Static and dynamic analysis the influence of underlying tunnel construction on surrounding rock of existing tunnel-type anchorages

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Abstract: Due to the constraints of construction period and site conditions, it is necessary to study the influence of construction of underlying tunnel on the surrounding rock of the existing tunnel-type anchorages. Through static excavation and dynamic blasting, this paper systematically analyzes the influence of underpass tunnel construction on the surrounding rock of the tunnel-type anchorages from the aspects of stress, deformation, plastic zone, velocity and loose circle of the surrounding rock. The results show that the effect of excavation of underlying tunnel on the maximum principal stress around the tunnel-type anchorages is small. The maximum relative displacement of the tunnel-type anchorages is smaller than the allowable relative displacement of the code. The distribution of plastic zone around the tunnel-type anchorages is not affected by the excavation of underlying tunnel. When underlying tunnel blasting, the peak velocity of monitoring points of the top arch in each section of the right underlying tunnel is largest. From the area closer to the explosion source to the far area, the reduction of peak velocity in all directions is large. Based on the elasto-plastic theory, Hoek-Brown strength criterion is used to calculate the thickness of the loose circle.

1. Introduction
Tunnel-type anchorage is one of the key bearing structures of a suspension bridge. Its overall stability and stress state directly affect the safety and normal use of the suspension bridge [1]. In the shallow part of the bank slope, there are highway tunnels passing through the lower part. Due to the constraints of construction time, the upper tunnel-type anchorage will be built before the lower tunnels. In this case, the construction of underlying tunnels will have effect on the surrounding rock of existing tunnel-type anchorage, which determines the long-term operation safety of the suspension bridge. Therefore, it is of great significance to study the influence of underlying tunnel construction on the surrounding rock of the existing tunnel-type anchorage.

For analysis of the impact of construction of the underpass tunnel on the existing building or structure, Qiu [2] systematically proposed the concept of the near-construction of underground engineering by analyzing and summarizing the relevant research results; Qian et al. [3] carried out numerical simulation and combined with some on-site monitoring data to analyze the variation of the mechanical characteristics of the tunnel structure during the whole process of construction and operation of the multi-layer cross tunnel in soft soil layer. However, the current researches on the impact of the excavation of the underlying tunnel on the surrounding rock of the existing tunnel-type anchorage are relatively rare [4, 5].
The study of the influence of blasting vibration on tunnels mainly includes two aspects: one is the influence of blasting vibration on adjacent tunnels, and the other is the effect of blasting vibration on the tunnel itself. This paper mainly studies the first one. Many people have done a lot of research on the influence of blasting vibration on the surrounding rock of existing tunnels. For example, Xie [6] studied the dynamic response of tunnel blasting construction to existing tunnels; Ye et al. [7] combined with the Nanshanxia tunnel crossing the Qiancangshan tunnel engineering to monitor the vibration impact of the close-cross tunnel blasting construction on the existing tunnel. The above research results provide a good reference for the research in this paper. However, the above-mentioned literatures focused on static excavation analysis or dynamic blasting analysis, the static and dynamic analysis of the influence of underlaying tunnel on the surrounding rock of the existing tunnel-type anchorage has not been comprehensively studied. Therefore, in this paper, numerical and theoretical methods are used to systematically study the effects of construction of underlaying tunnel on the surrounding rock of existing tunnel-type anchorages.

2. Engineering situation and Numerical Model
The stratum of the bridge area is mainly permian volcanic rocks, pyroclastic rocks, and stratum of quaternary. The rock types mainly include dense massive basalts, amygdaloidal basalts, and tuffs. The spatial location relationship between tunnel and tunnel-type anchorages is shown in Fig. 1.

![Figure 1: Geological vertical section of right bank and plane relationship diagram between tunnel and tunnel-type anchorages](image1)

The tunnel is a centerless arch tunnel. The IV-level surrounding rock of the right tunnel is constructed by the reserved core soil with three-step subsection excavation method, and the left tunnel is constructed by the three-step method. Taking into account the geological structure and geotechnical characteristics of the bank slope, the main geotechnical considerations in the numerical analysis process are strongly weathered basalt, moderately weathered basalt, and tuff rocks. The three-dimensional geomechanical model is shown in Fig. 2.

![Figure 2: Three-dimensional geomechanical model](image2)

3. The influence of underlying tunnel excavation on surrounding rock of tunnel-type anchorages

3.1 Research scheme
Due to the constraints of the construction period, in order to ensure the stability of the surrounding rock of the tunnels and the tunnel-type anchorages, when the two are located at the same mileage, the nearest space distance of the tunnel faces of the two should exceed 20m. At this time, the right tunnel-type anchorage has been excavated, and tunnel face of the left tunnel-type anchorage is excavated to 31.3m front of anchoring concrete-plug. According to the construction requirements and plan of the project, the right tunnel has been constructed before the overlying tunnel-type anchorages. Therefore, the excavation of the right tunnel does not affect the tunnel-type anchorages, but the left tunnel will. In order to study the influence of the excavation of the left tunnel on the surrounding rock of the tunnel-type anchorages, the construction design considered is arranged. The left tunnel is excavated from 160m to 222.72m (this position corresponds to the rear end face of the back anchorage of the tunnel-type anchorages) for a total of 6 steps. The first 5 steps are excavated 10m for each step, and the last step is to excavate 12.72m. For each step, the deformation of each monitoring point of the
sections (such as section 1-1, 2-2, 3-3, 4-4 and 5-5) of the left tunnel-type anchorage and the corresponding plastic zone distribution are monitored. The arrangement of the sections and the monitoring points are shown in Fig. 3, wherein the monitoring points 1, 2, 3 and 4 are the midpoint of the left wall, the midpoint of the top arch, the midpoint of the right wall and the midpoint of the bottom of each section, respectively.

![Figure 3 Layout of monitoring points of each section of left tunnel-type anchorage](image)

3.2 Analysis of results

3.2.1 Analysis for stress field

In the excavation process of the left tunnel, the distribution of the maximum principal stress of the surrounding rock at the 1-1 section of the left tunnel-type anchorage is shown in Fig. 4. It can be seen from Fig. 4 that the maximum principal stress of the surrounding rock of the tunnel-type anchorage at the section 1-1 is almost constant with the excavation of the left tunnel. The maximum principal stress of the anchorage at the section 1-1 is increased from 13.52 MPa to 15.38 MPa. The local tensile stress at the tunnel inversion at the 1-1 section of the left tunnel-type anchorage is reduced from 45.058 KPa to 12.424 KPa. Therefore, the excavation of the left tunnel has little effect on the maximum principal stress at the section 1-1 of the left tunnel-type anchorage.

![Figure 4 Chang of stress field at section 1-1 of left tunnel-type anchorage](image)

3.2.2 Analysis for displacement field of tunnel-type anchorages

In the excavation process of the left tunnel, the displacement field of the surrounding rock at the 1-1 section of the left tunnel-type anchorage is shown in Fig. 5. The total deformation curve of each monitoring point of the 2-2 section, 3-3 section, and 5-5 section of the left tunnel-type anchorage is shown in Fig. 6. It can be seen from Fig. 5 and Fig. 6 that the excavation of the left tunnel has little influence on the displacements of the monitoring points of the left tunnel-type anchorage, and the influence of the displacements of different monitoring points at different sections is different. The largest change in displacement is the No. 1 monitoring point of the 5-5 section, and the maximum variation is about 2.83 mm. The displacement of different monitoring points at different sections will appear slightly with the left tunnel excavation. For example, the 1~4 monitoring points of the 2-2 section are reduced by 1.01mm, 0.99mm, 0.87mm and 0.93mm, respectively. The displacement reduction of No.1, No.3 and No.4 of the 3-3 section are about 1.28mm, 0.35mm and 1.44mm, respectively. While the displacements of the monitoring points of the 5-5 section are gradually
increased with the excavation of the left tunnel. According to Section 9.3.4 of the Technical Specification for Highway Tunnel Construction of the Ministry of Communications and Section 6.4.3 of the Technical Monitoring Specifications for Highway Tunnel Construction of Hubei Province, when the thickness of the cover layer is 50~300m, the relative allowable displacement of the surrounding wall of the IV surrounding rock of tunnel is 0.4%~1.2%, and the maximum relative displacement of the left tunnel-type anchorage is about 0.026%, which meets the requirements of the specification. It can be concluded that the excavation of the left tunnel has little influence on the displacements of surrounding rock of the left tunnel-type anchorage.

![Image](image1.png)

**Figure 5** Variation of displacement field at section 1-1 of left tunnel-type anchorage

![Image](image2.png)

**Figure 6** The total deformation curve of each monitoring point at different sections of left tunnel-type anchorage

### 3.2.3 Analysis for plastic zone

In the excavation process of the left tunnel, the distribution of the plastic zone of the left tunnel-type anchorage at 3-3 section is shown in Fig. 7. It can be seen from Fig. 7 that the excavation of the left tunnel has little effect on the distribution of the plastic zone of surrounding rock of the left anchorage at the 3-3 section, but the tuff interlayer above the tunnel-type anchorage has local shear failure due to the weak lithology. Most of the plastic zone around the tunnel-type anchorage appears in the surrounding rock above the dome arch and above the middle of the waist wall, and the plastic zone above the middle of the waist wall of the two anchorage penetrates. A small amount of plastic zone distribution occurs at the bottom of the tunnel-type anchorage. Therefore, it is recommended that during the construction process, especially after the excavation of the left tunnel-type anchorage to 200m, the support of the tuff interlayer and the middle part of the waist wall of the anchorage should be strengthened in time, and the area above the middle part of the waist wall and the top of the top arch should be monitored. Grouting and karst caves within 10m around the anchorage should be grouted to ensure the stability of the surrounding rock. During the excavation of the left tunnel, the total volume change of the plastic zone is shown in Fig. 8. It can be seen from Fig. 8 that the total volume of the plastic zone of surrounding rock increases with the excavation of the left tunnel, and the total volume of the plastic zone of the model is increased sharply after the excavation to the fifth step.
4. The influence of underlying tunnel blasting on surrounding rock of existing tunnel-type anchorages

4.1 Research scheme
In order to study the maximum impact of blasting of the left tunnel on the tunnel-type anchorages, the face of the left tunnel is excavated to the corresponding position at the bottom of the 31.3m front of anchoring concrete-plug of the tunnel-type anchorages. Fig. 9 shows the layout of monitoring points at the line between the left tunnel and tunnel-type anchorages. In addition, monitoring points at the top arch, side wall, and invert of 10m and 20m before and after blasting surface of the right tunnel are also arranged.

4.2 Calculation result analysis

4.2.1 The right tunnel
Figs. 10 and 11 shows the velocity time histories and velocity peaks of monitoring points of the top arch, side wall and invert of 10m and 20m before and after blasting surface of the right tunnel. The following conclusion can be drawn from the figure.

(1) The order of the peak velocity of the monitoring points in each section of the right tunnel is: top arch > invert > sidewall, which is more obvious in the y direction. Therefore, for the excavation of the upper bench, the vibration velocity of the top arch is mainly monitored.

(2) In the blasting direction and receding direction of the blasting surface, the peak velocity of the top arch, side wall and invert in the x direction increases as the excavation distance increases. The peak velocity of the invert and the side walls in the y direction is the same within 20m before and after the blasting surface, while the top arches in the direction of tunnel excavation and the backward direction is substantially reduced with the distance from the blasting surface increasing. Compared with the blasting surface, the peak velocity reduction reached around 80%; In the z-direction, the peak velocity of the top arch, side wall, and invert is the same as the distance in the driving direction, and the velocity reaches the maximum near the blasting surface excavation direction, while in the backward direction, it gradually decreases as the distance from the blasting surface increase; The total peak speed is similar to the peak speed in the z direction.
4.2.2 The lines between tunnel and tunnel-type anchorages and tunnel-type anchorages

Fig. 12 is the relationship between the peak velocity of the monitoring points and the distance the connecting between tunnel and tunnel-type anchorages and tunnel-type anchorages. The number of the monitoring points is compiled from the nearest to the far end of the tunnel, and the monitoring points 1-6 are the equidistant monitoring points of the connection between the tunnel and the tunnel-type anchorages, and the monitoring points 7-9 are the bottom and the middle of the 31.3m section of the anchoring concrete-plug, as shown in Fig. 4. It can be seen from Fig. 12 that the blasting seismic wave sharply attenuates in the area closer to the blast source and flattens in the far area, and the peak velocity reduction in all directions is relatively large. For example, the peak speed of the monitoring point 2 has been reduced to about 50% of monitoring point 1 and peak velocity of monitoring point 6 only accounts for about 13% of monitoring point 1. For the total peak velocity, the peak velocity of the monitoring points 7-9 of the anchoring concrete-plug does not exceed 1 cm/s. According to the provisions of the “Blasting Safety Regulations” (GB6722-2014), the particle velocity of the traffic tunnel is allowed to be 15-20cm/s. Therefore, the blasting vibration velocity is less than the requirements specified in the safety regulations for blasting.

Figure. 10 Relationship between peak velocity of monitoring points and distance in right underlaying tunnel

Figure. 9 Connection of underlaying tunnel and tunnel-type anchorages and monitoring points layout of anchorage

Figure. 11 Relationship between total peak velocity of monitoring points and distance in right underlaying tunnel

Figure. 12 Relationship between peak velocity of monitoring points and distance in the connecting between tunnel and tunnel-type anchorages and tunnel-type anchorages
4.2.3 Loose circle analysis

Based on the calculation of elastic-plastic theory, the Hoek-Brown criterion is used to calculate the thickness of rock loose ring [8]. Then the hoop plastic stress is

\[
\sigma_h = \frac{\sigma_{ci}}{m_i} \left[ m_i (1-a) \ln \left( \frac{r}{r_0} \right) + \left( m_i \frac{P_i}{\sigma_{ci}} + s \right) \right]^{\frac{1}{\gamma_{ci}}} - \frac{\sigma_{ci}}{m_i s + \sigma_{ci}} \left[ m_i (1-a) \ln \left( \frac{r}{r_0} \right) + \left( m_i \frac{P_i}{\sigma_{ci}} + s \right) \right]^{\frac{1}{\gamma_{ci}}}
\]

\[m_i = \exp \left( \frac{GSI-100}{28-14D} \right) m_i\]

\[s = \exp \left( \frac{GSI-100}{9-3D} \right)\]

\[a = 0.5 + \frac{1}{6} \exp \left( \frac{-GSI}{15} \right) - \exp \left( \frac{-20}{3} \right)\]

According to the definition of the loose zone: The tangential stress on the boundary of the loose zone is the initial stress \(P_0\), and the radius \(R\) of the loose zone can be found by entering the formula above. \(P_i\) is the supporting force; \(r\) is the distance from the center of the tunnel, i.e., the radius of the loose ring; \(r_0\) is the radius of the tunnel. \(GSI\) is a geological strength index, which can be solved according to the modified \(GSI\) method (formula (5)) and formula (6) [9]. \(D\) is the perturbation parameter and the range of value is 0-1, depending on the degree of disturbance of the in-situ rock mass by external factors.

\[GSI = RMR - 5\]

\[BQ = 170 \ln \left( 15 + 0.24RMR \right) / 5.7 - 0.06RMR\]

According to the “Code for Design of Highway Tunnels (JTGD70-2004)”, the ratio of the load released by surrounding rock and initial support in Type IV surrounding rock is from 60% to 80%, and the initial support takes 80% of the surrounding rock pressure. Surrounding rock pressure is

\[q = 0.45 \times 2^{3-1} \times \gamma \omega\]

\[\omega = 1 + i(B - 5)\]

Where \(S\) is the grade of surrounding rock, \(B\) is the tunnel width. When \(B > 5\) m, \(i\) takes 0.1.

Therefore, according to the basic parameters of the project, the radius of the top arch loose circle by substituting (1) can be calculated as \(R_0 = 7.54\) m.

The thickness of the loose ring in the roof is:

\[D = R_0 - r = 7.54 - 5.5 = 2.04\text{ m}\]

Therefore, relative to the spatial distance between the underlying and the existing tunnel-type anchorages, the loose loop thickness generated by the tunnel blasting construction has little effect on the surrounding rock of the existing tunnel-type anchorages.

5. Conclusion

(1) The influence of the excavation of the left tunnel on the maximum principal stress of the surrounding rock of the tunnel-type anchorages does not exceed 0.06 MPa. The most significant influence on the displacement value of each monitoring point of the right tunnel-type anchorage chamber is the No. 1 monitoring point of the 5-5 section with a variation range of about 2.83 mm. The maximum relative displacement of the tunnel-type anchorages is about 0.026%, which is less than the allowable relative displacement value of the specification and meets the specification requirements.

(2) The excavation of the left tunnel has little effect on the distribution of the plastic zone around the tunnel-type anchorages.

(3) For the blasting of the upper bench, the order of the peak velocity of the monitoring points in each section of the right tunnel is: top arch > invert > sidewall. Therefore, the vibration velocity of the top arch is mainly monitored. The peak velocity of the monitoring points in the anchoring concrete-plug does not exceed 1 cm/s, which is less than the requirements specified in the safety
Based on the elasto-plastic theory and the Hoek-Brown strength criterion, the thickness of the loose ring of the top arch was calculated to be 2.04m. Therefore, blasting of underlaying tunnel has little effect on the surrounding rock of tunnel-type anchorages. The results obtained by numerical methods and theoretical analysis methods are the same. It indicates that the numerical method and the theoretical analysis method are very good analytical methods when preliminary analysis the effect blasting of underlaying tunnels on the surrounding rock of existing tunnel-type anchorages.

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