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## 隧道水平围岩压力计算方法

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**摘要:**分析了采用中隔壁法施工时隧道中隔壁的变形特性,研究了中隔壁变形和水平荷载之间的内在关系,提出了一种新的隧道水平围岩压力计算方法。采用结构力学分析理论,建立了中隔壁变形和水平围岩压力之间的关系,利用中隔壁变形监测数据,得到水平围岩压力。基于天恒山土质浅埋隧道V级围岩,采用谢家然法计算的水平围岩压力为88~145 kPa,采用新算法计算的水平围岩压力为110 kPa。其中,当围岩摩擦角为 $45^\circ$ 时,采用谢家然法计算的水平围岩压力为115 kPa,与采用新算法计算的水平围岩压力接近,两者相差5 kPa,验证了新算法的可行性。

**关键词:**隧道工程;围岩压力;中隔壁变形;水平荷载;压力计算方法

**中图分类号:**U451.2

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## Calculation method of horizontal surrounding rock pressure for tunnel

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**Abstract:** The deformation feature of middle wall for tunnel during construction was analyzed, the inner relationship between the deformation and the horizontal load was studied, and a new calculation method of horizontal surrounding rock pressure for tunnel was proposed. In the method, the analysis theory of structural mechanics was adopted to establish the relationship, in which the pressure was obtained by utilizing the monitoring deformations of middle wall. The pressure of V grade surrounding rock of shallow-buried soil tunnel in Tianhengshan is 110 kPa gained by the new method, and its range is 88-145 kPa gained by the Xie Jia-xiao method. When the friction angle of surrounding rock is  $45^\circ$  taken by the Xie Jia-xiao method, the pressure is 115 kPa and is close to the value computed by the new method, and their difference is 5 kPa, which proves that the new method is feasible. 1 tab, 10 figs, 15 refs.

**Key words:** tunnel engineering; surrounding rock pressure; deformation of middle wall; horizontal load; calculation method of pressure

## 0 Introduction

Surrounding rock pressure is one of basic key research topics in the field of tunnel engineering.

Its determination and calculation become quite difficult owing to the complexity and randomness of stratum nature, ground stress, boundary conditions and constructing process<sup>[1]</sup>. For a long

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time, the domestic and foreign scholars have adopted varied methods to conduct the relevant research, by which many valuable results are gained with respect to empirical formulas, theoretical calculations, practical measurements, model tests and back analysis.

Most representative empirical methods are MM theory<sup>[2]</sup>, Terzaghi theory<sup>[3]</sup>, Xie Jia-xiao method<sup>[4]</sup>, Barton method based on Q class rock classification<sup>[5]</sup> and Bieniawski method based on the rock classification of RMR system<sup>[6]</sup>, etc. These empirical methods, mostly adopting the limit equilibrium principle, have been widely applied to the surrounding rock pressure calculation and classification. Yang, et al utilized the limit equilibrium principle to calculate the surrounding rock pressure of shallow-buried tunnel according to Terzaghi destruction model<sup>[7-9]</sup>. However, whatever methods are adopted, e. g., the classic limit equilibrium principle or the limit analysis method, when the surrounding rock pressure of tunnel is calculated, the lateral pressure coefficient is still worked out according to Terzaghi empirical experience, which is subjective and over-generalizing. Consequently, the pressure calculation is defective.

In the theoretical calculation, the circular tunnel model is mainly adopted, and the surrounding rock pressure is calculated based on the ideal plastic medium. The classic and representative formulas include Caquot formula, Fenner formula, revised Fenner formula and Kastner formula<sup>[2]</sup>. In recent years, Jiang, et al considered the strain-softening calculation method of surrounding rocks and the calculation model of softening properties for materials<sup>[10-11]</sup>, which is an improvement and promotion of the classic formulas. Meanwhile, LU, et al also studied the analytic solutions of stress fields for the surrounding rocks of shallow-buried tunnels<sup>[12]</sup>. Nevertheless, due to the complexity and diversity of geological conditions, the obtainment of physical mechanics parameters of rock mass and constitutive equations is affected, which leads to

great constraints on the preciseness of the calculation result for the surrounding rock pressure.

The practical measurements include direct and indirect measurements, the former adopts the pressure cell to directly measure the surrounding rock pressure, the latter adopts the lining strain and the multi-point extensometer of tunnel to indirectly measure the surrounding rock pressure. Chen, et al adopted the pressure cell to measure the surrounding rock pressures of soil tunnels and stone tunnels<sup>[13-14]</sup>. However, the uncertainty exists in the measurement data due to the limitations of measurement technologies regarding above two instruments.

Based on size-reduced tunnel engineering, the model test is adopted to analyze the physical mechanics conditions and stress features of surrounding rocks of tunnels. The reaching conclusion is qualitative and has great error, so the test has obvious limitations.

The back analysis method is a new one to determine the surrounding rock pressure<sup>[1]</sup> and is developed in recent decades. In the method, the monitoring data, the finite element method and the boundary element method are adopted to back calculate the load of lining structure. In the back analysis prediction process, the constitutive equations are determined with regard to lining structure and surrounding rock. Nonetheless, the constitutive equations have also many influencing factors.

From above the discussions, it can be observed easily that the traditional calculation methods of surrounding rock pressure depend on the empirically calculated mechanics parameters of surrounding rock, which leads to the certain subjectivity and blindness of the calculation results. At present, although the existing calculation methods have been improved in some aspects, the questions for determining the constitutive equations of surrounding rocks and calculating the mechanics parameters do not be solved.

Authors have found the relationship between the middle wall deformation and the horizontal surrounding rock pressure of tunnel in field measurements, and propose a new calculation method of the pressure. The method is neither similar to the practical measurement method nor to the back analysis prediction method. It successfully avoids the trouble of determining constitutive equations for lining structure and surrounding rock.

## 1 Relationship between middle wall deformation and horizontal surrounding rock pressure

During the construction of large cross-section tunnel in poor strata, single side wall pilot method, CD construction method, CRD construction method and double side walls pilot method are usually adopted, and are illustrated in Fig. 1. The constructing methods are different in the excavated segment numbers and sequences, but they all adopt the middle wall. After the segment ① is excavated, the preliminary support and the temporary middle wall are constructed timely. The segment ② and the following constructing procedures are started only after the stabilization of the above

work. Before the segment ② is excavated, the excavated pilot of the segment ① becomes steady under the effect of preliminary support and temporary middle wall. At this time, the middle wall experiences the horizontal surrounding rock pressure from rock mass. After the segment ② is excavated, the pressure will be discharged and the middle wall experiences deformation. The following part is an example to analyze the relationship between the middle wall deformation and the horizontal rock pressure with CRD construction method.

After the excavating completion and support of the segment ① with CRD construction method, the structure reaches the basically steady mechanics state under the preliminary support and the temporary middle wall, as illustrated in Fig. 2.

At the moment, the middle wall stays in a steady state under the horizontal surrounding rock pressure of right rock mass. During the segment ② construction, the right rock mass of middle wall is dug out and the experienced horizontal surrounding rock pressure disappears, as illustrated in Fig. 3. The elastic deformation of middle wall under the horizontal rock pressure recovers after the surrounding rock pressure disappears, thus leading

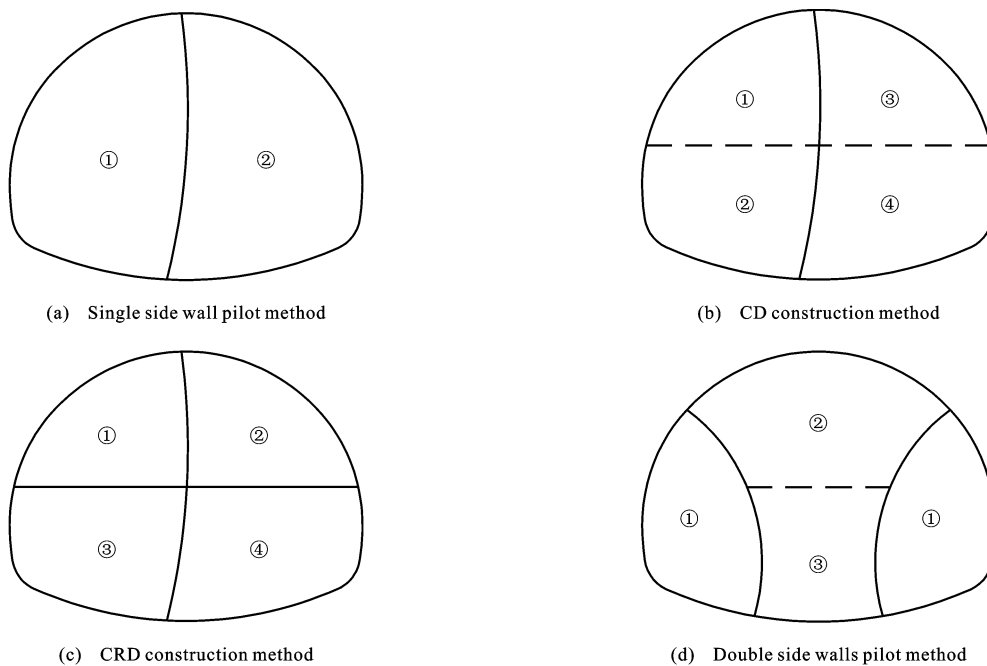


Fig. 1 Constructing methods

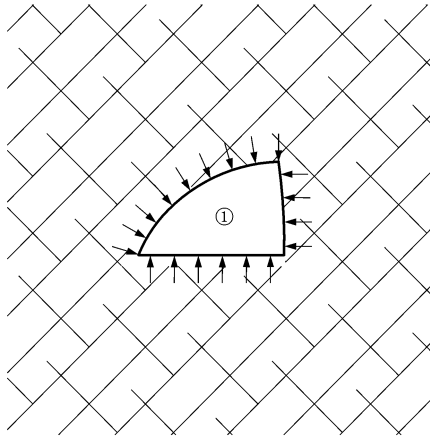


Fig. 2 Mechanics state of structure after stage ① being excavated

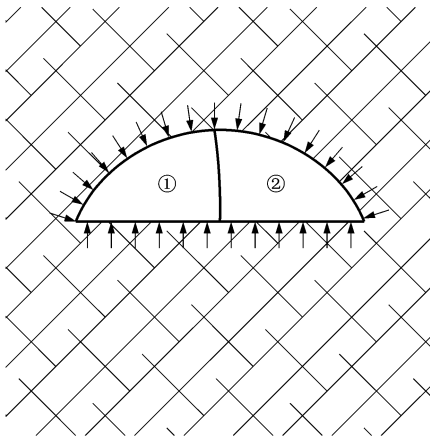


Fig. 3 Mechanics state of structure after stage ② being excavated

to the expanded deformation of middle wall.

Therefore, after the middle wall becomes steady in the excavation of segment ①, the expanded deformation has a close relation with the horizontal surrounding rock pressure of rock mass. Via the simplified calculation of middle wall structure model, the deflection caused by the effect of middle wall on the horizontal surrounding rock can be used to compute the horizontal surrounding rock pressure of rock mass.

## 2 Mechanics model of middle wall structure

After the segment ① construction with CRD construction method, the preliminary support of arch and the temporary support of middle wall and inverted arch are constructed. For the temporary support of middle wall, welded with steel frames

of arches and temporary inverted arches, structural steel frame and shotcrete are adopted. Now, authors analyze the middle wall separately and establish the structural mechanics model.

### 2.1 Basic hypotheses

Before constructing the model, the following hypotheses are made:

(1) The middle wall is straight bar vertical to the horizontal surface.

(2) The middle wall is simplified as the rigid material via the equivalent stiffness.

(3) the weld points of the connection part of middle wall belongs to rigid joints.

(4) the horizontal surrounding rock pressures experienced by the middle wall belongs to uniform load.

### 2.2 Model establishment

The full length of middle wall is  $l$ , the elastic modulus is  $E$ , and the inertia moment is  $I$ . Its both ends are hold-down supports, the uniform load  $q$  acts on right side, the direction along the middle wall is  $x$ , and the direction vertical to the middle wall is  $y$ . The model is presented in Fig. 4.

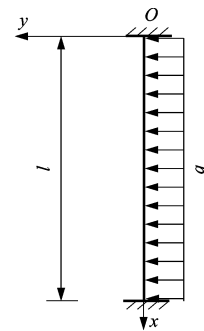


Fig. 4 Simplified model

### 2.3 Structural mechanics calculation

In Fig. 4, the model belongs to classic single-span statically indeterminate beam, and its methanics performances are calculated via the basic structure presented in Fig. 5. It has three redundant restraints, which are statically indeterminate for three times. The anti-rotating restraints  $X_1$  and  $X_2$  are removed from ends A and B, and the horizontal restraint  $X_3$  is removed from end B. The fundamental mechanics system of a simply supported beam is presented in Fig. 6. As the

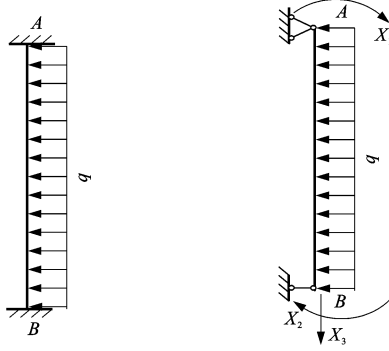


Fig. 5 Basic structure

Fig. 6 Basic mechanics system

fundamental system shall satisfy the corner angles at ends A and B, the horizontal displacement at end B equals zero, and the classic force method equations are presented as follows

$$\begin{cases} \delta_{11}X_1 + \delta_{12}X_2 + \delta_{13}X_3 + \Delta_1 = 0 \\ \delta_{21}X_1 + \delta_{22}X_2 + \delta_{23}X_3 + \Delta_2 = 0 \\ \delta_{31}X_1 + \delta_{32}X_2 + \delta_{33}X_3 + \Delta_3 = 0 \end{cases} \quad (1)$$

$$\delta_{11} = \delta_{22} = \frac{1}{EI} \cdot \frac{l}{2} \cdot \frac{2}{3} = \frac{l}{3EI} \quad (2)$$

$$\delta_{33} = \int_0^l \frac{\bar{M}_3^2}{EI} dx + \int_0^l \frac{N^2}{EA} dx = \frac{l}{EI} \quad (3)$$

$$\delta_{12} = \delta_{21} = -\frac{1}{EI} \cdot \frac{l}{2} \cdot \frac{1}{3} = -\frac{l}{6EI} \quad (4)$$

$$\delta_{13} = \delta_{31} = \delta_{23} = \delta_{32} = 0 \quad (5)$$

$$\Delta_1 = -\Delta_2 = \frac{1}{EI} \cdot \frac{2}{3} \cdot \frac{1}{8} ql^2 \cdot l \cdot \frac{1}{2} = \frac{ql^3}{24EI} \quad (6)$$

$$\Delta_3 = 0 \quad (7)$$

Where  $N$  is the internal force under  $X_3$  effect;  $A$  is the section area of middle wall model.

First, the bending moments  $\bar{M}_1$ ,  $\bar{M}_2$  and  $\bar{M}_3$  are provided under the separate effects of  $X_1 = 1 \text{ kN} \cdot \text{m}$ ,  $X_2 = 1 \text{ kN} \cdot \text{m}$  and  $X_3 = 1 \text{ kN}$  in Fig. 7(a)-(c). The bending moment  $\bar{M}$  under the single effect of the load on the fundamental structure is presented in Fig. 7(d). The total bending moment  $M$  is presented in Fig. 7(e). According to  $\bar{M}_1$ ,  $\bar{M}_2$ ,  $\bar{M}_3$  and  $\bar{M}$ , the coefficients  $\delta_{ij}$  and the free terms  $\Delta_i$  in Eq. (1) are calculated only considering the influence of bending deformation. Substituting the coefficients and the free terms calculated in Eqs. (2)-(7) into the Eq. (1), the equations are obtained as follows

$$\begin{cases} 2X_1 - X_2 + \frac{1}{4}ql^2 = 0 \\ -X_1 + 2X_2 - \frac{1}{4}ql^2 = 0 \\ \frac{lX_3}{EA} = 0 \end{cases} \quad (8)$$

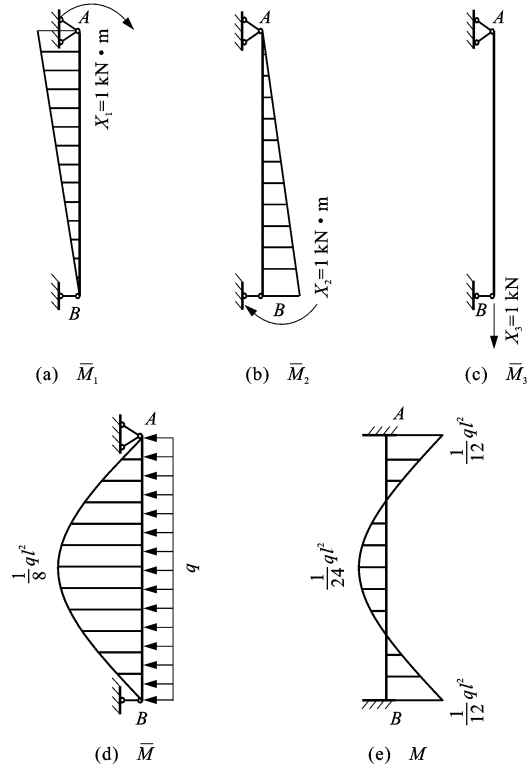


Fig. 7 Bending moments

After solving Eq. (8),  $X_1$ - $X_3$  are obtained as follows

$$X_1 = -X_2 = -\frac{1}{12}ql^2$$

$$X_3 = 0$$

According to

$$M = \bar{M}_1X_1 + \bar{M}_2X_2 + \bar{M}_3X_3$$

the bending moment  $M$  is worked out in Fig. 7(e). It can be known that the bending moment diagram is a second degree parabola. Then, the coordinate mechanics system as shown in Fig. 7(e) is established.  $M(x)$  is the total moment at the point  $x$  of middle wall, and  $a$ ,  $b$ ,  $c$  is its coefficients

$$M(x) = ax^2 + bx + c \quad (9)$$

Its characteristic points are obtained as follows

$$\begin{cases} M(0) = M(l) = -\frac{1}{12}ql^2 \\ M(\frac{l}{2}) = \frac{1}{24}ql^2 \end{cases} \quad (10)$$

Substituting Eq. (10) into Eq. (9), it is gotten

$$M(x) = -\frac{1}{2}qx^2 + \frac{1}{2}ql^2x - \frac{1}{12}ql^2 \quad (11)$$

The relation between the bending moment  $M(x)$  and the deflection  $\omega(x)$  in material mechanics is

$$\frac{d^2\omega(x)}{dx^2} = -\frac{M(x)}{EI} \quad (12)$$

When the double integral is calculated for  $M(x)$ , it is gotten

$$\omega(x) = \frac{qx^2}{24EI}(x^2 - 2lx + l^2) \quad (13)$$

From Eq. (13), the load  $q$  is obtained

$$q = \frac{24EI\omega(x)}{x^4 - 2lx^3 + l^2x^2} \quad (14)$$

Therefore, if the deformation  $\omega(x)$  at any position of middle wall can be monitored, the horizontal load  $q$  can be worked out according to Eq. (14), i. e., the horizontal surrounding rock pressure can be worked out.

### 3 Experimental verification

#### 3.1 Project introduction

Tianhengshan Tunnel, situated at the northeast of Harbin Ring Expressway, a national main trunk line, is up-and-down separation type<sup>[15]</sup>. The up line is 1 660 m in length, and the down line 1 690 m in length. The effective clear width of the tunnel is

$$2 \times 3.75 + 0.75 + 1.25 + 2 \times 1.00 = 11.50 \text{ m}$$

The geomorphic unit of the tunnel belongs to the plain terrain of hillock mound. The earth erosions and fluctuations are strong. The slope is mild in gradient with flat rolling hills. In some areas, there are V-shape gullies with the cutting depths 10-25 m. The frontal edges are connected by steep banks with the back edges of high floodplains on Songhua River, and their elevation difference is about 40 m. What the tunnel penetrates through the strata is mainly cohesive soil, its water content is 16.63%-30.90%, and its natural density is 18.90-20.01 kN · m<sup>3</sup>. The underground water within the tunnel is perched water, existing in the soft plastic and sticky strata. Water seepage arises after holes are drilled.

The traverse passageway for car and the emergency parking strips of the tunnel are

constructed by CRD method in Fig.8. The segment ① is constructed first, and then the preliminary support of arch and the temporary supports of middle wall and inverted arch are completed. Next, the segment ② is constructed, and the preliminary support of arch and the temporary support of inverted arch are given timely. The same method is used for the segments ③ and ④ with connected structural steel frames and temporary steel frames. Then, the foot-locking anchor is set, the preliminary support of inverted arch is completed, and the closed ring forms. For the temporary support, the I 18 steel frames are adopted, the elasticity modulus  $E$  is 210 GPa, and the inertia moment  $I$  is 1 660 cm<sup>4</sup>.

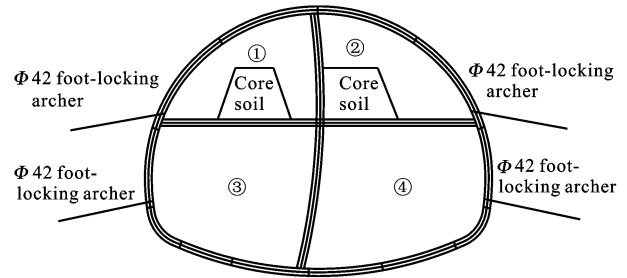


Fig. 8 CRD construction scheme

#### 3.2 Monitoring

The monitoring is conducted for the clearance change of the excavated segment ①. A convergence-measuring line is arranged above the support of inverted arch in Fig. 9. Then, the typical cross section of XK89+138 is selected, and the time curve of clearance convergence is presented in Fig. 10.

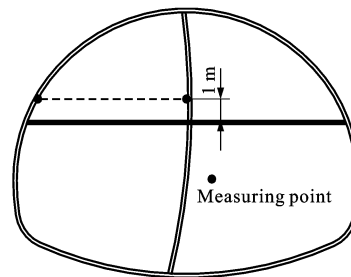


Fig. 9 Layout of test line for convergence

From Fig. 10, in the CRD construction of the segment ①, the clearance convergence increases rapidly, the maximum convergence value reaches 66 mm. In the construction of the segment ②, the clearance convergence decreases sharply by 21 mm

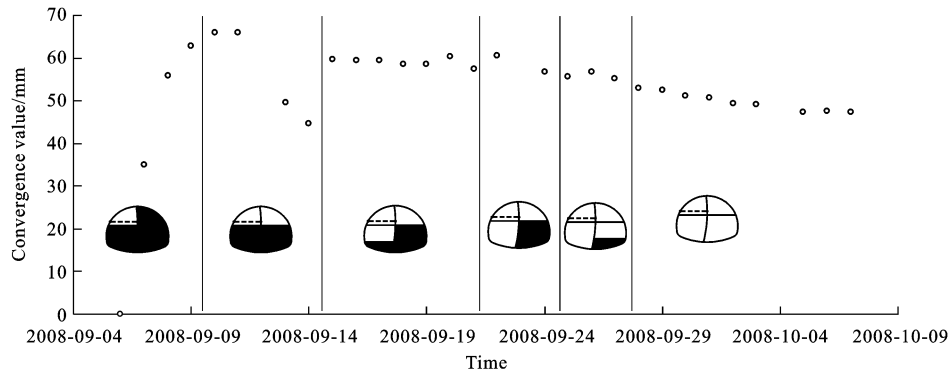


Fig. 10 Time curve of convergence

to 45 mm. The decreasing deformation value is actually the generated deformation value for the middle wall at the location after the horizontal surrounding rock pressure disappears. The relation equation between the deformation value and the horizontal load can be calculated by Eq. (14).

### 3.3 Analysis of calculation result for horizontal surrounding rock pressure

The burial depth of cross section is 23 m, and the maximum excavation width is 16.83 m. The excavation width of the segment ① is 7.86 m. The arc length of middle wall is 5.01 m. Since the middle wall has certain radian, when the curved middle wall is put into horizontal projection, its length is 5 m. The coordinate system is established by setting the joint point between the middle wall and the arch top as the origin, the down vertical direction is taken as the  $x$  axis, and the left horizontal direction as the  $y$  axis. When  $x=4$  m,  $\omega(x)=0.021$  m,  $l=5$  m,  $EI=3.486 \times 10^6$  N  $\cdot$  m<sup>2</sup>, the horizontal load  $q$  is 110 kPa calculated by using Eq. (14).

In order to conduct the feasibility analysis of the calculation method, the Xie Jia-xiao method is used to calculate the horizontal surrounding pressure of the tunnel. The surrounding rock of tunnel belongs to V grade, so the value of the friction angle  $\varphi_g$  is recommended to be 40°–50°. Then, the unit weight of surrounding rock  $\gamma$  is 19 kN  $\cdot$  m<sup>-3</sup>, the excavation width of the tunnel  $L$  is 7.87 m, the excavation height  $h$  is 5 m, and the roof soil friction angle  $\theta$  is  $0.5\varphi_g$ . The calculation result of horizontal surrounding rock pressure is

presented in Tab. 1.

Tab. 1 Calculation results of horizontal surrounding rock pressure

$\varphi_g/(^\circ)$		40	45	50
Pressure/kPa	Xie Jia-xiao method	145	115	88
	New method	110		

From Tab. 1, it can be observed that the horizontal surrounding rock pressure calculated by the new method is 110 kPa, whilst the value calculated by the Xie Jia-xiao method is 88–145 kPa. When the friction angle is 45° taken by the Xie Jia-xiao method, the horizontal rock pressure is 115 kPa, which is close to the pressure calculated by the presented method, the difference is only 5 kPa, and the relative error is only 4%.

Since the values of the friction angles of surrounding rocks and the both sides of top plate by the Xie Jia-xiao method are gained from the experience, so they are different from the actual situation. In contrast, the advanced method is direct and reliable since the data come from practical measurement. The advanced method needs no mechanics calculation parameters of surrounding rock but only measure data. In comparison with the obtainment of mechanics parameters of surrounding rock, the measurement of relative convergence deformation for the field middle wall is easier and more convenient.

## 4 Conclusions

The new calculation method is clearer in concept, easier in formulas, and simpler in operation. It needs no mechanics calculation parameters of surrounding rock but only field measuring data, therefore, and is

easier and more convenient compared with the traditional ones. The computed value by the new method is close to the value computed by the Xie Jia-xiao method, so it can be applied to calculate the horizontal surrounding rock pressures of shallow-buried tunnels. In later research, its feasibility will be verified to the deep-buried tunnels.

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