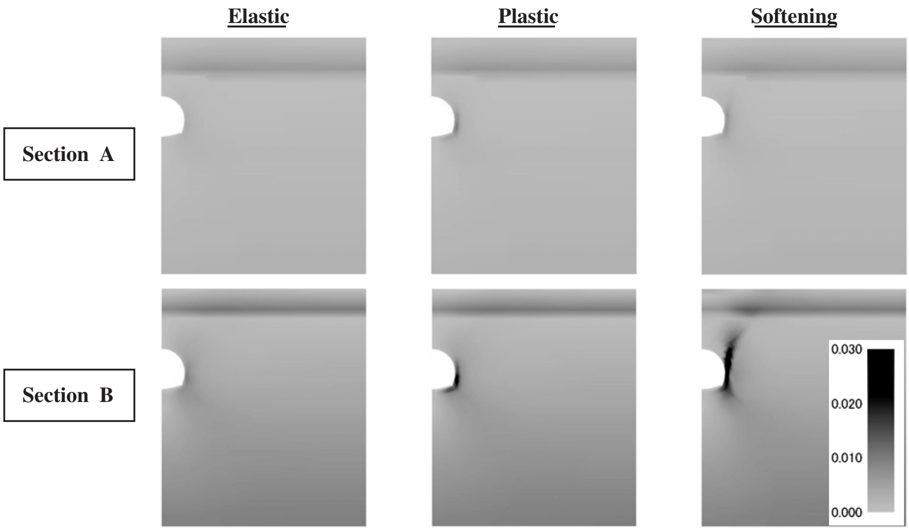

Figure 10. Maximum shear strain distributions.

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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