Contemporary Supports in Squeezing Rock and a Possible Alternative Method

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Abstract. Challenges of the support for tunnelling in squeezing ground are still common and have yet to be completely resolved. In this paper, contemporary supports are introduced and 3 critical problems for future support methods through squeezing ground are summarized. With the continuous development of tunnel construction equipment, based on the perspective of preserving and exerting the self-bearing capacity of surrounding rock, an innovative clue of tunnel support, pre-structuralized method, is put forward. The proposed method with a different construction sequence stacking up against the existing support evolved from traditional advanced pre-support. Pre-structure should be formed along with tunnel profile before excavation at the tunnelling face. On the one hand, it could effectively control the deformation of surrounding rock in front of the excavating face; on the other hand, the pre-structure can ensure the surrounding rock under triaxial compression from beginning to end to boost its strength. There be two possible technical routes to achieve a pre-structure in this preliminary study. One is to construct lining in advance of excavating, the other is to reinforce the surrounding rock in advance to form a pre-reinforcement hollow. Based on the numerical simulation, considering the rheological effect of squeezing rock, the deformation control effect of the pre-structure method is studied. Firstly, the numerical model is verified and calibrated by the analytical solution of an unsupported circular opening. Then, the comparative analysis of the deformation control effects of the traditional support, the pre-lining in advance, and the pre-reinforcement hollow in advance is carried out, respectively. The results show that: 1. Compared with the traditional support, the pre-structuralized method can effectively preserve the surrounding rock after excavation. The plastic zone and deformation convergence of surrounding rock of the latter are much smaller than that of the traditional one; 2. As for the pre-structuralized method, compared with the pre-reinforcement hollow in advance, the deformation control effect of the pre-lining in advance is better, but the internal force of the latter has exceeded the design value. Further considering the stability of lining in the actual construction process, the pre-lining in advance is not a preferable scheme, which needs further study. In contrast, the pre-reinforcement hollow in advance can reduce the deformation by improving the physical and mechanical properties, and self-bearing capacity of surrounding rock, which is the future direction of support.

1. Introduction
Weak rocks under a high overburden characterizing squeezing behaviour are typically called “squeezing rocks”. It is one of the most difficult geological conditions in which time-dependent large deformation and even collapse of the opening occur after excavation (Hoek E et al. 2009, Barla G et al. 2011, Kimura F et al. 1987 and Zhang Z 2003). Squeezing large deformation had attracted people’s attention since Tauern Motorway Tunnel excavated in 1975. The maximum displacement and displacement rate of the tunnel are 1000mm and 200mm/d respectively (Hoek E et al. 2001). Since then, as relevant project cases surged, tunnelling in squeezing ground had been an increasingly common but tricky challenge. Well-known projects include Arlberg Tunnel (1979) in Austria, Enasan Tunnel (1985) in Japan, Gotthard Railway Tunnel (2016) in Switzerland and so on. With the widely construction of tunnels in West China, unprecedented squeezing conditions and large convergences occurred in some projects, such as Wushaoling Railway Tunnel, Muzhailing Tunnel and Guanjiao Tunnel.

K. Terzaghi (1946) defined squeezing rock, which has a high content of mica or clay minerals, and the squeezing deformation gradually intrudes into the opening clearance. Squeezing deformation is also a time-dependent deformation behavior, which usually occurs around the excavating face and is generally caused by creep due to instability of ultimate shear stress. The deformation may stop during excavation or last for a long time (Barla et al. 111). Han et al. (2020) attributed squeezing deformation to the stress disequilibrium of surrounding rock after excavation. The key to control deformation is to put the surrounding rock in an equilibrium state. To sum up, tunnelling through squeezing ground has the following characteristics. Firstly, large convergence, high convergence rate, and large plastic zone occur after excavation. Secondly, the rheological properties of surrounding rock are significant causing time-dependent deformation which does not converge for a long time, and the damage of tunnel support structures may happen in operation stage.

Since overstressing caused by squeezing large deformation can pose a threat to the safety of support structure, various support methods were discussed in the past years. The present paper starts with an overview of contemporary support methods (Section 2) and a possible alternative method with an innovative support concept, pre-structuralized method, is proposed (Section 3). Finally, numerical simulations are conducted, and the pre-structuralized method is proved to be more effective than traditional methods (Section 4 and 5).

2. Contemporary supports for squeezing conditions

The control technology of squeezing large deformation can be roughly divided into two categories (Kovári, 1998). 1. Resistance principle: concrete-filled steel tubular support, high-strength prestressed anchor bolts (cables) and other conventional support methods; 2. Yielding principle: layered support and yielding structure (including retractable steel arch, yielding lining, yielding anchor bolts), etc.

2.1. Rigid support technology

Because of high geo-stress and rheological effect of soft rock, the support structure needs enough stiffness reserve to ensure the long-term stability of the tunnel. The most common route is the traditional support with parameters beyond specifications. Muzhailing Tunnel adopted 122b steel arch with an only 50cm distance, and the thickness of shotcrete layer and secondary lining are more than 25cm and 60cm, respectively (Han et al. 2020). Li put forward various types of concrete-filled steel tubular support systems, which have adopted in some mine roadway projects (Li et al. 2016).

2.2. Deformable support technology

Overstressing of the support structure is the main hazard of tunnelling in squeezing rock. According to the yielding principle, the rock pressure can be reduced by allowing deformations to occur (Ramoni and Anagnostou, 2010). Layered support divides primary support into two or even multi-layer and the pressure is released in the time interval between the construction of each layer. Layered support has been successfully applied in Wushaoling Tunnel and Maoxian Tunnel (He et al. 2018). The most typical retractable steel arch is the U-shaped steel retractable support. Different from the rigid-joint steel arch, the retractable joints can accommodate with surrounding rock to deform within the
designed yielding deformation. The constant resistance large deformation anchor bolt proposed by He et al. (2014) can absorb the deformation energy by the structure deformation. During the energy absorption process, the anchor can still maintain the constant resistance and stable deformation to ensure the stability of the surrounding rock.

2.3. Critical problems for future support methods in squeezing ground
Based on the previous research results and considering squeezing behaviors in weak rock tunnel under a high overburden, 3 critical problems are proposed for the development and application of support system. 1. The supporting structure should fully utilize the self-bearing capacity of rock mass. The supporting structure, as a safety reserve, plays the auxiliary role; 2. The supporting structure must have enough rigidity and strength to resist the continuous pressure such as long-term rheology and crushing expansion to ensure the long-term stability of the opening; 3. The effect of advanced pre-support on deformation control should be emphasized. Advanced pre-support should not only be an endeavor to prevent collapse during tunnel excavating, but also a critical role of maintaining stress equilibrium of surrounding rock before and after excavation.

3. The proposal of the pre-structuralized method
Conventional tunnel methods including drilling-blasting method and TBM method cannot avoid breaking the stress equilibrium of surrounding rock because their construction sequences are basically excavating and then installing supports. To overcome the major drawback of conventional methods, only the pre-structuralized method with a different construction sequence is feasible. Pre-structure should be formed along with tunnel profile before excavation at the tunnelling face. On the one hand, it could effectively control the deformation of surrounding rock in front of the excavating face; on the other hand, the pre-structure can ensure the surrounding rock under triaxial compression from beginning to end to boost its strength.

Based on the above analysis, there are two possible technical routes to achieve a pre-structure in this preliminary study. One is to construct the lining in advance of excavating to maintain the stress equilibrium of surrounding rock before and after excavation, namely the pre-lining in advance, the other is to reinforce the surrounding rock in advance to form a pre-reinforcement hollow to promote the disturbed surrounding rock reaches equilibrium state as soon as possible, namely the pre-reinforcement hollow in advance. The construction sequences of the above support methods are shown in Figure 1.
4. Comparative analysis of various support methods

In this section, based on the numerical simulation, considering the rheological effect of squeezing rock, the deformation control effect of the pre-structuralized method is investigated. Firstly, the numerical model is verified and calibrated by the analytical solution of an unsupported circular opening. Then, the comparative analysis of the deformation control effects is carried out.

4.1. Main assumptions and model establishment

The main assumptions of the numerical simulation are as follows: 1. The surrounding rock and the tunnel structure are homogeneous and isotropic continuous in hydrostatic pressure field. 2. M-C strength criterion is used for the strength of surrounding rock and pre-reinforcement hollow, and elastic model is used for the support structure. 3. Kelvin-Voigt viscoelastic-plastic constitutive model is used as the creep model of surrounding rock and pre-reinforcement hollow. 4. As for pre-lining in advance, the lining is assumed to be a homogeneous elastic ring to simplify the analysis and its creep behaviour is neglected.

FLAC3D is used to establish a two-dimensional plane strain creep numerical model, in which the solid element of hexahedral mesh is used to simulate the surrounding rock and reinforcement hollow, and the shell element is used to simulate the pre-lining.

The radius of the opening is 4m. A quarter of the actual model is taken along the X and Z axes for analysis due to the hydrostatic-pressure assumption. Based on Saint-Venant theorem, the model radius is 100 m, which is 25 times of the opening radius (4 m). The radial tectonic stress of 16.7 MPa was applied along the perimeter of the model. In terms of boundary conditions, displacements of X-direction and Z-direction are constrained by the left and bottom boundary, and Y-direction displacement is constrained by the front and back boundary of the model (Figure 2).

4.2. Constitutive model and parameters

The squeezing behaviour of surrounding rock is characterized by Kelvin-Voigt viscoelastic-plastic constitutive model, which consist of a Hooke body, a Kelvin body, and a Plastic body with M-C strength criterion. The model can represent instantaneous deformation stage, decay creep stage and
accelerated creep stage of rock mass (Figure 3). When $\sigma<\sigma_s$, the model is equivalent to a Kelvin-Voigt model.

|           | Bulk modulus $K$ (GPa) | Shear modulus $G$ (GPa) | Density $\rho$ (kg/m$^3$) | Cohesion $c$ (MPa) | Friction angle $\varphi$ ($^\circ$) | UCS $\sigma_c$ (MPa) |
|-----------|------------------------|-------------------------|---------------------------|-------------------|-----------------------------------|----------------------|
| Mudstone  | 1.2                    | 0.8                     | 2000                      | 0.80              | 25                                | 2.51                 |
| C50 concrete lining | 19.2               | 14.4                    | 2500                      | -                 | -                                 | -                    |
| Pre-reinforcement hollow | 2.4                | 1.6                     | 2000                      | 1.51              | 28                                | 5.02                 |

**Table 2. Rheological parameters of surrounding rock**

|           | Shear modulus - Hooke $G_m$ (GPa) | Shear modulus - Kelvin $G_k$ (GPa) | Viscosity coefficient - Kelvin $\eta_k$ (GPa·h) |
|-----------|----------------------------------|-----------------------------------|-----------------------------------------------|
| Mudstone  | 0.8                              | 0.2                               | 50                                            |
| Pre-reinforcement hollow | 1.60                          | 0.40                             | 100                                           |

4.3. Verification and calibration of constitutive model and numerical model

4.3.1. Verification of creep constitutive model. In this section, taking microelements as object to carry out uniaxial compression test (Figure 4), viscoelastic part of K-V viscoelastic-plastic constitutive model is verified between the results of numerical simulation and analytic solution as in equation (1). The maximum relative error is only 3.02%, so the creep model built in FLAC$^{3D}$ can truly reflect the creep characteristics of K-V viscoelastic model (Figure 5).
4.3.2. Verification of numerical model. In this section, the numerical model is verified by the elastic-plastic analytical solution and viscoelastic analytical solution of a circular opening. Limited to the length of the paper, the detailed analytical solutions are listed in the appendix. For elastic-plastic deformation, the maximum relative error of the minimum principal stress is 13.96%; the maximum relative error of the maximum principal stress is 15.95% (Figure 6). For viscoelastic deformation, the maximum relative error of radial displacement of surrounding rock is 0.21% (Figure 7). In conclusion, the relative errors between the numerical simulation and the analytical solution are limited enough, so the numerical model is verified.

4.4. Comparative analysis of deformation control effect
In this section, it is assumed that the tunnel will be supported 12 hours after excavation. Figure 8 shows the 15 days creep curves among different support sequences. According to the creep curves, the radial deformation of surrounding rock can be divided into three stages. The first stage is the instantaneous elastic-plastic deformation after excavation, the second stage is the time-dependent viscoelastic decay creep deformation, and the third stage is the stable stage of surrounding rock with deformation rate approaching zero. The viscoelastic creep deformation mainly occurs after the excavation of the opening and before the support construction, except for the unlined tunnel. The
viscoelastic creep deformation is not obvious after the support construction, and the deformation is basically stable, which indicates that the support reaction can effectively control the creep deformation. Moreover, there is no obvious creep deformation in the whole process of pre-lining in advance.

The radial deformation of the pre-structuralized method is far less than that of the traditional support method. In terms of the initial deformation, the pre-reinforcement in advance and pre-lining in advance are 35.6% and 97.1% less than the traditional support respectively, which proves that the pre-structure, especially the pre-lining can effectively control the initial deformation of the surrounding rock.

![Figure 8. Creep curves of various support methods](image)

4.5. Comparative analysis of plastic zone and stress distribution

The location of the plastic zone boundary of the surrounding rock of the opening is determined by the maximum tangential compressive stress, that is, the location of the maximum principal stress. The distributions of the maximum principal stress of the surrounding rock of the traditional support method and the pre-lining method are shown in Figure 9.

From the stress curve characteristics, the maximum principal stress of the surrounding rock first increases sharply and reaches the maximum, and then gradually decreases to the original rock stress of 16.7 MPa. The maximum principal stress distribution of the pre-lining method is more uniform, and the maximum tangential compressive stress and shear stress are smaller than those of traditional support one, which are 20.03 MPa and 3.36 MPa, respectively (Figure 3). The plastic zone depth of the pre-lining structure method tunnel is 69.1% smaller than that of the traditional supporting tunnel, which has advantages in the stability of surrounding rock compared with the traditional support.

| Table 3. Stress characteristic value of surrounding rock among various support methods (unit: MPa) |
|-----------------|-----------------|-----------------|
|                  | Unlined tunnel  | Traditional support | Pre-lining in advance |
| Maximum principal stress | 23.75           | 23.02            | 20.03                |
| Shear stress     | 6.72            | 6.00             | 3.36                 |
4.6. **Comparative analysis of structural stress**

For the circular opening in hydrostatic pressure field, the supporting structure mainly bears the axial force, and the bending moment and shear force can be ignored, so the earth pressure on the supporting structure and the tangential compressive stress inside the structure are mainly discussed.

The earth pressure of the pre-lining method is 26.5% larger than that of the traditional support structure, and the earth pressure is closer to the original rock stress (16.7 MPa). The results show that the time-dependent effect of earth pressure is not significant, which indicates that the rheological effect of surrounding rock is significantly weakened, while the earth pressure of traditional support increases with time with an obvious rheological (Figure 10).

![Figure 9](image)

(a) Traditional support  
(b) Pre-lining in advance

**Figure 9.** Characteristic curve of maximum principal stress in surrounding rock on 50th day

| Compressive stress of supporting structure (unit: MPa) |
|-------------------------------------------------------|
| Traditional support | Pre-lining in advance |
|----------------------|------------------------|
| 87.38                | 124.50                 |

**Table 4.**

![Figure 10](image)

**Figure 10.** Time history curve of earth pressure of support structure in 20 days

The compressive stress of the pre-lining is 42.5% higher than that of traditional support. The maximum compressive stress of the former is 124.50 MPa, but the design value of axial compressive strength $f_{cd}$ of C50 concrete is only 23.1 MPa, which means pre-lining method with C50 concrete cannot meet the demand of bearing capacity (Figure 4).
Above all, compared with the traditional support, the pre-lining in advance has obvious effect in controlling the deformation and weakening the squeezing behaviour of surrounding rock. However, considering the structural stress, the current materials and technology cannot meet the requirements, which is worthy of further study in the future. In contrast, the deformation control effect of the pre-reinforcement in advance is between the traditional support and the pre-lining in advance, which is more feasible in practical engineering, so it is the most promising method to deal with squeezing large deformation in the future.

5. Conclusion
This paper introduced contemporary supports and summarized 3 critical problems for future support methods through squeezing ground. An innovative clue of tunnel support, pre-structuralized method, was proposed. The numerical simulation was verified and conducted to investigate the deformation control effect among various support methods.

The results show that: 1. Compared with the traditional support, the pre-structuralized method can effectively preserve the surrounding rock after excavation. The plastic zone and deformation convergence of surrounding rock of the latter are much smaller than that of the traditional one; 2. As for the pre-structuralized method, compared with the pre-reinforcement hollow in advance, the deformation control effect of the pre-lining in advance is better, but the internal force of the latter has exceeded the design value. Further considering the stability of lining in the actual construction process, the pre-lining in advance is not a preferable scheme, which needs further study. In contrast, the pre-reinforcement hollow in advance can reduce the deformation by improving the physical and mechanical properties, and self-bearing capacity of surrounding rock, which is the future direction of support.

6. Appendices
The analytic solution for this problem may be found in Salen (1969). The yield zone radius, $R_0$, may be expressed, in general terms, as

$$
\frac{R_0}{a} = \left\{ \frac{2}{K_p + 1} + \frac{P_i}{P_0} k_p \right\}^{1/k_p} \quad (2)
$$

in which $a$ is the hole radius, $P_0$ is the absolute value of the in-situ isotropic stress, $P_i$ is the pressure inside the hole (0 Pa, in this case), and

$$
K_p = \frac{1 + \sin \phi}{1 - \sin \phi} \quad (3)
$$

$$
k_p = \frac{1}{K_p - 1} \quad (4)
$$

$$
q = 2c_0 \sqrt{K_p} \quad (5)
$$

The radial stress at the elastic/plastic interface may be written as

$$
\frac{\sigma_{re}}{P_0} = -\frac{1}{K_p + 1} \left( 2 - \frac{q}{P_0} \right) \quad (6)
$$

The stresses in the plastic zone have the form
\[
\sigma_r = \frac{q}{P_0} k_p - \left( \frac{P}{P_0} + \frac{q}{k_p} \frac{r}{a} \right)^{k_r - 1}
\]  
\[\sigma_\theta = \frac{q}{P_0} k_p - K_p \left( \frac{P}{P_0} + \frac{q}{k_p} \frac{r}{a} \right)^{k_r - 1}
\]
in which \( r \) is the distance from the hole axis.

The stresses in the elastic zone are
\[\frac{\sigma_r}{P_0} = -1 + \left( 1 - \frac{\sigma_\theta}{P_0} \right) \left( \frac{R_0}{r} \frac{a}{r} \right)^2
\]
\[\frac{\sigma_\theta}{P_0} = -\frac{\sigma_r}{P_0} - 2
\]

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