Research on Influence Zoning of Tunnel Side Crossing Existing Urban Roads Based on Ultimate shear strain Failure Criterion

Mingjie Tian¹, Daiqi Zhang¹, Guiyang Qu¹, Lei Gao¹, Yuqiong Li², Huijian Zhang³, Wenge Qiu*¹

¹China Changjiang Construction Investment. Corp. Ltd, Chengdu, Sichuan, 610000, China
²PowerChina Chengdu Engineering Corporation Limited, Chengdu, Sichuan, 610072, China
³Department of Civil Engineering, Southwest Jiaotong University, Chengdu, Sichuan, 610031, China

*Corresponding author’s e-mail: qiuwen_qw@163.com

Abstract. At present, there are numerous examples of new tunnels crossing through existing urban roads. The stability of the new tunnel underneath the existing urban road is analyzed by strength reduction method. This thesis analyzes on the problems of the instability criterion of the different surrounding rocks at present, and the ultimate shear strain is selected as the criterion of instability of surrounding rock. By comparing the single tunnel with the adjacent system of the tunnel to the existing urban road, taking the difference of safety factor as the threshold value of the influence zone, the mechanical calculation models of different tunnels near the existing urban roads are established. Through the multi-factor analysis of the calculation results, the standard zoning diagram of the tunnels crossing the existing urban roads is obtained, which provides a reference for the similar projects in the future.

1. Introduction
At present, under the large-scale burst of urban infrastructure, the pressure on urban traffic is unprecedented due to the rapid expansion of urban traffic. At present, urban ground traffic has been near saturation, and urban traffic has been transferred from the ground to underground. [1-4].

Zheng Yuchao, Yuan Zhu, and other scholars based on the Changchun Rapid Rail Transit Light Rail Transit Phase III tunnel under a certain section of the railway project, through Ansys established a "ballasted track-spring-subgrade" system static model, according to the railway line management and line maintenance guidelines, studied based on the displacement criterion (settlement) of the relevant threshold of the impact zone. Assaf Klar et al. used fiber-optic distribute strain technology, by laying fiber on the road above the tunnel and applying it to two engineering examples, one is excavating the tunnel with TBM method at the buried depth of 18m, the other is excavating the tunnel with pipe jacking method at the buried depth of 6m. The three-dimensional excavation influence zoning model based on surface displacement is established by dynamic measurement and the peck formula.
At present, the related research on the influence zoning of the tunnel adjacent to the existing urban roads, mostly based on a project. For the zoning of the project, the results obtained are not applicable and universal. In this thesis, the strength reduction method is used, and the ultimate shear strain is introduced as the criterion of safety factor, and the influence on the existing urban roads is discussed in multi-dimensions by considering the factors such as the parallel underpass of the tunnel, the different cross-section forms of the tunnel and the different surrounding rock grades, and the general rules are summarized, which can provide the corresponding technical reference for the future proximity engineering.

2. Theoretical basis for proximate influence zoning

2.1. Strength reduction method

In the numerical limit analysis of rock and soil mass, the physical and mechanical strength index of rock and soil material (cohesion force and internal friction angle of rock and soil mass are reduced synchronously according to a certain proportion) or incremental loading is continuously reduced, so that the rock and soil material finally reaches the failure state and forms the failure surface in the numerical calculation. This finite element numerical limit analysis method is the strength reduction method [5-7].

Taking Mohr-Coulomb material widely used in geotechnical engineering as an example,

\[
\tau = \frac{c+\sigma \tan \varphi}{w} = \frac{c+\sigma \tan \varphi}{w} = \frac{c+\sigma \tan \varphi}{w}
\]

\[
c = \frac{c}{w}, \tan \varphi = \frac{\tan \varphi}{w}
\]

2.2. Ultimate shear strain

The stress-strain curve of an ideal elastoplastic material is shown in Figure 2:
When the yield is reached, it is the initial yield of the material. At this time, there is an elastic ultimate strain $\gamma_y$, that is, the critical strain just entering the plastic state after the material reaches the peak stress, and the material is not destroyed. With the development of plasticity, the material is destroyed, at this time, the strain reaches the ultimate strain $\gamma_f$, that is, the critical strain when the material reaches the failure. For a point, reaching the ultimate strain means that the point has reached the failure condition, but for the whole, although the local failure has been reached and cracks have appeared, it will still appear higher strain until the ultimate strain penetrates, so it is the lowest strain value for the failure of the material.

When the medium principal stress is not considered, the elastic compressive strain and shear strain should meet the Mohr-Coulomb criterion. According to Abir's theory, according to the elastic principal strain, the elastic ultimate shear strain can be expressed as:

$$
\varepsilon_{1y} = \frac{\sigma_{1y} - 2\nu\sigma_{3y}}{E}, \quad \varepsilon_{2y} = \varepsilon_{3y} = \frac{(1-\nu)\sigma_{1y} - \nu\sigma_{3y}}{E}
$$

(3)

$$
\sqrt{J^*_{2y}} = \frac{\varepsilon_{1y} - \varepsilon_{3y}}{\sqrt{3}} = \frac{1 + \nu}{\sqrt{3E}} \frac{2c \cos \phi + 2\sigma_{1y} \sin \phi}{1 - \sin \phi}
$$

(4)

Where: $\varepsilon_{1y}$, $\varepsilon_{2y}$ and $\varepsilon_{3y}$ are the first, second and third elastic principal strains, respectively; $E$ and $\nu$ are the elastic modulus and Poisson's ratio, respectively; $\sigma_{1y}$ and $\sigma_{3y}$ are the first and third principal stresses, respectively; $c$ and $\phi$ are the cohesive force and internal friction angle; $\sqrt{J^*_{2y}}$ is the elastic ultimate shear strain.

The relation between compressive strain and shear strain in elastic-plastic total strain can be obtained from the general formula of strain. If the polar coordinates are taken on the deviating strain plane $r^e$ and $\theta^e$, the relation between principal strain on the deviating strain plane and shear strain $\sqrt{J^*_{2y}}$ and Lode's angle $\theta^e$ is:

$$
\varepsilon_2 - \varepsilon_m = \frac{r^e}{\sqrt{3}} \sin \theta_e = \frac{1}{\sqrt{3}} \sqrt{J^*_{2y}} \sin \theta_e
$$

(5)

If the Poisson's ratio is constant and the isotropic material is considered under uniaxial loading, then the strain Lode angle $\theta_e = -30^\circ$ is:

$$
\varepsilon_2 = -\frac{1}{\sqrt{6}} r^e + \varepsilon_m = -\frac{1}{\sqrt{3}} \sqrt{J^*_{2y}} + \varepsilon_m
$$

(6)

Therefore, the relationship between the elastic-plastic total compressive strain and the total shear strain satisfying the ultimate failure state can be obtained under uniaxial loading (the confining pressure is 0), and the expression of the ultimate plastic shear strain is:

$$
\sqrt{J^*_{2f}} = \frac{\varepsilon_1 - \varepsilon_2}{\sqrt{3}}
$$

(7)

According to this principle, in this thesis, the rock and soil masses of grade III, IV and V are selected, and the parameters of grade III, IV and V in the code are as shown in Table 1 below. The ultimate plastic shear strain of the selected rock and soil masses is calculated, which can be used as the criterion for the instability of the rock and soil masses.
Table 1. Parameters of Different Surrounding Rock Grades

| Surrounding rock grade | Bulk density $\gamma$ ($kN/m^3$) | Deformation modulus $E$ (GPa) | Poisson's ratio $\nu$ | Internal friction angle $\varphi$ (°) | Cohesion $c$ (MPa) |
|------------------------|----------------------------------|-------------------------------|----------------------|--------------------------------------|-------------------|
| III                    | 23–25                            | 6–20                          | 0.25–0.3             | 39–50                                | 0.7–1.5           |
| IV                     | 20–23                            | 1.3–6                         | 0.3–0.35             | 27–39                                | 0.2–0.7           |
| V                      | 17–20                            | 1–2                           | 0.35–0.45            | 20–27                                | 0.05–0.2          |

Using FLAC3D software, the uniaxial compression test of rock and soil mass is numerically simulated, and the cylinder model with the size of 50 mm × 100 mm is established, as shown in Figure 3:

![Figure 3. Numerical Model of Uniaxial Compression Test](image)

The ultimate plastic shear strain and compressive strength of rock and soil with different surrounding rock grade under uniaxial compression are obtained by uniaxial compression numerical test, as shown in Table 2 below.

Table 2. Limit Plastic Shear Strain Values of Rock and Soil Mass with Different Surrounding Rock Grades

| Surrounding rock grade | Compressive strength $\sigma$ (MPa) | Ultimate plastic shear strain $\sqrt{J_2^{3f}}/\%_0$ |
|------------------------|-------------------------------------|-----------------------------------------------|
| III$_{up}$             | 8.24                                | 0.95                                          |
| IV$_{up}$              | 2.93                                | 1.17                                          |
| IV$_{low}$             | 0.65                                | 0.75                                          |
| V$_{low}$              | 0.14                                | 0.52                                          |

The ultimate shear strain of rock and soil mass under each surrounding rock is obtained by numerical calculation. When the ultimate shear strain of rock and soil mass is reached in actual working conditions, the self-bearing capacity gradually fails, and the instability of surrounding rock will lead to the instability of the tunnel. As that two structure are in proximity, when the tunnel clearance is fixed, under different stress distributions, the rock-soil mass may reach the ultimate plastic shear strain first, at which time the tunnel and surrounding rock do not show significant displacement abrupt change, or the model calculation does not converge in the numerical calculation process. Because of the shear slip or shear failure of surrounding rock, the tunnel is unstable first, the plastic range is expanding, and the plastic zone is penetrating, so the tunnel is unstable.

3. Influence zoning of new tunnels adjacent to existing urban roads

3.1. Calculation model
The Mohr-Coulomb yield criterion is used in numerical calculation. Considering the stress field according to the self-weight stress field, in order to weaken the boundary effect, the tunnel diameter of 5 times on both sides of the model tunnel and 3 times on the bottom of the model tunnel is taken. Many scholars have done sufficient research on the calculation of vehicle loads on the existing roads, and the value of 10kPa ~ 20kPa recommended by the Metro Code can be applied. In this calculation, 20kPa is taken as the equivalent static load.

As shown in Figure 4, that calculation model with different position is selected, and the calculation parameters are the same as above.

3.2. Calculation conditions
In building the tunnel, not only will tunnels cross the existing urban roads, at that same time, there will be tunnel running parallel to the exist urban roads. In order to calculate the influence zone of tunnel excavation on the lateral existing urban roads, considering the relative position of the tunnel and the urban road, the center of the road and the vault of the tunnel are selected as the relative position reference points. Considering the symmetry, four directions of 0 °, 15 °, 45 °, 60 ° and 90 ° are arranged for the two lanes, and four directions of 0 °, 15 °, 45 °, 75 ° and 90 ° are arranged for the three lanes, with 6 to 12 working conditions in each direction, as shown in Figure 5.
3.3. Calculation results

Through the analysis of safety factor equivalent points under different surrounding rocks and different angles, the change law of safety factor equivalent points is obtained, as shown in Figure 6:

Figure 6. Distribution Law of Equivalent Points of Safety Coefficients at Different Angles under Different Surrounding Rocks

Summarize the influence zoning maps of the existing urban roads under different surrounding rocks and parallel tunnels, as shown in Figures 7 and 8. The zoning maps can be used as a reference for similar projects in the future.
Figure 7. Standard Zoning Diagram of Two-lane Tunnel Parallel Down-crossing Existing Urban Roads under Different Surrounding Rocks

Figure 8. Standard Zoning Diagram of Three-lane Tunnels Running Parallel to Existing Urban Roads under Different Surrounding Rocks

4. conclusion
The main conclusions are as follows:

(1) When the new tunnel runs parallel to the existing urban road, with the increase of the surrounding rock grade, the surrounding rock parameters decrease, the stress redistribution range caused by tunnel excavation increases, the strength reserve of surrounding rock decreases, and the risk of tunnel construction near the existing urban road increases;

(2) With the increase of the surrounding rock grade, the surrounding rock parameters decrease, and the additional stress range of the existing road and the secondary stress range formed by stress redistribution after tunnel excavation increase;

(3) With the increase of the buried depth, the overall safety factor of the adjacent excavation is the same as that of the single excavation, both of which increase at first and then decrease. With the increase of tunnel buried depth, the shallow buried pressure arch is gradually formed.

Acknowledgments
The research was conducted with funding provided by the National Key research and Development Program of China (Grant No. 2017YFC0806006).

References
[1] Qiu Wenge. Study on Mechanics Principle and Countermeasure of Proximity Construction in Underground Engineering [D]. Southwest Jiaotong University, 2003.
[2] Zheng Yuchao. Study on Influence Degree of Proximity Construction of Three-hole Parallel Shield Tunnel [D]. Southwest Jiaotong University, 2007. 2006(03):376-380.
[3] Zheng Yuchao, Qiu Cultural Revolution. Three-dimensional elastoplastic numerical simulation of internal force evolution of overlapping tunnel structures [J]. Journal of Southwest Jiaotong University, 2006 (03): 376-380.
[4] Klar A, Dromy I, Linker R. Monitoring tunneling induced ground displacements using distributed fiber-optic sensing[J]. TUNNELLING AND UNDERGROUND SPACE TECHNOLOGY, 2014,40:141-150.
[5] Chen Lihua, Jin Xiaoguang. Applicability of Three Failure Criteria for Slopes in Finite Element Strength Reduction Method [J]. Journal of Civil Engineering, 2012, 45 (09): 136-146.

[6] Zhang Aijun, Mo Haihong. Improvement of sudden change criterion of slope instability and displacement in finite element strength reduction method [J]. Geomechanics, 2013, 34 (S2): 332-337.

[7] Cong Yu, Kong Liang, Zheng Yingren, et al. Experimental study on shear strength of concrete [J]. Concrete, 2015 (05): 40-45.