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The Squeezing Potential of Rock Around Tunnels: Theory and Prediction with Examples Taken from Japan

By

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Summary

Large deformations of surrounding media around tunnels are often encountered during excavations in rocks with squeezing characteristics. These deformations may sometimes continue for a long period of time. Predictions of deformations of tunnels in such grounds are urgently needed, not because of stability concerns, but also of their sevicability. In the present study, the squeezing phenomenon of rock around tunnels and its mechanism and associated factors are first clarified by carefully studying failures of tunnels, and a survey of tunnels in squeezing rocks in Japan is presented and its results are summarised. Then, a practical method is proposed to predict the squeezing potential and deformation of tunnels in squeezing rock and this method has been applied to actual tunnelling projects, where squeezing problems were encountered, to check its applicability and validity. Finally, an extension of this method to the time-dependent behaviour of squeezing rocks is given and an application of this method to an actual tunnel is presented.

1. Introduction

There is a recent general interest in tunnels which underwent large deformation. The cause of large deformation of tunnels is said to be due to the yielding of intact rock under a redistributed state of stress following excavation, which exceeds its strength. If this deformation takes place instantaneously, it is called rock-bursting. On the other hand, if the deformation takes place gradually, it is termed as squeezing.

Muirwood (1972) initially proposed the competency factor which is defined as the ratio of uniaxial strength of rock to overburden stress to assess the stability of tunnels. This parameter was later used by Nakano (1979) to recognize the squeezing potential of soft-rock tunnelling in Japan. Saari (1982) suggested the use of intensity of the tangential strain of tunnels as a parameter to assess the degree of squeezing of the rock and he suggested a threshold value of 1% for the recognition of squeezing. Saari conceived the squeezing phenomenon as an elastic-visco-plastic behaviour of rock and he did some numerical analyses and proposed closed form solutions for some special cases. However, his model does not consider the deterioration of the strength of the medium in relation to straining. Tanimoto (1984) conceived the squeezing phenomenon as an elasto-plastic behaviour of the surrounding rock and he proposed an elasto-plastic solution with a strain-softening constitutive law to estimate the degree of straining of the rock around the tunnel. He suggested that squeezing would occur when the rock is strained to its residual plastic state (flow state). However, such a suggestion corresponds to the threshold value for heavy squeezing. It should be also noted that the squeezing is initiated long before the flow of rock occurs. Therefore, better models are necessary to predict the squeezing potential of rocks and its degree together with a physical interpretation of the state of squeezing rocks around the tunnels. In the first part of this paper, the squeezing phenomenon of rock around tunnels and its mechanism and associated factors are clarified by carefully studying observed failures in-situ. Then, an extensive survey of tunnels in squeezing rocks in Japan is presented and the results of this survey is summarised.

In the second part of the paper, a practical method is proposed to predict the squeezing potential and deformation of tunnels in squeezing rock. Then, the method is applied to actual tunnelling projects, where squeezing problems were encountered, to check its applicability and validity. Several applications of the method to predict the squeezing potential and deformation of the rock around actual tunnels are given and compared with actual observations.

In the third part of this paper, an extension of this method to the time-dependent domain of rocks is given and an application of this method to an actual tunnel is presented.

2. Descriptions of Squeezing Phenomenon

Although the mechanism of squeezing failure of tunnels has been poorly understood, it is generally envisaged as the reduction of the cross-section of a tunnel due to large deformation of the surrounding medium. When this deformation is constrained by support members, it results in the failure of the members if their strength is insufficient.

The first scientific descriptions for squeezing rocks and swelling rocks were given by Terzaghi (1946) as follows:

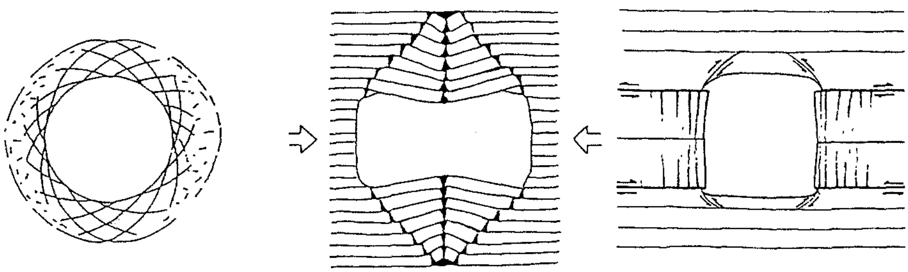
- *Squeezing rock* advances into the tunnel without perceptible volume increase. A prerequisite for squeeze is a high percentage of microscopic and sub-microscopic particles of micaceous minerals or of clay minerals with a low swelling capacity.
- *Swelling rock* advances into the tunnel chiefly on account of expansion. The capacity to swell seems to be limited to those rocks which contain clay minerals such as montmorillonite, with a high swelling capacity.

As noted from these descriptions. Terzaghi (1946) made a clear distinction between squeezing and swelling phenomena. Nevertheless, since swelling and squeezing phenomena were generally observed to be occurring together in many tunnelling projects, the term of expansive ground was adopted for the description of large deformation of the surrounding ground around tunnels (i.e. in Japan). Considering the experience of Terzaghi and others, this phenomenon is observed at relatively shallow depths in weak rocks such as phylitte, mudstone, siltstone, salt, potash or weathered and/or sheared metamorphic and igneous rocks. A more general description for squeezing and swelling phenomena from both phenomenological and mechanical points of view are necessary and an attempt for such a general description will be made herein.

2.1 Phenomenological Description

Since squeezing is regarded as a large inward closure of tunnels, no distinction is made on the nature of the motion. However, the squeezing type closure may, phenomenologically, involve three possible forms of failure of the surrounding medium (Aydan, 1989; Aydan et al., 1992, 1993b):

a) Complete shear failure: This involves the complete process of shearing of the medium in comparison with the rock-bursting, in which the initiation by shearing process is followed by splitting and sudden detachment of the surrounding rock as shown in Fig. 1a.
b) Buckling failure: This type of failure is generally observed in metamorphic rocks (i.e. phylitte, mica-shists) or thinly bedded ductile sedimentary rocks (i.e. mudstone, shale, siltstone, sandstone, evaporitic rocks) as illustrated in Fig. 1b.
c) Shearing and sliding failure: It is observed in relatively thickly bedded sedimentary rocks and it involves sliding along bedding planes and shearing of intact rock as illustrated in Fig. 1c.



a Complete shear failure, **b** buckling failure, **c** tensile splitting shearing and sliding

Fig. 1. Classification of failure forms of tunnels in squeezing rocks

2.2 Mechanical Description

In comparison with the instantaneous deformational behaviour of rock in the event of rock-bursting, the deformational behaviour of the rock in the event of squeezing is much slower.

The squeezing phenomenon may be mechanically treated as an elasto-visco-plastic behaviour of the medium under the existing stress state (Akagi et al., 1984; Gioda, 1982; Saari, 1982; Aydan et al., 1995). In other works, it can only occur when the rock is yielded by the redistributed state of stress following the excavation of the tunnel. It is a physical process and involves the dilatant behaviour of rocks.

2.3 Swelling Phenomenon and its Difference from the Squeezing Phenomenon

The swelling phenomenon, on the other hand, is a chemical process involved with the exchange of ions between some minerals and water. Therefore, it is a coupled process involving ground water seepage and diffusion and stress changes. If rock behaves elastically and the swelling is treated as a purely volumetric phenomenon, the surrounding rock first shrinks during excavation due to water loss, and then tends to swell to its original state following the construction of concrete lining (Aydan et al., 1993a, 1994). Unless rock is yielded and/or there is a large variation of volumetric stress, swelling is unlikely to become a problem. On the other hand, if the rock yields, the swelling may result in additional deformation of the ground, as phenomenologically discussed by Nakano (1979).

3. A Survey of Tunnels in Squeezing Rocks in Japan

Almost one third of Japan is covered with tertiary sedimentary rocks and volcanic origin and many highways and railways have been already constructed or are planned for construction in such rocks. As a result, the number of squeezing tunnels has been increasing and it presents an urgent necessity to predict the squeezing potential of rocks and the selection of appropriate means of reinforcement and excavation techniques in such rocks.

In Japan, when tunnels exhibit large deformation during excavation, rocks have been generally termed as expansive rocks. The expansion of rocks was sometime ago considered to be purely due to swelling of minerals contained in rocks because of their chemical reactions with water. As long as the rock is not fully saturated and yielded, it is deemed that large deformation of the surrounding medium could not be solely caused by swelling minerals as discussed in the previous section. It has recently been recognised that the cause of expansiveness was mainly due to the squeezing of the surrounding media. Therefore, most of the tunnels experiencing large deformation in Japan could be classified as tunnels in squeezing rocks.

The authors have made a general survey of reported cases to identify the factors involved in large deformation of tunnels. A list of surveyed tunnels and their main

Table 1. Surveyed tunnels in squeezing rocks in Japan

Tunnel name	Type	Length (m)	Maximum overburden (m)	Rock type
Enrei	Railway	5994	110	Mudstone
Shirasaka-1	Railway	1296	150	Mudstone
Shirasaka-2	Railway	1765	150	Mudstone
Akakura	Railway	4220	380	Siltstone, Mudstone
Mikai	Railway	1000	60	Mudstone
Ibikijo	Railway	11350	180	Mudstone
Siekan	Railway	25000	270	Andesite, Tuff, Mudstone
Nabetachiyama	Railway	9117	280	Mudstone
Nakayama	Railway	14830	400	Tuff
Orizume	Roadway	2300	200	Tuff
Fujishiro	Roadway	1823	260	Mudstone
Inari	Roadway	1441	140	Serpantin, Shale
Komadome	Roadway	2000	300	Tuff
Shinfuku	Roadway	2400	200	Mudstone, Tuff
Myojin	Roadway	3700	250	Tuff, Shale
Eno	Roadway	955	170	Andesite
Kofuchi	Roadway	4555	280	Tuff, Serpantin, Slate, Granite
Enasan	Roadway	8489	1000	Diorite
Nousie	Roadway	2992	79	Tuff, Mudstone
Mineoka	Roadway	735	90	Shale, Tuff
Nigamine II	Roadway	3831	400	Micaschist

characteristics are given in Table 1. The main items of the survey were selected to be:

a) Physical and mechanical properties of surrounding media,
b) geology, initial stress state, overburden, tunnel geometry,
c) seepage rate, excavation advance rate, support systems,
d) elapsed time for the expansion of the surrounding medium and fracturing and yielding of support members.

Item (a) is briefly presented in the next section. Data associated with main factors of squeezing have been plotted in Fig. 2. The terms of "squeezing" and "non-squeezing" in the figure are the descriptions of the authors of respective papers. On the basis of our survey, it is distinguished that the following factors are particularly involved with large deformations of the surrounding medium:

1) Competency factor, which is defined as the ratio of the uniaxial strength σ_c to the overburden pressure γH (Muirwood, 1972; Nakano, 1979), should be generally less than 2, except in the event of buckling failure, which may take place at higher competency factors.
2) The tangential strain of the tunnel wall, which is defined as $\epsilon_\theta = u/R$, should be more than 1%.
3) Although the water content is not directly involved, the water content of squeezing rocks was more than 25%. It is believed that the porosity of rocks is a much more important parameter than their water content regarding the

squeezing potential of rocks. As the porosity increases, rocks become weaker in their mechanical resistance.

4) The geology of tunnels generally consisted of layered sedimentary rocks or fracture zones which generally contained clay minerals with swelling characteristics.
5) The observed forms of failure during squeezing phenomenon were those as described in Section 2.
6) The elapsed time for the recognition of the squeezing phenomenon varies with the competency factor of the rock. The smaller the ratio is, the quicker the squeezing failure.

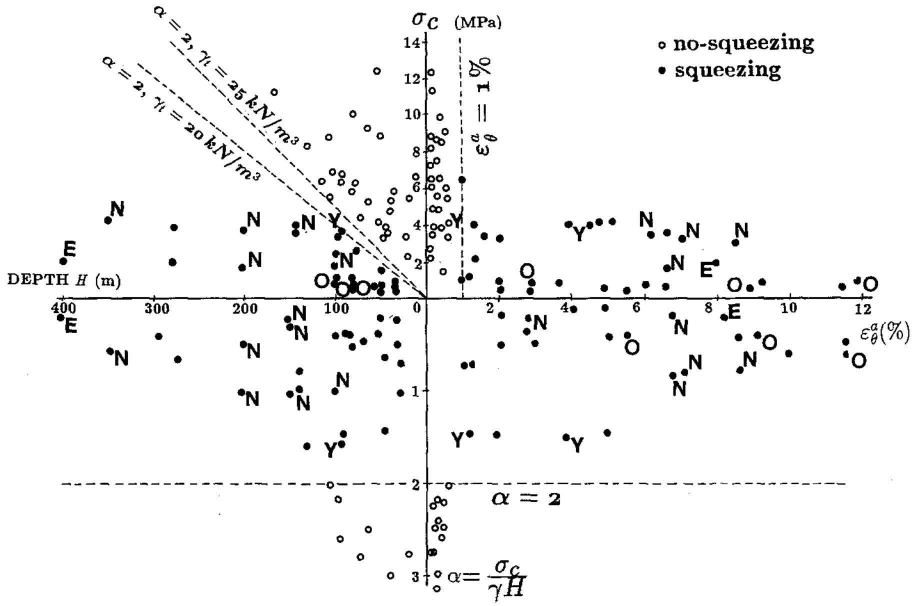


Fig. 2. A plot of data on surveyed tunnels

4. Mechanical and Physical Properties of Squeezing Rocks

4.1 Short Term Properties

From an extensive survey of data, we developed a data-base, called SQRPROP and plotted various physical and mechanical properties of rocks as a function of the uniaxial strength of rocks and established relationships between these properties and the uniaxial strength of squeezing rocks. The geometric conceptional models for deformational and strength properties of squeezing rocks are shown in Fig. 3 and Fig. 4. The relationships between physical and mechanical properties and uniaxial strength of squeezing rocks are as follow:

1) Physical properties: unit weight γ (kN/m^3), porosity n (%), elastic wave velocity V_p (km/s)

$$\gamma = 10(1 + 0.8\sigma_c^{0.15}), \quad n = 60e^{-1\sigma_c}, \quad V_p = 1.4 + 0.2\sigma_c^{0.7}. \tag{1}$$

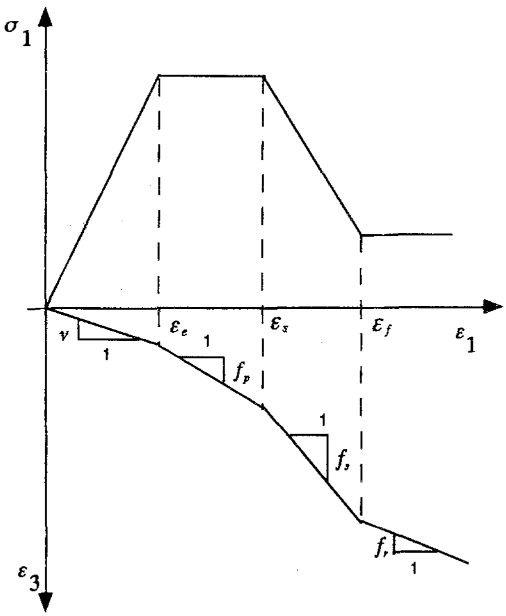


Fig. 3. Idealised stress-strain relation for squeezing rocks

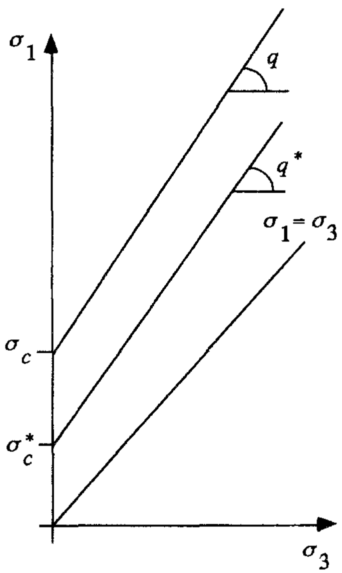


Fig. 4. Idealised Mohr-Coulomb yield criterion for squeezing rocks

2) Elastic constants: elastic modulus E (MPa), Poisson's ratio v.

$$E = 80\sigma_c^{1.4}, \quad v = 0.25(1 + e^{-0.2\sigma_c}). \tag{2}$$

3) Plastic constraints: peak friction angle ϕ_p (°), residual uniaxial strength

σ_c^r (MPa), residual friction angle ϕ_r (°), peak plastic Poisson's ratio f_p, softening plastic Poisson's ratio f_s, residual plastic Poisson's ratio f_r,

$$\phi_p = 20\sigma_c^{0.25}, \quad \frac{\sigma_c^r}{\sigma_c} = e^{-0.3\sigma_c}, \quad \frac{\phi_r}{\phi_p} = 1.3 - 0.3e^{-0.2\sigma_c}, \tag{3}$$

$$f_p = 0.5 + 0.25\sigma_c^{0.8}, \quad f_s = 1.0 + 0.25\sigma_c^{0.7}, \quad f_r = 0.5 + 0.25\sigma_c^{0.4}. \tag{4}$$

It should be noted the unit of uniaxial strength is MPa in the above relations.

4.2 Long Term Properties

It is well known that deformation and strength characteristics of rocks depend upon the stress rate or strain rate used in tests (Bieniawski, 1967; Lama and Vutukuri, 1978; Aydan et al., 1994). Figure 5 shows uniaxial stress-strain curves for a mudstone tested using strain rates of 0.1%/min, 0.05%/min and 0.001%/min. From this figure, deformation modulus and peak strength of rock as a function of strain rate are obtained and plotted in Fig. 6a and Fig. 6b. As seen from these figures, the deformation and strength characteristics of rocks depend on not only strain but also strain rate.

Creep tests and relaxation tests are also commonly used to determine the time-dependent characteristics of rocks. There is a common conception that is, the creep of rocks does not occur unless the applied stress is greater than a threshold stress, value which is called the *creep threshold* (Ladanyi, 1993). This threshold stress level is generally related to the stress level at which fractures are initiated.

The so-called *transient creep* is likely to be as a result of actual visco-elastic behaviour of rock. The *secondary creep*, on the other hand, is due to the stable crack propagation, and the *tertiary creep* is due to the unstable crack propagation (Aydan et al., 1995). Therefore, the secondary and tertiary creep is a visco-plastic phenomenon rather than visco-elastic phenomenon as it involves energy

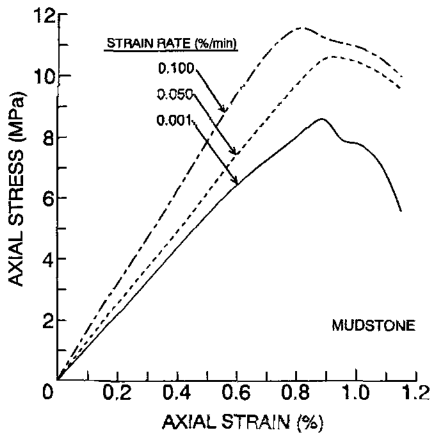


Fig. 5. Effect of strain rate on stress-strain response of a mudstone

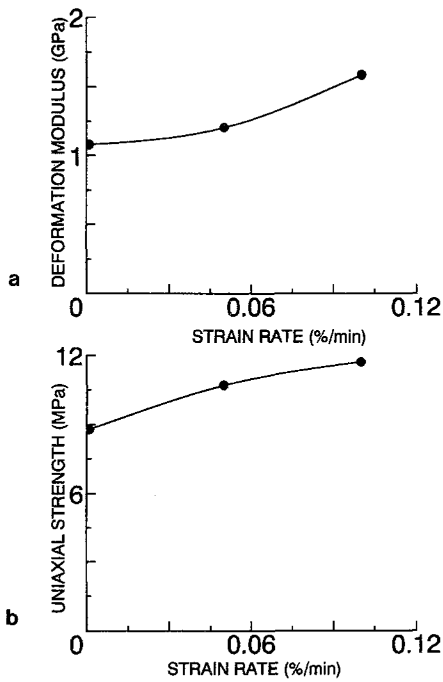


Fig. 6. Effect of strain rate on deformation modulus and strength of a mudstone

dissipation by fracturing. Figure 7 shows the normalised uniaxial creep stress by the short term uniaxial strength for mudstone and Oya tuff as a function of the failure time (time elapsed until the failure). The degradation of the uniaxial strength of rock may be represented by the following function

$$\frac{\sigma}{\sigma_c^o} = \frac{\sigma_c^\infty}{\sigma_c^o} + \left(1 - \frac{\sigma_c^\infty}{\sigma_c^o}\right)e^{-\frac{t}{\tau}} \tag{5}$$

where σ_c^∞ is ultimate uniaxial strength, σ_c^o is short term strength, τ is retardation failure time. The plot of the above function is shown by continuous curves in the respective figures. This simple concept may be also extended to multi-axial stress state in which the yield surface of rocks under multi-axial loading conditions is modelled as the shrinkage of the surface(Aydan et al., 1995).

5. A Method of Prediction of Squeezing Potential and its Degree

5.1 Fundamentals of the Proposed Method

The fundamental concept of the method is based on the analogy between the axial stress-strain response of rocks in laboratory tests and tangential stress-strain response of rocks surrounding tunnels. Experiments have shown that one can hypothetically distinguish five distinct states of specimens during a

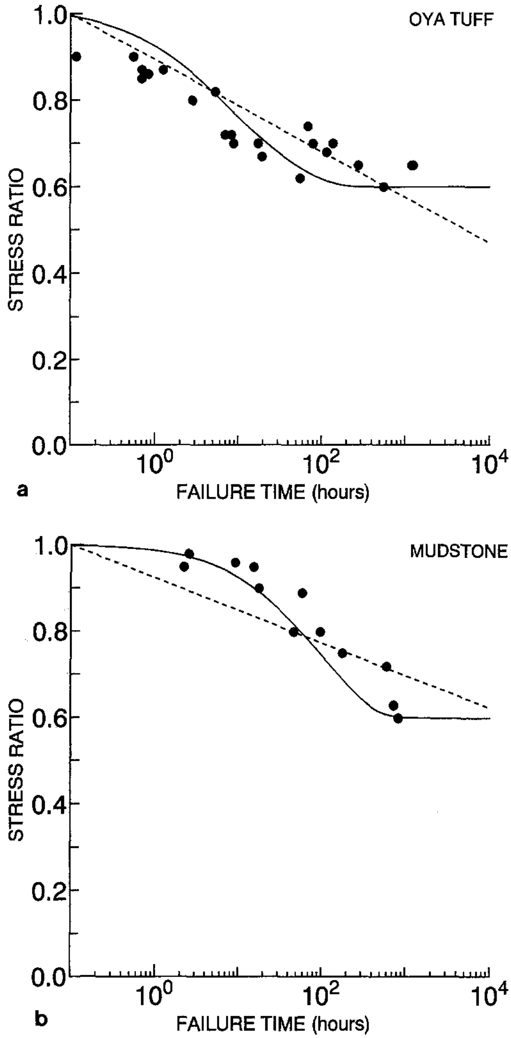


Fig. 7. a, b. Degradation of strength of Oya tuff and mudstone in creep tests

complete testing procedure. Stress-strain curves of rocks and soils obtained from uniaxial or triaxial tests at low confining pressures σ_3 (i.e. $\sigma_3/\sigma_c \leq 0.1$) are modelled and various levels of straining are defined as shown in Fig. 8a. The states of samples during a complete test are illustrated as shown in Fig. 8b. The following relations are established between normalised strain levels η_p, η_s, η_f obtained from normalising strain levels ϵ_p, ϵ_s, ϵ_f by the elastic strain limit ϵ_e defined in Fig. 8a from the data gathered by the authors (Aydan et al., 1992):

$$\eta_p = \frac{\epsilon_p}{\epsilon_e} = 2\sigma_c^{-0.17}, \quad \eta_s = \frac{\epsilon_s}{\epsilon_e} = 3\sigma_c^{-0.25}, \quad \eta_f = \frac{\epsilon_f}{\epsilon_e} = 5\sigma_c^{-0.32}. \tag{6}$$

We will introduce the concept of the five states observed in tests of rocks to make a proposal to predict and to define the squeezing potential of rocks and its degree. Considering the state around the opening, the tangential stress is most likely to be the maximum stress component. Thus, an analogy exists between the axial stress-axial strain curve in tests and the tangential stress-tangential strain response of rocks surrounding the openings. Accordingly, we can relate the five states of uniaxial and triaxial tests of rocks in laboratory or in-situ tests, to the squeezing potential of surrounding rock and its degree and classify them as given in Table 2.

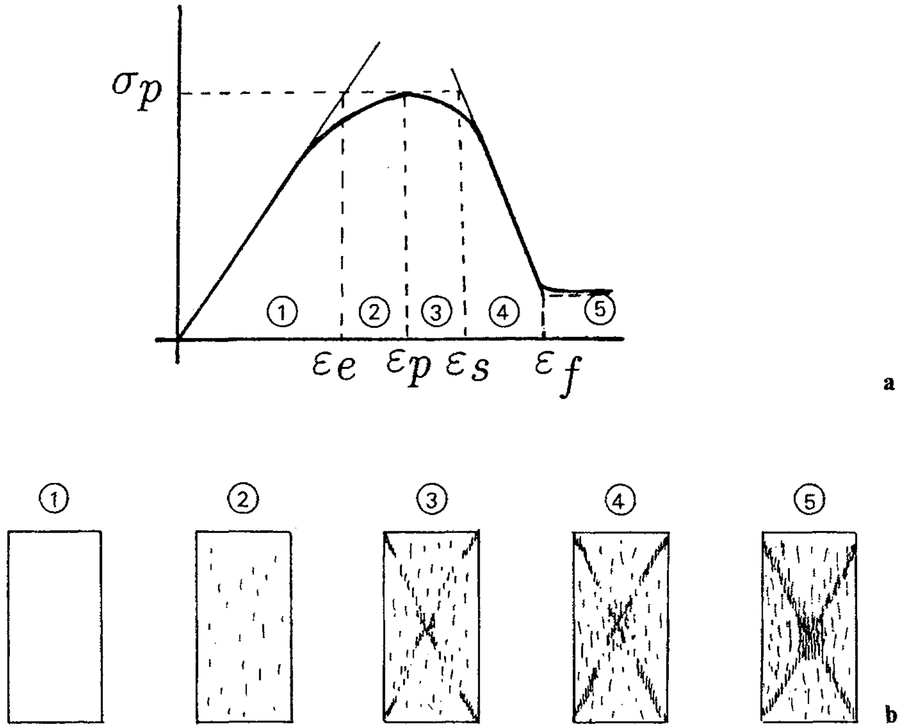


Fig. 8. Idealised stress-strain curve and associated state for squeezing rocks

5.2 An Analytical Model for Prediction of Strains and the Level of Squeezing of Tunnels

An analytical solution has been already developed by the authors for predicting strain and stress fields and the level of squeezing of circular tunnels in squeezing rocks in a hydrostatic state of stress (see Aydan et al., 1993b). Therefore, the final expressions for tangential wall strain for each state will be given only:

1) Elastic state

Tangential strain and stress at tunnel wall can be obtained as

$$\epsilon_\theta^a = \frac{1+v}{E}(p_0 - p_i), \quad \sigma_\theta^a = 2p_0 - p_i. \tag{7}$$

Table 2. Classification of the degree of squeezing by the proposed method

Class No.	Squeezing degree	Symbol	Theoretical expression	Comments on tunnel behaviour
1	no-squeezing	NS	$\epsilon_\theta^e/\epsilon_\theta^e \leq 1$	The rock behaves elastically and the tunnel will be stable as the face effect ceases
2	light-squeezing	LS	$1 < \epsilon_\theta^a/\epsilon_\theta^e \leq \eta_p$	The rock exhibits a strain-hardening behaviour. As a result, the tunnel will be stable and the displacement will converge as the face effect ceases
3	fair-squeezing	FS	$\eta_p < \epsilon_\theta^a/\epsilon_\theta^e \leq \eta_s$	The rock exhibits a strain-softening behaviour, and the displacement will be large. However, it will converge as the face effect ceases
4	heavy-squeezing	HS	$\eta_s < \epsilon_\theta^a/\epsilon_\theta^e \leq \eta_f$	The rock exhibits a strain-softening behaviour at much higher rate. Subsequently, displacement will be larger and it will not tend to coverge as the face effect ceases
5	heavy-squeezing	VHS	$\eta_f < \epsilon_\theta^a/\epsilon_\theta^e$	The rock flows which will result in the collapse of the medium and the displacement will be very large. And it will be necessary to re-excavate the opening and install heavy supports

If the tunnel is strained to its elastic limit, then $\sigma_\theta^a = \sigma_c$ with $p_i = 0$. Thus, we have the elastic strain limit as:

$$\epsilon_\theta^e = \frac{1+v}{E}\frac{\sigma_c}{2}. \tag{8}$$

Using the relation above in Eq. (7), one obtains the normalised tunnel wall strain as:

$$\xi = \frac{\epsilon_\theta^a}{\epsilon_\theta^e} = 2\frac{1-\beta}{\alpha} \leq 1, \quad \beta = \frac{p_i}{p_o}, \quad \alpha = \frac{\sigma_c}{p_o}. \tag{9}$$

2) Perfectly plastic state

Tangential strain at tunnel wall can be obtained as

$$\epsilon_\theta^a = \frac{1+v}{E}(p_0 - \sigma_{rp}), \ \frac{R_{pp}}{a}^{f+1}, \tag{10}$$

elastic limit is given as:

$$\epsilon_\theta^e = \frac{1+v}{E}(p_0 - \sigma_{rp}). \tag{11}$$

Using the above relation in Eq. (10), one obtains the normalised tunnel wall strain as:

$$\xi = \frac{\epsilon_\theta^a}{\epsilon_\theta^e} = \left[\frac{2}{q+1}\left\{\frac{(q-1)+\alpha}{(q-1)\beta+\alpha}\right\}\right]^{\frac{f+1}{q-1}}. \tag{12}$$

3) Residual plastic state
Tangential strain at tunnel wall can be obtained as

$$\epsilon_\theta^a = \frac{1+v}{E}(p_0 - \sigma_{rp})\eta_{sf}\frac{R_{pb}}{a}^{f^*+1} . \tag{13}$$

Using Eqs. (11) and (13), one obtains the normalised tunnel wall strain as:

$$\xi = \frac{\epsilon_\theta^a}{\epsilon_\theta^e} = \eta_{sf}\left[\frac{\frac{2}{q+1}\left\{\frac{(q-1)+\alpha}{(q-1)}\right\}\eta_{sf}^{-\frac{q-1}{f+1}} - \frac{\alpha}{q-1} + \frac{\alpha^*}{q^*-1}}{\beta + \frac{\alpha^*}{q^*-1}}\right]^{\frac{f^*+1}{q^*-1}}, \alpha^* = \frac{\sigma_c^*}{p_o}. \tag{14}$$

5.3 Consideration of Time-Dependent Behaviour of Squeezing Rocks

Relationships established in Section 4 can be used if available data for computations is insufficient. If the time-dependent characteristics of the uniaxial strength of rocks are known, it may be possible to determine the time-dependent variation of various mechanical properties of rocks from the relationships established in Section 4.

Using the approach originally proposed by Ladanyi (1974), together with the time-dependent variation of mechanical properties involved in the equations given in the previous sub-section, it is possible to determine the time dependent deformation of tunnels in squeezing rocks. Under multi-axial initial stress conditions and complex tunnel geometry, excavation scheme and boundary conditions, the use of numerical techniques is necessary. For numerical analyses, the time-dependent behaviour of squeezing rocks may be modelled by an approach proposed by Aydan et al. (1995).

6. Applications to Actual Tunnels and Discussions

Next, we applied our method to predict the squeezing potential of rocks and their degree during the excavation of Orizume Tunnel in Tohoku region of Japan, which is a 2300 m long highway tunnel with a cross-section area of 70 m^2. During the excavation of a 300 m long section of the tunnel between Stations 97 + 00 and 100 + 00, very large deformations exceeding 2000 mm at some locations were observed. The geological formations along the tunnel alignment consisted of tuff and mudstone of tertiary period. In the squeezed section, the rocks contained Na-type montmorillonite with a range of 10–60%. The strength data along the tunnel alignment are scarce. However, an elastic wave velocity zoning was available. Therefore, we used the distribution of elastic wave velocities of rocks to obtain the uniaxial strength of rocks from Eq. (1) and other required data from other equations for calculations and predicted the squeezing potential of rock and its degree. Figure 9a shows the elastic wave distribution zoning along the analysed section. Figure 9b shows the predicted squeezing potential of surrounding rocks, together with observed tangential strain levels. As noted from the figure, the calculations closely predict the actual performance of surrounding rocks. Good agreement between predictions and observations confirms the validity and reliability of the proposed method.

Similarly, predictions for Nou Tunnel, Nabetachiyama Tunnel and Taw. Tunnel were performed and results are shown in Figs. 10, 11, 12. As also noted from these figures, the calculations closely predict the actual performance of surrounding rocks. Good agreement between predictions and observations confirms the validity and reliability of the proposed method once again.

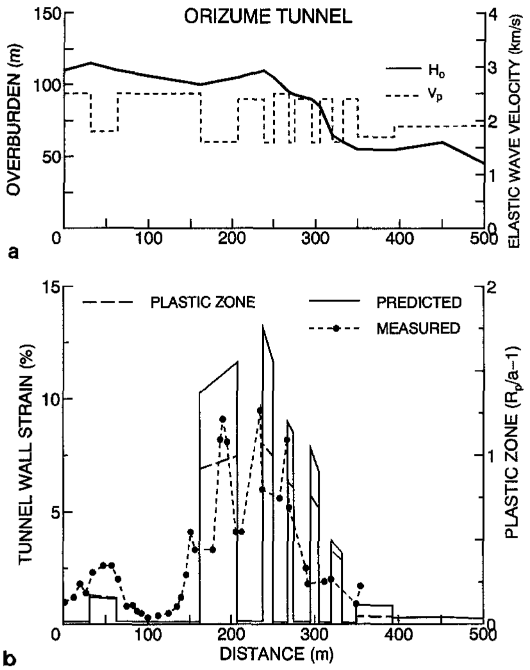


Fig. 9. Elastic wave distribution and predicted squeezing potential of rocks of Orizume tunnel

The deformation of Taw. tunnel in the squeezing section shown in Fig. 12 had been continuing for more than 1500 days after the completion of excavation when the authors got involved with this tunnel. Therefore some investigations were necessary for the causes of time-dependent deformations. Additional laboratory tests and in-situ pressuremeter tests and borings were conducted at certain locations along this section. Some swelling tests were also performed. The swelling tests indicated that rocks in this section have no swelling potential. This was unexpected and further tests on the time-dependent characteristics of rocks were felt to be necessary. Although these tests have not been undertaken yet, some attempts were made to gather information on long-term tests on soft rocks available in literature and observations on measured

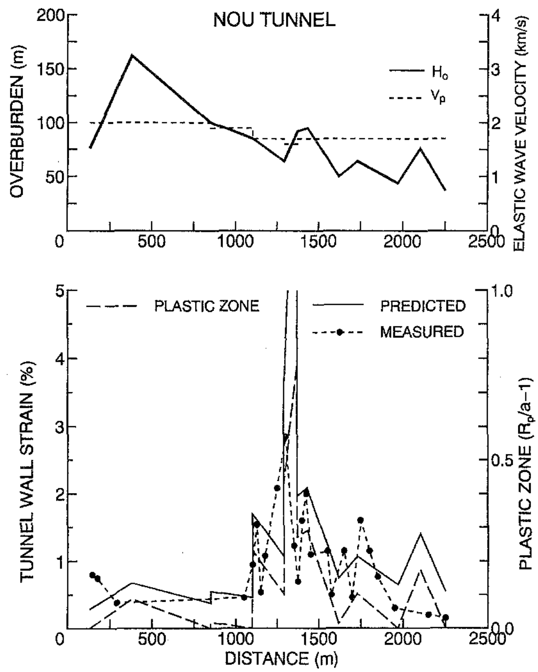


Fig. 10. Elastic wave distribution and predicted squeezing potential of rocks of Nou tunnel

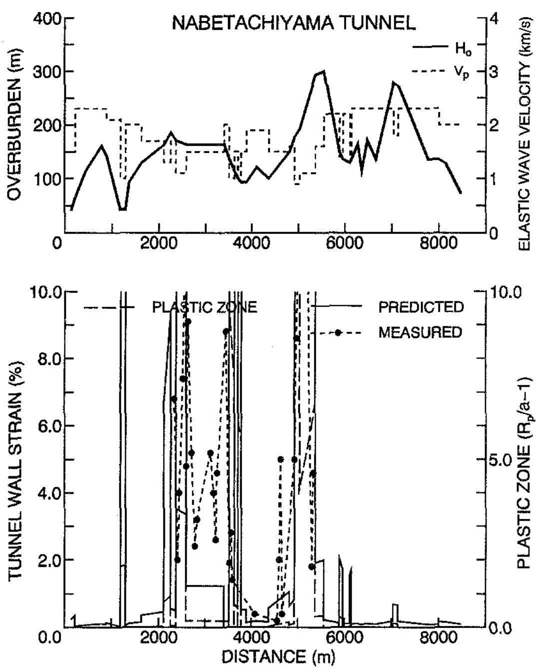


Fig. 11. Elastic wave distribution and predicted squeezing potential of rocks of Nabetachiyama tunnel

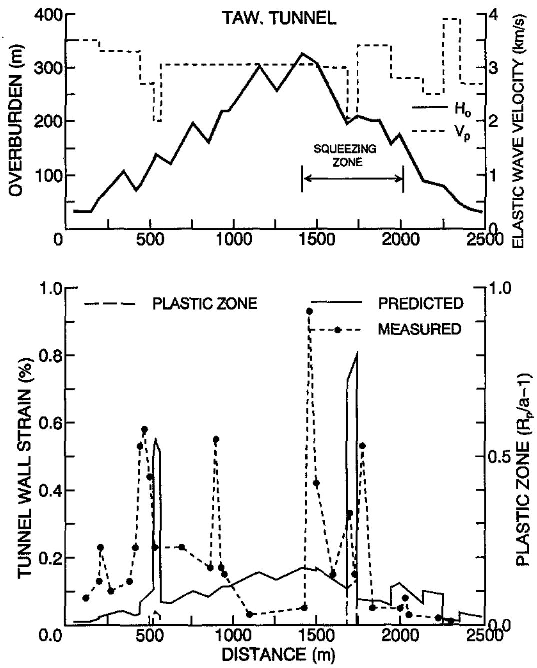


Fig. 12. Elastic wave distribution and predicted squeezing potential of rocks of Taw. tunnel

deformation – time relations so that degradation models for mechanical characteristics could be developed.

The degradation models have been incorporated in the theoretical model in which each parameter changes its value with time. Figure 13 shows predicted deformation responses for the tunnel without any support system for different values of short term uniaxial strength of rock in the squeezing section by assuming that the long term strength was 0.6 times the short term one together with a retardation failure time of 500 days. The response of the tunnel for a short term uniaxial strength of 7.3 MPa seemed to be reasonable by taking into account the observed deformation responses when initial predictions were made. The short term uniaxial strength of rock in this section was varied between 4.8 MPa and 9.7 MPa. Next a series of parametric studies were carried out by varying the ratio of the long term strength to the short term between 0.5 and 0.6 by keeping the short term strength as 7.3 MPa. Figure 14 shows measured displacement responses at the center of the floor together with those from parametric analyses (note that the initial deformation response due to excavation is subtracted from the calculated displacement responses). The calculated displacement-time response curves are similar to those observed. It is interesting to note that sudden changes of deformation gradient observed in measurements can be also simulated by the proposed model which is associated with the transition of the surrounding rock from perfect plastic behaviour to brittle plastic behaviour. In-spite of slight differences between the observations and predictions, the general

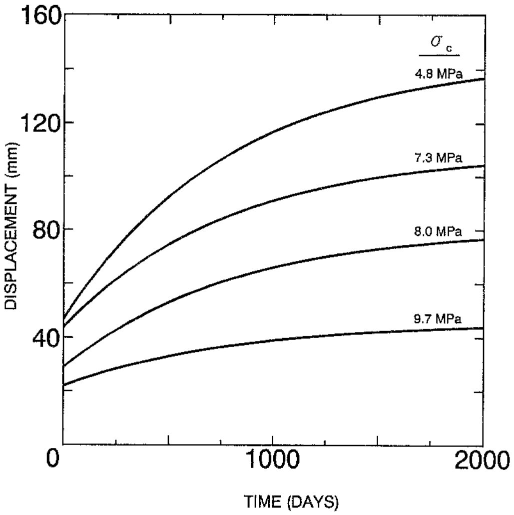


Fig. 13. Predicted time-dependent deformation of the squeezing section of Taw. tunnel (short-term unixial strength is variable)

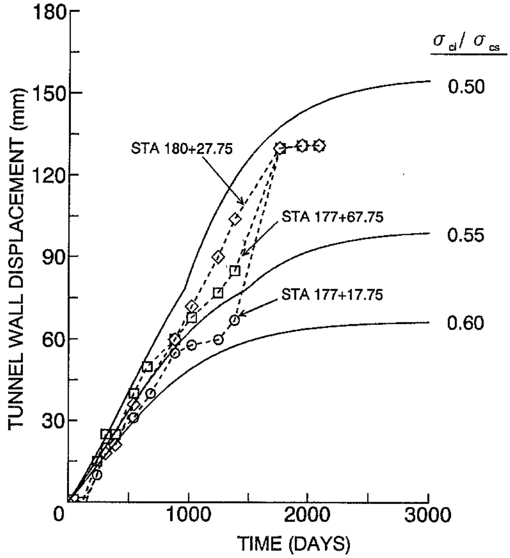


Fig. 14. Comparison of predicted time-dependent displacement response with observed floor heave for the squeezing section of Taw. tunnel

tendency confirms the basic idea for modelling the time-dependent characteristics of squeezing rocks.

7. Conclusions

The present study clarified the squeezing phenomenon of rocks in tunnelling, its mechanism and associated factors by carefully studying failures and observations on tunnels in Japan. Then a general method proposed to predict the squeezing potential of rocks around tunnels and its degree and a specific application of the method to circular tunnels under hydrostatic state of stress is described. The applicability and validity of the proposed method has been checked by comparing the predictions with actual observations. It is found that the predictions by the proposed method well agree with observations.

Then time-dependent behaviour of squeezing rocks is modelled as the degradation of deformation and strength characteristics of rocks as a function of time by utilising information obtained from creep tests. The model has been proved to be successful to explain the observed time-dependent behaviour of a tunnel driven through squeezing sedimentary rocks of volcanic origin, which has been continuing to deform even though it has passed more than four years after the completion of its construction. The computed results are generally consistent with observations.

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