Analysis of deformation and failure characteristics of soft and broken surrounding rocks of super-large section tunnel constructed using benching tunneling method

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Abstract. The benching tunneling method can overcome the problems of insufficient working space and low construction efficiency in the soft and broken surrounding rock tunnel with super large section, but the design and construction theory and method are under developing. In order to solve the problem of insufficient basis for the design of the large-section bench method in the weak and broken surrounding rock, a three-dimensional elastoplastic model was established. The deformation and fracturing of the surrounding rock outside the tunnel excavation profile and in front of the tunnel face were used as indicators to analyze the deformation and failure characteristics of the weak and broken surrounding rock under benching tunneling method. The calculation results show that the grade of surrounding rock worsens (grade IV-V). Depending on the initial support design conditions, instability of the supporting structure or the tunnel face may occur. In the condition that grade of surrounding rock does not lose stability, the leading deformation accounts for a larger proportion of the whole deformation, indicating that the leading deformation is the key to the stability control in the benching tunneling method. When the tunnel face is unstable, the surrounding rock within 4.5m in front of the tunnel and 1.5m above the arch collapses, and the deformation and damage develop from the vault down to the foot of the wall.

1. Introduction
The excavation of large-section tunnels with weak and broken surrounding rock often adopts traditional construction methods such as double-side drift method, CD method, and CRD method. However, these methods have limitations such as multiple construction sections, narrow construction space, cumbersome procedures, and low degree of mechanization, which often result in low construction efficiency, increased engineering cost and tight construction period. Large-section or full-section tunnel has large space and allows large-scale machinery rapid construction, which can improve construction efficiency and construction quality. The construction conditions are also conducive to ensuring construction safety. However, due to the surrounding rock conditions of the tunnel, the excavation span is limited. Therefore, the large-span tunnel construction can only be applied to hard rock, but it cannot not applied in soft surrounding rock [1].
With the development of surrounding rock reinforcement technology and large-scale construction machinery, the techniques for the excavation of tunnel with large-section or full-section in soft surrounding rock have gradually matured. At present, the full-face excavation method abroad is the rock-soil control deformation analysis method (Pietro Lunardi method) [2]. The core point is that the stability of the core soil in front of the tunnel face determines the stability of the entire tunnel. The use of glass fiber anchors is recommended. Advanced construction methods such as near-horizontal grouting and mechanical pre-cutting to pre-protect or reinforce advanced core soil have been successfully applied to a large number of tunnel projects in Europe in the past 30 years, with a total length of more than 1000km. However, limited by the construction cost and the average technical level of the construction team in China, this method is difficult to implement the advanced construction method emphasized by the Pietro Lunardi method. Most mountain tunnels in China are still constructed by the traditional new Austrian method. The attempts of Pietro Lunardi method have only been carried out in the Liuyang River Tunnel and Taoshuping Tunnel. Therefore, according to the staged status quo of tunnel engineering in our country, scholars and experts have adopted the benching tunneling method for the construction of super-large section tunnels in soft and broken surrounding rock [3-4]. At present, some successful construction design experience has been accumulated, but the systematic construction design theory and methods have not formed yet, which limits the promotion of the benching tunneling method for the construction of tunnel with large section in the soft and broken surrounding rock. Therefore, this paper used the strength reduction method [5] to establish a numerical model for the benching tunneling method, and analyzed the deformation and failure characteristics of the super large-section tunnel in soft and broken surrounding rock by the benching tunneling method. This paper provides a basis for the design of benching tunneling method for the construction scheme of super large-section tunnel in soft and broken surrounding rock.

2. Analysis scheme

2.1. Calculation model

A three-dimensional model was established for calculation. The section of the tunnel is a three-lane with three-center circular curved side wall, with a clear width of 13.75m, a clear height of 5.0m, and a buried depth of 90m. In order to reduce the influence of the model boundary effect, the distance between the left and right boundary of the model and the center line of the tunnel is about 4 times the tunnel diameter, and the distance between the bottom boundary and the bottom of the tunnel is 4 times the tunnel height. The positive direction of the surrounding rock model in the other range is the tunnel excavation direction, the vertical upward direction is the positive direction of the Z axis, and the right direction of the tunnel excavation cross section is the positive direction of the x axis. The model length×width×height=140m×50m×162m. The model is excavated by the benching tunneling method. The initial support is C20 sprayed concrete + I22b steel frame + steel mesh, and the support thickness is 29cm. The model grid is shown in Figure 1.

Fig 1. The grid of the model.
2.2. Calculation parameters
In the calculation model, only the effect of initial support is considered. The surrounding rock and the initial support are Mohr-Coulomb ideal isotropic elastoplastic materials. The initial support strength is considered by the strength equivalent method. The cohesion and friction angle of the initial support and the maximum compressive strength and maximum tensile strength of the initial supporting concrete are drawn by drawing a Mohr circle to find the common tangent. According to the relevant recommended parameters in the design code of highway tunnels, the values of the surrounding rock (considered based on grade IV surrounding rock) and the calculation parameters of the supporting structure in the numerical model are shown in Table 1.

| Materials         | Density (kg/m³) | Elasticity modulus (GPa) | Poisson's ratio | Cohesive force (MPa) | Friction angle (°) |
|-------------------|-----------------|--------------------------|----------------|----------------------|-------------------|
| Surrounding rock  | 23              | 6.0                      | 0.25           | 0.10                 | 39                |
| Initial support   | 26              | 31.4                     | 0.2            | 2.27                 | 53                |

2.3. Calculation working condition
The strength reduction method was used to reduce the strength parameters of the surrounding rock (with the grade of the surrounding rock from good to bad, the reduction coefficient is gradually increased from 1.1 at 0.1 intervals until the surrounding rock becomes unstable). The reduction formula is as follows:

\[
c' = \frac{c}{\omega}, \quad \varphi' = \arctan \left( \frac{\tan \varphi}{\omega} \right)
\]

Where \( c \) and \( \varphi \) is the cohesion and friction angle of the surrounding rock before reduction, \( \omega \) is the strength reduction coefficient, and \( c' \) and \( \varphi' \) is the cohesion and friction angle of the surrounding rock after reduction.

2.4. Construction procedures
The specific construction steps of the calculation model are as follows:

1. Excavate the upper bench. The excavation height of the upper bench is 6m, and the excavation footage is 1m;
2. construct the initial support of the upper bench followed by excavating 1m, and the support length is 1m;
3. Excavate the lower bench with the excavation height of 4.3m. The excavation of the lower bench lags behind the upper bench by 6m. The excavation footage is the same as that of the upper bench;
4. Construct the initial support for the lower bench followed by excavating 1m, and the support length is 1m;
5. Excavate the invert. The invert is behind 12m from lower bench. The length of excavation at one time is 6m;
6. Construct the initial support for the invert, ibid.

3. The evolution of deformation of surrounding rock
The grade of surrounding rock gradually becomes worse with the continuous increase of the reduction coefficient. When the reduction coefficient of surrounding rock is 3.8, the surrounding rock becomes unstable. When the reduction factor is 1.5, 2.0, 2.5, 3.0, 3.5, 3.6, 3.7 and Y=20m, the variation law of the settlement of the surrounding rock vault in the study section with the construction step is shown in Figure 2.
Fig 2. The settlement of the surrounding rock vault with the construction step.

It can be seen from Fig. 2 that in the advanced deformation stage (before the excavation of the upper bench, 1-20 steps), the reduction coefficient (1.5-3.7) corresponds to the advanced vault settlement of 12.7mm, 16.1mm, 21.3mm, 32.2mm, 61.4mm, 73.2mm and 89.3mm. In the upper bench excavation stage (before the lower bench excavation, 20 steps to 25 steps), the reduction coefficient (1.5 to 3.7) corresponds to the settlement of 7.0mm, 5.3mm, 4.4mm, 9.1mm, 25.5mm, 40.7mm and 52.8mm. In the lower bench excavation stage (steps 26 to 35), the reduction coefficient (1.5~3.7) corresponds to the settlement 2.9mm, 3.0mm, 4.2mm, 9.5mm, 48.3mm, 70.4mm and 111.9mm, respectively.

The ratios of the settlement of the surrounding rock vault in the typical construction stage to the total deformation when the reduction coefficient is 1.5 to 3.7 are summarized in Table 2.

Table 2. The ratios of the settlement of the surrounding rock vault to the total deformation at the reduction coefficients.

| Construction steps         | Reduction coefficients |
|----------------------------|------------------------|
|                            | 1.5    | 2       | 2.5    | 3.0    | 3.5    | 3.6    | 3.7    |
| Advanced deformation       | 51.1%  | 63.5%   | 67.3%  | 61.9%  | 43.5%  | 37.6%  | 33.3%  |
| Upper bench excavation     | 34.7%  | 23.3%   | 18.3%  | 18.6%  | 19.5%  | 22.8%  | 21.4%  |
| Lower bench excavation     | 14.3%  | 13.2%   | 14.4%  | 19.6%  | 37.1%  | 39.6%  | 45.3%  |

It can be seen from Table 2 that with the increase of the reduction coefficient, the settlement proportion of the surrounding rock vault at the advanced deformation stage first increases and then decreases. However, at the excavation stage of the upper and lower bench, it first decreases and then increases. That is, when the reduction coefficient is less than 2.5, the settlement of the surrounding rock vault mainly occurs in the advanced deformation stage, and the surrounding rock stabilizes quickly after the supporting structure is constructed. When the reduction coefficient is greater than 2.5, for the worse surrounding rock (the grade of the surrounding rock is higher), the deformation after support is larger than that in the advance deformation stage. The settlement of the surrounding rock vault mainly occurs after the support structure is constructed, and the support structure tends to lose stability.

4. The analysis of surrounding rock destruction process

When the surrounding rock reduction coefficient is 3.8, the extrusion displacement value of the tunnel face is 590mm (losing stability at 29 construction steps). In order to analyze the failure process of the surrounding rock, the maximum shear strain and tensile strain greater than 0.2 are taken as the failure threshold, that is, when the shear strain and tensile strain are greater than 0.2, the material is considered to have shear slip and tensile failure. Figure 3 shows the longitudinal section of the changes in the surrounding rock failure zone at steps 22, 23, 24, 27, 28, and 29.
Fig 3. The development of the surrounding rock failure zone during tunnel construction.

It can be seen from Figure 3 that during tunnel excavation (before S28), the surrounding rock of the arch of the supported section within a certain range behind the face has been damaged, but the tunnel has not been unstable due to the support structure. At step S29, large-scale damage at the surrounding rock of the tunnel face causes the tunnel to lose stability. The damage range is about 4.5m in front of the tunnel face and 1.5m above the vault.

In order to further analyze the development law of the surrounding rock failure area, the changes in the surrounding rock failure area in the study section at Y=28m and Y=29m are extracted as shown in Figure 4.

Fig 4. The surrounding rock failure at typical sections.

It can be seen from Figure 4:

1. In the process of tunnel excavation, the damage pattern of surrounding rock is from vault to arch, then to arch foot and to side wall.

2. Before the excavation section arrives, the upper surrounding rock of the face has been partially damaged. After excavation, the surrounding rock at the vault has been damaged. As the excavation surface advances, the damaged area of the supporting section of the upper bench is gradually enlarged.

5. Deformation law of surrounding rock
The Y=28m section is selected as the research object, and the curve of the total displacement of monitoring points at key locations of the tunnel excavation profile with the excavation step is shown in Figure 5.
Fig 5. Curve of total displacement of monitoring points at key locations of tunnel excavation profile with excavation steps.

It can be seen from Figure 5 that the displacement at each monitoring point gradually increases with the excavation step, and the deformation gradually develops from the vault to the foot of the wall, and the amount of deformation at vault, arch, arch foot, side wall and the foot of the wall decreases in sequence.

The Y=29m section is selected as the research object, and the displacement curve of the rock mass at monitoring points within 15m above the tunnel (vertical displacement) and within 10m in front of the tunnel (longitudinal displacement) face with excavation steps is drawn as shown in 6.

Fig 6. (a) (b) The deep rock deformation with the excavation steps.
It can be seen from Fig. 6 that the vertical displacement of the measuring points G1, G2, and G3 at the arch is relatively large. That is, the surrounding rock within about 9m of the arch is greatly disturbed. The displacement of the measuring points Q1, Q2, and Q3 in front of the face are large, that is, the surrounding rock within about 6m in front of the tunnel face is greatly disturbed.

Selecting Y=15m, 20m, 25m, 30m, 35m, 40m, 45m and 50m sections as the research objects, the curve of the vertical displacement of the vault with the excavation steps is as shown in Figure 7.

**Fig 7.** Vertical displacement curve of surrounding rock at the vault of typical sections.

It can be seen from Figure 7:

1. The advanced impact distance of tunnel excavation is about 20m. At the 8th step (excavation 8m-9m), vault settlements at the 15m, 20m, 25m section are 21.9mm, 6.7mm and 2.8mm, respectively. The surrounding rock at 35m to 50m section is less or almost unaffected. During the excavation step (10m-11m excavation step), the vault settlements at 15m, 20m, 25m, 30m sections are 35.3mm, 10.7mm, 5.4mm and 3.0mm, respectively. The surrounding rocks at 40m-50m sections are less or almost unaffected.

2. The surrounding rock within 5m of the front of the tunnel face is greatly disturbed by excavation. The deformation of the 15m, 20m, and 25m sections at 5m from the tunnel face accounts for 33.3%, 40.2%, and 27.2% of the total cumulative deformation, respectively.

### 6. Conclusions

Based on the numerical calculation methods, the stability of tunnel in benching tunneling method under various surrounding rock strength reduction coefficients was analyzed. The following main conclusions can be drawn from this paper:

1. The worse the grade of the surrounding rock, the greater the amount of advance deformation of the tunnel. And also, the deformation after the completion of the initial support construction accounts for a larger proportion of the whole deformation, resulting in two types of failures in stability in the course of benching tunneling construction of the large section tunnel in the weak and broken surrounding rock: 1) for tunnels with poor surrounding rock (from grade IV to V) may experience instability of the supporting structure or tunnel face, in which the instability of the supporting structure can be controlled by strengthening the supporting structure parameters; and 2) when the surrounding rock is very poor (grade V), tunnels may experience instability of the tunnel face, which should be controlled by advanced support measures.

2. With the surrounding rock grade which results in tunnel face instability, the impact distance of the upper bench excavation of the tunnel is about 20m, and the advanced deformation accounts for about 40% of the total deformation. The deformation and failure of the surrounding rock at the unstable section develop gradually from the vault down to the foot of the wall, and the amount of deformation at vault,
arch, arch foot, side wall and the foot of the wall decreases in sequence. The surrounding rock collapses at about 4.5m in front of the tunnel face and about 1.5m above the arch.

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References
[1] Zhang Junru, Wu Jie, Yan Congwen, et al. construction technology of super-large section of highway tunnels with four or more lanes in China[J]. China Journal of Highway and Transport, 2020, 33(01): 14-31.
[2] Lunardi p. Design and construction of tunnels: analysis of controlled deformation in rocks and soils(ADECO-RS)[M]. Springer: [s. n.], 2008
[3] Xia Runhe. Study on Construction Technology of Large Arch Foot andbenching tunnelling method for Large Section Railway Tunnel in Soft Surrounding Rock Railway[J]. Standard Design. 2010(04): 78-81.
[4] Xiong Chengyu, Liu Xiangyang. Discussion on efficient construction technology of soft surrounding rock in large-span tunnel[J]. Road Machinery & Construction Mechanization, 2019, 36(07): 73-78.
[5] Zheng Yingren, Qiu Chenyu, Zhang Hong, et al. Exploration of stability analysis methods for surrounding rocks of soil tunnel[J]. Chinese Journal of Rock Mechanics and Engineering, 2008(10): 1968-1980.