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The proper width of the intermittent trough for tunnel enlarging

Yi-Zhong Tan [1,2], Yuan-Xue Liu [*1,2] and Zhong-You Li [1,2]

[1] *Department of Civil Engineering, Logistical Engineering University, Chongqing, China*
[2] *Chongqing Key Laboratory of Geomechanics & Geoenvironmental Protection, Chongqing, China*

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Abstract. As the traffic increased, the original capacity of the tunnel has been unable to meet the needs, so it must be expanded. Based on the features of tunnel, the intermittent trough method must be supposed for tunnel enlarging. Under the situation on the buried deep of the tunnel, it could be used the reasonable arch axis model to descript the past covered rock pressure for mechanism calculating of self-bearing arch. Then establish the three-arch combination effectible model for the analysis which is relied on the tunneling enlarging to Chongqing Yu-Zhou tunnel. It has determined the proper width for the intermittent trough in shallow buried tunnel enlarging.

Keywords: tunnel enlarging; intermittent trough; arch effect; the reasonable arch axis; combination arches

1. Introduction

Under the reconstruction of the existing railway lines, the subsistent tunnels on the line must be simultaneously transformed. Because of the limited conditions of geological and construction, it must be enlarged on the basis of the existed tunnel instead of creating a new double track. The enlarging tunnel section could be increased to meet the increasing traffic demands (Hu and Huang 2009). Currently, the research on the extending of the existed tunnel has seen little, especially for the theory of tunnel enlarging. So study on the existed tunnel enlarging is particularly important. Then Kim *et al*. (2006) has thought that the open-TBM, pre-excavating method by TBM prior to enlarge tunnel by blast, is more suitable than only a drill and blast method or shield TBM after considering all conditions. Mashimo and Ishimura (2003) compiled almost 40 case histories of tunnel enlargements, which allowed for traffic during construction. The writers also conducted three-dimensional analyses and field observations on different construction methods. Hiroshi Dobashi *et al*. (2006) has described the construction of the cut and open of a part of steel segments installed in two large dimensional shield tunnels over a total length of 100 meters using the non-open-cut method. In China, Zhu (2009) has generalized the extension of construction method in Ban-Jia Mountain which lines on Da-Cheng railway. Due to this method, it could prevent the old tunnel line not to be entirely slumping effectively and ensure the tunnel line's sinking should be control in allowable range. However, these scholars have raised a number of ways and means for expanding, but they didn't formulate the theory for loading mechanics. So these study results

*Corresponding author, Professor, Ph.D., E-mail: 357202780@qq.com

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are not so much more universal. Therefore, it is significant to study on the intermittent trough for tunnel enlarging and important to draw a reasonable parameter.

2. Rationality of the intermittent trough in tunnel enlarging

Due to the well geological condition, the surrounding rock could make full use of their own strength to control surrounding rock's deformation in constructions. Presently, many tunnels' enlarging methods are similar to the methods of building a new tunnel, that is to say, these tunnels are excavated step by step, as well as only for 1~2 m per day. As a result, the restricted time limited and increased cost must be faced austerely. To enlarge a tunnel in hard rock, according to the need of the actual requirement, you may assume the method of the jump cutting and interval blasting, instead of the approach of building a new tunnel. The intermittent trough method could be cut in multiple-stage at the same time. Considering long-term stability of the old tunnel, the no cutting parts of the surrounding rock and lining, which can support the surround rock of the arch crown makes the role of a column. The dome of the surrounding rock and rock pillars can work together to form a support structure. When the old tunnel was cut, the rock-soil mass will produce uneven settlement together with the soil enwrapped (Li *et al*. 2006). So it could be generated which is called the arch effect in a certain range of rock and soil. Therefore, the lengthwise intermittent trough method of the tunnel enlarging excavation is reasonable.

3. Study on the proper width of the intermittent trough

3.1 Analysis of the arch effect

3.1.1 Establish the combinated model of arches

Fig. 1 is the facade and elevation of intermittent trough method, where the over burden depth of the old tunnel is H, the recycling footage is L, the original length of tunnel section is l'. With the increasing of the cutting span, the arch effect is more and more obvious, until the height of the rock arch arriving the surface, and the enwrapped action which is created by non-uniform settlement will gradually disappear. The pre-existing arch will slowly degenerate until the soil is damaged thoroughly by cutting-slippage. Then the arching cross-cut should be the maximum width. When the both sides were cut, all of the overlying rock pressure of soil arching must transfer to the surrounding rock, then pass on to the old tunnel lining. So the research on the cycle footage of intermittent trough can be figured out by the principle of soil arch effect. After the extension excavation, the section still keep the form of arch section, so the section of the intermittent trough is covered by series of arches, it can be taken for a arch face. That is to say, after intermittent excavating, the mother-rock could adjust the state automatically, then a three-dimension arch body is formed, and the whole arch face back and front must directly effect on the mother-rock till arriving to the arch springing of the new tunnel section. Then it is found that the left and right of the rock arch can directly act on the liner of the original tunnel. The oval-throwing arch face can transfer the horizontal confining pressure to the no cutting liner of the original tunnel, which must bear all the pressures from the two arch pillars. So in order to be easy calculation, the maximum span of the department can be selected as the research object for analysis. Then the calculation could be simplified for a combination arch model by three arches. So it could be

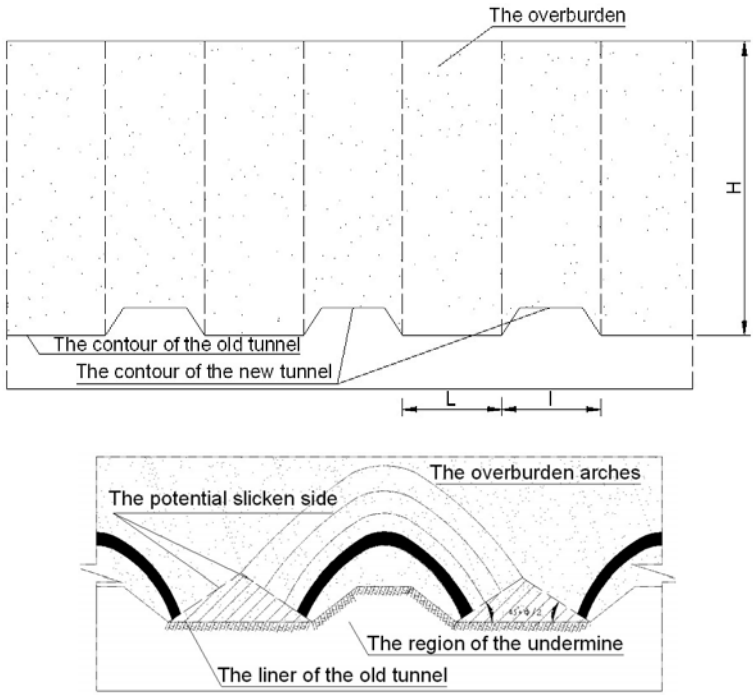


Fig. 1 The facade and elevation of intermittent trough method

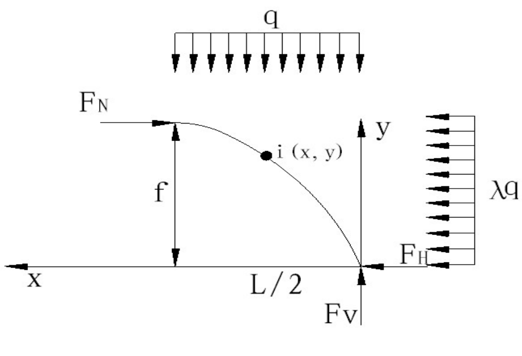


Fig. 2 The calculation model of rock arch

assumed by the rock arch conditions (Yang *et al*. 1994, Hou *et al*. 2006, Dong *et al*. 2009, Zhou and Pu 1999) as follows: (a) The rock arch which was formed by enlarging can apply on the overlying rock and the old concrete lined. So the arch feet of the rock arch must be existed and it must being directly or indirectly loading on the concrete lined. Owning to the control of the rock's anti-shear strength and the lateral pressure, it must form a stability arch feet (Jia *et al*. 2003); (b) It is considered that the stress of the old lined could not be impacted in enlarging, and the covering earth pressure q is the uniform distributed loading, where except to the deadweight of the rock arch. The research object is the arch ring which is the maximum span of the limited situation. For retaining the security value, the tensile stress can be ignored. So the rock arch meets the assumption of the reasonable arch axis.

3.1.2 Analysis for rock arch stress

The calculation model of rock arch in Fig. 2 is established under the assumptions. The x axis gets on well with the tunnel's stretching direction, and the y axis is the direction of the arch rise. The arbitrary point of the axis coordinates is $i(x, y)$, the arch span is L, the rise of arch is f, the vertical uniformly distributed load is q, the lateral uniformly distributed load is λq.

According to the definition of the rational arch axis (Long *et al.* 1994), the arch axis equation can be expressed as

$$F_N y = \frac{qx^2}{2} + \frac{\lambda q y^2}{2} \tag{1}$$

Let $a^2 = \dfrac{F_N^2}{\lambda q^2}$, $b^2 = \dfrac{F_N^2}{\lambda^2 q^2}$, $F_N = \dfrac{qL^2}{8f} + \dfrac{\lambda q f}{2}$

Then the arch axis equations can be shown as an ellipse

$$\frac{x^2}{a^2} + \frac{(y-b)^2}{b^2} = 1 \tag{2}$$

According to the force and moment-equilibrium, the counterforce of the arch springing can be expressed as follows

$$F_V = \frac{qL}{2} \tag{3}$$

$$F_H = \frac{qL^2}{8f} + \frac{\lambda q f}{2} \tag{4}$$

In order to ensure the stability of the arch in horizontal direction, The F_H should be less than the maximum friction of the rock stratum, which could meet the limited state $F_H = F_V \tan\varphi$, then the comparison expression between L and f is

$$L^2 - 2fL\tan\varphi - 4\lambda f^2 = 0 \tag{5}$$

The maximum and the minimum principal stress at the arch springing are

$$\sigma_{1f} = \frac{qL}{2t\cos\varphi}, \quad \sigma_{3f} = q\sin\varphi + \lambda q\cos\varphi \tag{6}$$

The relationship between maximum and minimum principal stresses should meet the equality as follows

$$\sigma_1 = \sigma_3 K_P + 2c\sqrt{K_P} \tag{7}$$

Where $K_P = \tan^2\left(45^\circ + \dfrac{\varphi}{2}\right)$.

Substituting Eq. (6) into Eq. (7), the thickness of the rock arch t is obtained

$$t = \frac{qL}{Aq + B} \qquad \text{where} \qquad \begin{cases} A = K_P(\sin 2\varphi + 2\lambda \cos^2 \varphi) \\ B = 4c\cos\varphi\sqrt{K_P} \end{cases} \tag{8}$$

The uniformly distributed force which is applied on the tunnel lining can be expressed as

$$q' = \frac{qL}{2t} \tag{9}$$

3.1.3 Analysis of the concrete-lined for the old tunnel

It is found that the cross-sections in most of the existing tunnel reconstruction engineering is always for birdcage and stalk. So based on the fact, the arch-section calculating model could be simplified for arch models in this paper. Also the horizontal thrust which is engendered by the intermittent trough must be transferred to the overlying of rock mass. So the concrete lining is not only suffered from the pressure of surrounding rock but also affected by the component of force in vertically. According to the loading format, the tunnel lining could be simplified for a structure of circular arc arch, which section is rectangular and must meet the assumption of the plane-section.

The single-side enlarged calculation model is a three times hypostatic structure and the force is in asymmetric distribution. The calculation is so complex, that could be solved by the force method of the superimposition. Then the original calculation model could be decompounded for two parts, which is shown in Fig. 3. The Part II could be calculated by the method of double-sided enlarging, where the part I could be calculated by the asymmetry loading of cardinal arch. Then

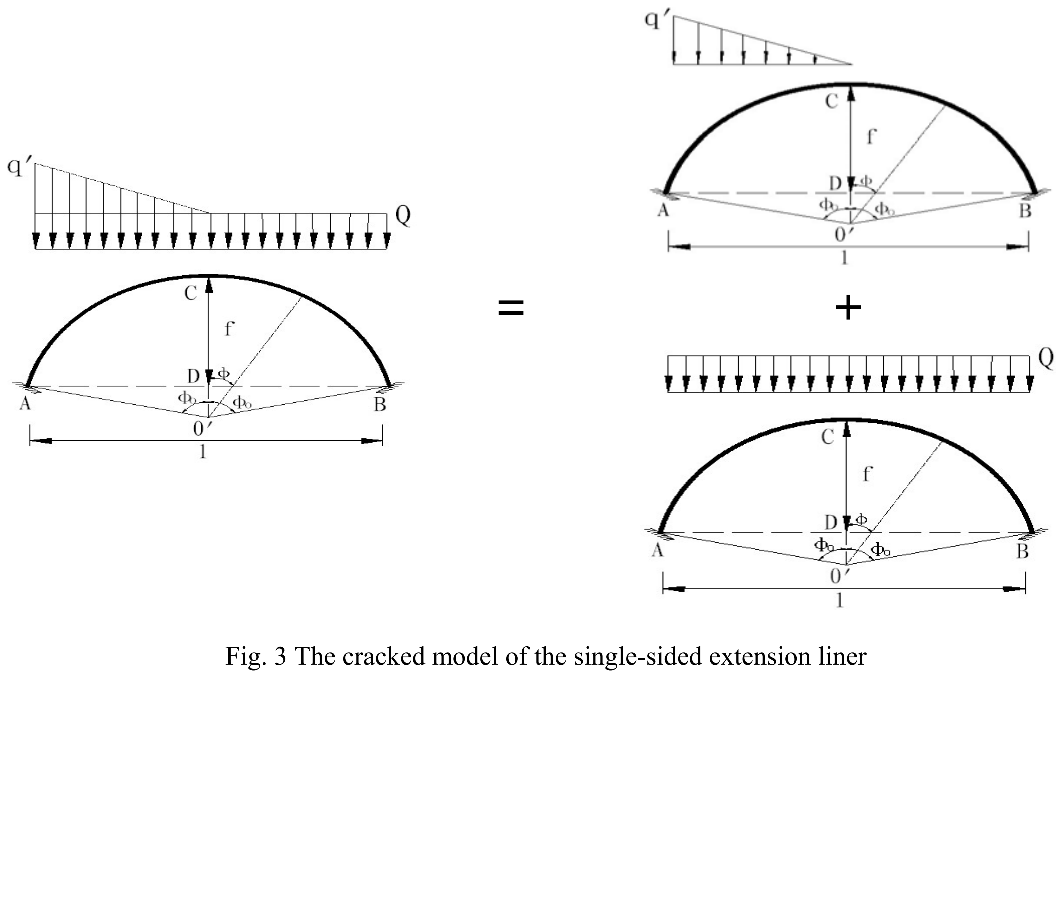


Fig. 3 The cracked model of the single-sided extension liner

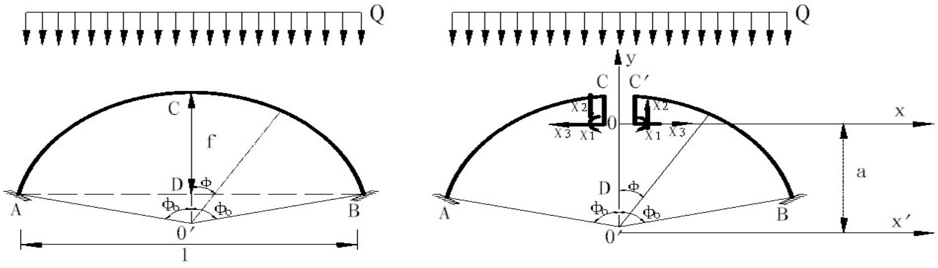


Fig. 4 The uniform load stress model of the liner

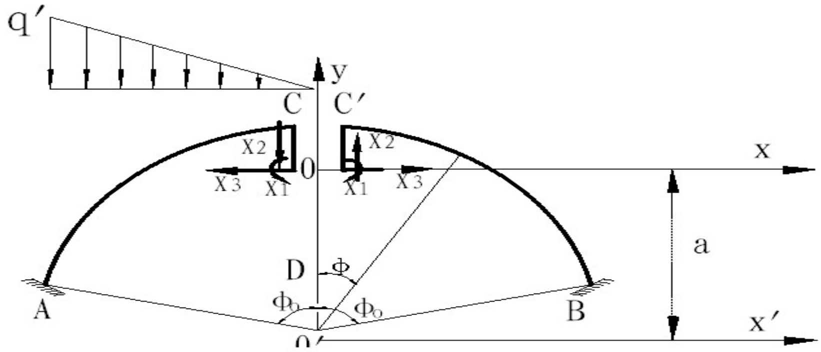


Fig. 5 The linearity stress model of the liner of single-sided extension

take the flexural torque valued for X_1, the axial force valued for X_2 and the shear valued for X_3. Because they are not in zero value, the equations can be built as follows.

$$\left.\begin{aligned}\delta_{11}X_1 + \Delta_{1P} = 0\\ \delta_{22}X_2 + \Delta_{2P} = 0\\ \delta_{33}X_3 + \Delta_{3P} = 0\end{aligned}\right\} \tag{10}$$

The calculating model in Fig. 4 is also a three times hypostatic structure. Because both of the structure and the stress are in a symmetrical condition, the shear value for X_3 is 0.

According to the simplified calculation of elastic center method, the establishing of the equation can be shown as

$$\left.\begin{aligned}\delta_{11}X_1 + \Delta_{1P} = 0\\ \delta_{33}X_3 + \Delta_{3P} = 0\end{aligned}\right\} \tag{11}$$

Fixing the site of the elastic center (Bradford *et al.* 2002)

$$a = \frac{R\sin\phi_0}{\phi_0} \tag{12}$$

According to the mechanics, it could be determined by these parameters for

$$\delta_{11} = \frac{2R\phi_0}{EI} \qquad \delta_{33} = \frac{2R^3}{EI}\left(\phi_0 - \frac{2\sin^2\phi_0}{\phi_0} + \frac{\sin 2\phi_0}{2}\right)$$

$$M_P = -\frac{QR^2}{2}\sin^2\phi \qquad N_P = QR\sin^2\phi$$

$$\Delta_{1P} = \frac{-QR^3}{EI}\left(\frac{\phi_0}{2} - \frac{\sin 2\phi_0}{4}\right) \qquad \Delta_{3P} = \frac{-QR^4}{EI}\left(\frac{\sin\phi_0}{2} - \frac{\sin\phi_0 \sin 2\phi_0}{4\phi_0} - \frac{\sin^3\phi_0}{3}\right)$$

Then take these previous parameters into Eq. (10), it could be obtained

$$X_1 = QR^2 \frac{(2\phi_0 - \sin 2\phi_0)}{8\phi_0} \tag{13}$$

$$X_3 = QR\left(\frac{\sin\phi_0}{2} - \frac{\sin\phi_0 \sin 2\phi_0}{4\phi_0} - \frac{\sin^3\phi_0}{3}\right) \bigg/ 2\left(\frac{\phi_0}{2} - \frac{\sin^2\phi_0}{\phi_0} - \frac{\sin 2\phi_0}{4}\right) \tag{14}$$

$$\overline{M}_{1L} = \overline{M}_{1R} = 1 \qquad \overline{M}_{2L} = -R\sin\phi$$

$$\overline{M}_{2R} = \sin\phi \qquad \overline{M}_{3L} = \overline{M}_{3R} = R\left(\frac{\sin\phi_0}{\phi_0} - \cos\phi\right)$$

$$\delta_{11} = \frac{2R\phi_0}{EI} \qquad \delta_{22} = \frac{R^3}{EI}\left(\phi_0 - \frac{\sin 2\phi_0}{2}\right)$$

$$\delta_{33} = \frac{2R^3}{EI}\left(-\frac{\sin^2\phi_0}{\phi_0} + \frac{\sin 2\phi_0}{4} + \frac{\phi_0}{2}\right) \qquad M_{PL} = -\frac{q'R^2}{6}\sin^2\phi$$

$$M_{PR} = 0 \qquad N_{PL} = -\frac{qR\sin^2\phi}{2}$$

$$N_{PR} = 0 \qquad \Delta_{1P} = -\frac{q'R^2}{6EI}\left(\frac{\phi_0}{2} - \frac{\sin 2\phi_0}{4}\right)$$

$$\Delta_{2P} = -\frac{q'R^2}{6EI}\left(-\frac{2}{2} + \cos\phi_0 - \frac{\cos^3\phi_0}{3}\right) \qquad \Delta_{3P} = -\frac{q'R^4}{6EI}\left(\frac{\sin\phi_0}{2} - \frac{\sin\phi_0 \sin 2\phi_0}{4\phi_0} - \frac{\sin^3\phi_0}{3}\right)$$

Take the previous parameters into these equations, it could be obtained

$$X_1 = q'R^2\left(\frac{\phi_0}{2} - \frac{\sin 2\phi_0}{4}\right) \Big/ 12\phi_0 \tag{15}$$

$$X_2 = -q'R\left(\frac{2}{3} - \cos\phi_0 - \frac{\cos^3\phi_0}{3}\right) \Big/ 6\left(\phi_0 - \frac{\sin 2\phi_0}{2}\right) \tag{16}$$

$$X_3 = q'R\left(\frac{\sin\phi_0}{2} - \frac{\sin\phi_0 \sin 2\phi_0}{4\phi_0} - \frac{\sin^3\phi_0}{3}\right) \Big/ 12\left(-\frac{\sin^2\phi_0}{\phi_0} + \frac{\sin 2\phi_0}{4} + \frac{\phi_0}{2}\right) \tag{17}$$

3.2 Calculate the proper width of the intermittent trough

The intermittent trough enlarging which is involving the combined effects of the three-arch should meet the following conditions. When the force exerts on the original concrete arch which passes to the feet of the rock arch, the concrete arch don't have the destabilizing effect. That means the concrete arch can't be damaged by tensile and compressive force. By the analysis of three-arch stress which could be combined to calculate by the anterior job. The rock arch is formed later than the stress, the arch body must be stable itself, so the stability problems of the rock-arch could not be discussed (Xu *et al.*, 2001). Due to the stability of surrounding pressure, the overlying rock-arch could be calculated. In that way, the bending moment and axial force which is formed by the arch-rock pressure can be delivered respectively.

$$\begin{aligned} M_{\phi L} = {} & \frac{(6Q+q')R^2(2\phi_0 - \sin 2\phi_0)}{48\phi_0} + \frac{(6Q+q')R\left(\frac{\sin\phi_0}{2} - \frac{\sin\phi_0 \sin 2\phi_0}{4\phi_0} - \frac{\sin^3\phi_0}{3}\right)(R\cos\phi - a)}{12\left(\frac{\phi_0}{2} - \frac{\sin^2\phi_0}{\phi_0} + \frac{\sin 2\phi_0}{4}\right)} \\ & - \frac{(3Q+q')R^2}{6}\sin^2\phi_0 - \frac{q'R^2\left(\frac{2}{3} - \cos\phi_0 - \frac{\cos^3\phi_0}{3}\right)\sin\phi}{6\left(\phi_0 - \frac{\sin 2\phi_0}{2}\right)} \end{aligned} \tag{18}$$

$$\begin{aligned} M_{\phi R} = {} & \frac{(6Q+q')R^2(2\phi_0 - \sin 2\phi_0)}{48\phi_0} + \frac{(6Q+q')R\left(\frac{\sin\phi_0}{2} - \frac{\sin\phi_0 \sin 2\phi_0}{4\phi_0} - \frac{\sin^3\phi_0}{3}\right)(R\cos\phi - a)}{12\left(\frac{\phi_0}{2} - \frac{\sin^2\phi_0}{\phi_0} + \frac{\sin 2\phi_0}{4}\right)} \\ & - \frac{QR^2}{2}\sin^2\phi - \frac{q'R^2\left(\frac{2}{3} - \cos\phi_0 + \frac{\cos^3\phi_0}{3}\right)\sin\phi}{6\left(\phi_0 - \frac{\sin 2\phi_0}{2}\right)} \end{aligned} \tag{19}$$

$$N_{\phi L}=\frac{(6Q+q')R\left(\frac{\sin\phi_0}{2}-\frac{\sin\phi_0\sin 2\phi_0}{4\phi_0}-\frac{\sin^3\phi_0}{3}\right)\cos\phi}{12\left(\frac{\phi_0}{2}-\frac{\sin^2\phi_0}{\phi_0}+\frac{\sin 2\phi_0}{4}\right)} -\frac{(2Q+q')R\sin^2\phi}{2}+\frac{q'R^2\left(\frac{2}{3}-\cos\phi_0+\frac{\cos^3\phi_0}{3}\right)\sin\phi}{6\left(\phi_0-\frac{\sin 2\phi_0}{2}\right)} \tag{20}$$

$$N_{\phi R}=\frac{(6Q+q')R\left(\frac{\sin\phi_0}{2}-\frac{\sin\phi_0\sin 2\phi_0}{4\phi_0}-\frac{\sin^3\phi_0}{3}\right)\cos\phi}{12\left(\frac{\phi_0}{2}-\frac{\sin^2\phi_0}{\phi_0}+\frac{\sin 2\phi_0}{4}\right)} +QR\sin^2\phi+\frac{q'R^2\left(\frac{2}{3}-\cos\phi_0+\frac{\cos^3\phi_0}{3}\right)\sin\phi}{6\left(\phi_0-\frac{\sin 2\phi_0}{2}\right)} \tag{21}$$

According to the code (JTG D70-2004 2004), if the rectangle concrete-lined section could meet the term $e_0 \le 0.20h$, it could be considered that the bearing capacity is controlled by the compression strength. No more calculating for stretching resistance and the formula of the compression strength is $KN/bh \le \varphi\alpha R_a$. Then the maximal endogen could be determined by the liner. If the rectangle concrete-lined section could meet the term $e_0 \le 0.20h$, it could be considered that the bearing capacity is controlled by the tensile strength. No more calculating for compressive resistance and the formula of the compression strength is $KN/bh \le 1.75\varphi R_1 / (6e_0/h-1)$. Then the maximal endogen could be also determined by the liner.

If $e_0 \le 0.20h$, take Eqs. (15) and (16) into $KN/bh \le \varphi\alpha R_a$, the maximal length L for undermine can be solved by $M_\phi / I_z + N_\phi / A \le \varphi\alpha R_a / K$. So the minimum length of the preformed groove could be solved for twice of the thickness to the overlying rock arch ring.

If $e_0 \le 0.20h$, take Eqs. (15) and (16) into $KN/bh \le 1.75\varphi R_1 / (6e_0/h-1)$, the maximal length L for undermine can be solved by $N_\phi / A \le 1.75\varphi R_1 / K(6e_0/h-1)$. So the minimum length of the preformed groove could be also solved for twice of the thickness to the overlying rock arch ring.

4. One tunnel enlarging example in Chongqing area

Take an example of the tunnel enlarging and reconstruction in Chongqing Yu-zhou Road, and analyze the effects of the three-arch combination model for intermittent trough. The old Yu-zhou tunnel is a single cave of two lanes, where the clear span is 10 m, the clear height is 6.7 meters. The plain concrete liner is for birdcage and stalk and the concrete grade is C25. Where the vector height of the arc-arch is 2.8 meters, the thickness of the arch is 0.75 meters, the thickness of the

Table 1 Parameters of the country rock

The sort of the country rock	Young's modulus E / GPa	Poisson's ratio μ	Weight γ / kN·m^3	Cohesive strength c / kPa	Internal friction angle φ / (°)
IV-rock	1.8	0.20	22	681	32.8

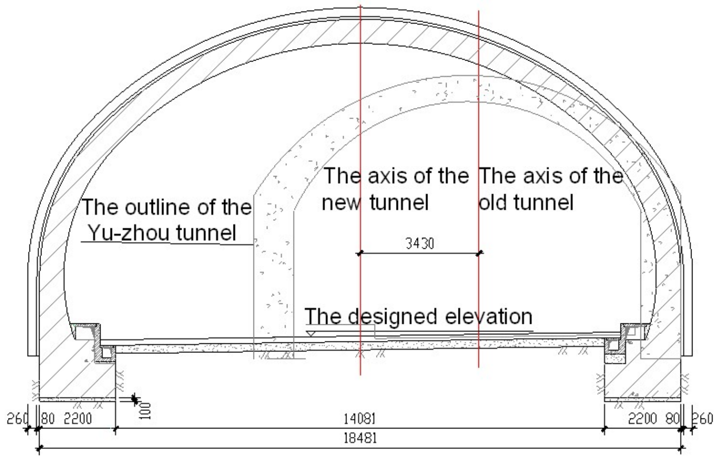


Fig. 6 The abridged general view of single-sided tunnel extension

side wall is 1.1 meters. After reconstructing, it becomes to be a tunnel of four-lane, where the clear span and height are respectively reached for 17.081 m and 8.482 m. the rebuilt tunnel section is shown in Fig. 6 below. The average depth is about 20.375 meters and the overlying rock stratum was mainly the weathered sandstone, the artificial filling and the diluvia layer. According to the scene investigation report, the parameters are shown in Table 1.

According to the given parameters, the stress of the liner could be calculated. Firstly, fix the relationship of the arch height and the arch span. Then it could be obtained the equation of f = 0.57 L by Eq. (5). Based on the Poisson's ratio in Table 1, the lateral pressure coefficient λ can be calculated, the value is 0.25. Then the uniformly distributed loading which is loaded on the rock arch can be calculated by the Terzaghi Theory (Terzaghi 1948, Zhu and Song 2007) available for shallow tunnel.

$$q = \frac{\gamma b}{\tan\varphi}\left(1 - e^{-\frac{h\tan\varphi}{b}}\right) = 26.1L\left(1 - e^{-\frac{0.696(10-0.57L)}{L}}\right) \tag{22}$$

Where $b = 0.5L + 0.57L\tan(45° - \varphi/2)$.

Under the foregone conditions, the radius R and the radius angle ϕ_0 could be solved.

$$R = \frac{l^2 + 4f^2}{8f} = 5.86m$$

$$\sin\phi_0 = l/2R = 0.8532, \quad \text{then} \quad \cos\phi_0 = 0.5216, \quad \phi_0 = 1.0221rad$$

Take the parameters into Eqs. (12) and (13), the flexural torque and axial force in arbitrary cross could be obtained.

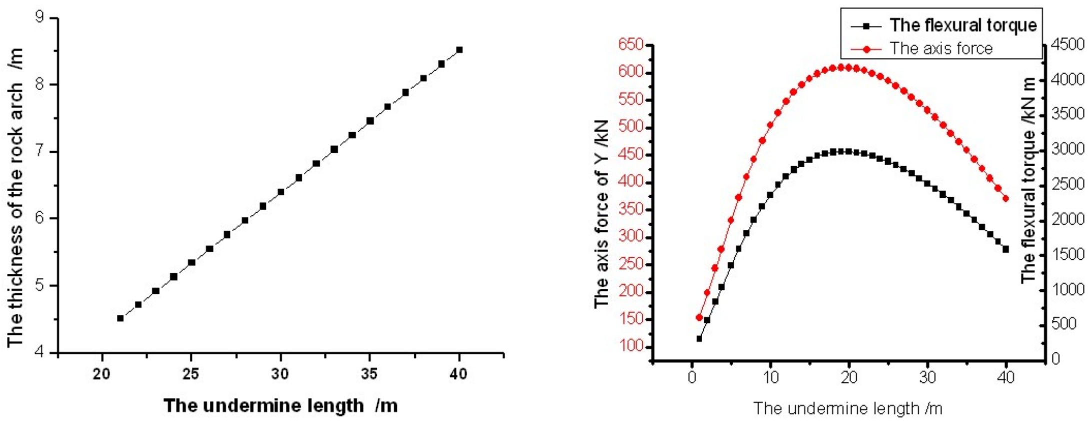
$$\left.\begin{aligned} M_{\phi L} &= 0.8079(6Q+q') + 4.5438(6Q+q')(0.8348-\cos\phi) \\ &\quad -5.7233(3Q+q')\sin^2\phi - 1.908q'\sin\phi \\ M_{\phi R} &= 0.8079(6Q+q') + 0.7754(6Q+q')(4.89-5.86\cos\phi) \\ &\quad -17.17Q\sin^2\phi - 1.908q'\sin\phi \\ N_{\phi L} &= 0.7754(6Q+q')\cos\phi + 2.93(2Q+q')\sin^2\phi + 0.3256q'\sin\phi \\ N_{\phi R} &= 0.7754(6Q+q')\cos\phi + 5.86Q\sin^2\phi + 0.3256q'\sin\phi \end{aligned}\right\} \tag{23}$$

According to the code (JTG D70-2004 2004), the not cutting surrounding rock pressure could be calculated firstly for

$$Q = q_{\text{Shallow}} = \gamma h\left(1-\frac{H}{B_t}\lambda\tan\theta\right) = 186.3\,kN/m^2 \tag{24}$$

Take these parameters into Eq. (8), the relationship of the undermine length and the arch thickness could be seen in Fig. 7(a). In this curve, it can be found that at a given buried deep the application of the adjacent rock pressure is very visible. It means that the relationship of arch thickness and the trough is linear. If the arch crown arrived at the ground surface, the arch effect also disappeared, at this time, there is not self-supporting of arch effect any more.

By the calculating, it is found that the maximum flexural torque and axial force are in the skew



(a) (b)

Fig. 7 (a) The relation of the arch-land and undermine footage; (b) The profile of the flexural torque and axes stress in arch springing

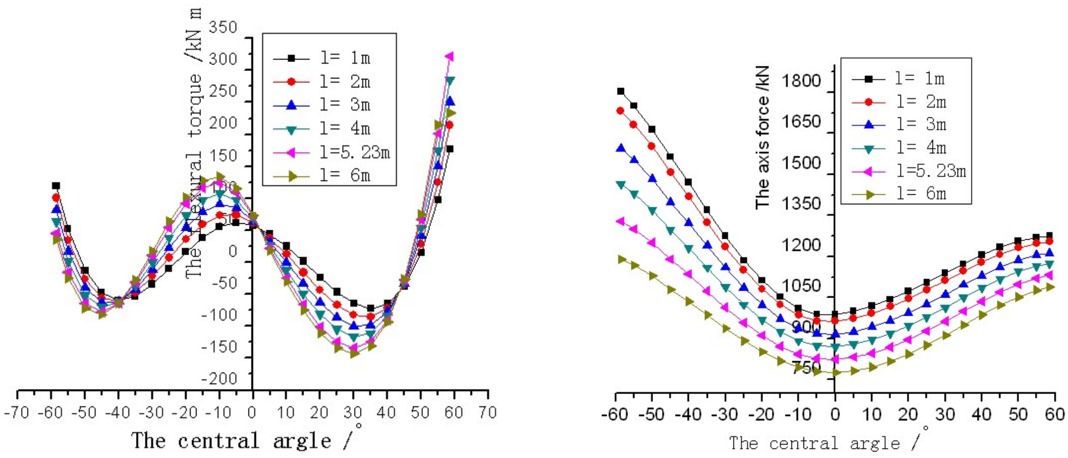


Fig. 8 The flexural torque section and axes stress in every cross-section of the liner

back, and the most dangerous sections are here. So the flexural torque and axial force which is in skew back can be calculated as the changing of the undermine length, the curves are shown in Fig. 7(b).

According to the calculation, the maximum eccentricity of the section is 0.193 meters < 0.20 h. So the maximum crushing stress of the lining can be calculated by the formula $KN \le \varphi \alpha R_a bh$. Take the assurance factor K=1.5 and take the parameters into the formula $M_\phi / I_z + N_\phi / A \le \varphi \alpha R_a / K$, then the result is 11.57 MPa. So when the maximum compression stress arrived at 11.57 MPa, the compression strength may in a limited station of safety. Then the undermine length could be calculated of 5.23 meters, at the same time the arch thickness are 1.11 meters. In order to avoid the superimposing effect of the arch springing for the two sides arch effect, it must be cut by the jumping grid cut slot method. The preformed groove length must satisfy to the relationship of $l' \ge 2t = 1.11 \times 2 = 2.22$ m. Considered to the convenient of the construction, it must retain a certain amount of security reserves. So both of the undermine length and reserved length can take in 5 meters appropriately. Then the bending moment and axial force of the cross sections after and before the limited station of the concrete lining can be drawn out in Fig. 8.

5. Conclusions

Based on the analysis, some useful conclusions can be seen hereinafter.

- Arch effect is a common phenomenon in geotechnical engineering, the analysis method of arching effect is simple and useful, and it is also proved that the calculation of the intermittent trough enlarging method is feasible. It can fully mobilize the rock arching effect. That is, with the increasing of undermine length, the arch thickness is also increasing until the arch crown reaching the surface. Then the undermine length must be the best, in theoretically.
- After calculation and analysis, the theoretical proper width of the intermittent trough is determined ultimately. Based on the project, the proper width of the intermittent trough is

5.23 m while the depth of burial is 20.375 m. So the old tunnel can be reserved the length of 5m to retain the lining in steady-going way, it can be divided into multiple operating segment construction at the same time. So it can be greatly shorten the duration and reduce the cost for tunnel enlarging, as well.

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