

# Stability Analysis of Large Underground Station Based on Coupled Fluid-Solid Theorem

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**Abstract.** Tunnelling may disturb the intrinsic balance of a stratum, and result in accidents like caving or gushing. In order to assess the security of underground station project, numerical analysis for the stability of surrounding rock was done with fast Lagrangian analysis of continua in three dimensions (FLAC3D). Multiple factors were considered, including surrounding rock classes, tunnel depths, groundwater tables, construction methods and initial supporting systems. According to the results of principal stresses, displacements, plastic zones, pore pressure distribution and the mechanical characters of supporting system including anchors and shotcrete, the seepage mechanism of underground station has been discussed. The pore pressure distribution of deep-buried tunnel was studied as well. The study results can provide a theoretical basis for the design of tunnel and underground works in aquifer strata.

**Keywords.** Underground station, FLAC3D, coupled fluid-solid theorem, stability analysis, computer modeling and simulation

## 1. Introduction

The construction of underground buildings is an important topic of civil engineering in the twenty-first century, because of increases in the scale of construction. This results in the production of a number of large section, great depth, large flat, super close-ups of the projects. The increased technical difficulty of these construction engineering need important higher analysis of experimental, and theoretical research and system support, especially for the stability of surrounding rock of underground engineering<sup>[1]</sup>.

Badaling underground station is the focus control project of the proposed new Beijing-Zhangjiakou intercity railway. The railway station will be built in the mountain which is behind the Great Wall. It is the first construction of the type in China and it is rare in the world. Because of deep setting depth, large excavating area, as well as the complex structural type, similar projects for reference are missing in preliminary design stages. As a result, there is great ambiguity and uncertainty about the selection of construction methods and support parameters. As for the specific rock character surrounding the construction, the excavation methods, and supporting form can be changed, if elastic foundation beam method or other features are unable to obtain satisfactory results. Using the numerical simulation methods development can effectively make up for the limitations of traditional methods, so using the numerical simulation can simulate better than traditional methods. It can also predict the safety of the underground engineering. This paper adopts FLAC<sup>3D</sup> to simulate and analysis a case of underground engineering.

## 2. Engineering and Excavation Methods

Badaling underground station will be built in the mountain. It is the first time in domestic construction work that such a large underground station has been built, and similar projects rarely happen. Because of deep buried depth (75~110m), large excavating area (722.59m<sup>2</sup>), as well as complex structural type, the whole section is divided into nine blocks to excavate which avoids surrounding rock instability caused by large-scale excavation. The diagram of excavation part and sketch of underground station are shown in Fig.1.

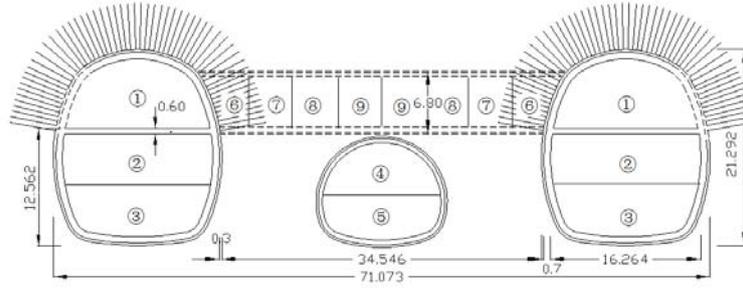


Fig.1 The diagram of excavation part and sketch of underground station (unit: m)

The underground station is constructed with the three-hole-type platform-side structure construction method, each excavation step distance is 4m, and adjacent level spacing is 20m. Excavation step sequence and region size is shown in Table 1 (the same sign represents the synchronize excavation step).

Tab.1 excavation step sequence and region size

Region	Y <sub>1</sub> 0~4m	Y <sub>2</sub> 4~8m	Y <sub>3</sub> 8~12m	Y <sub>4</sub> 12~16m	Y <sub>5</sub> 16~20m	Span (m)	Highness (m)	Area (m <sup>2</sup> )
①	1-1	1-2	1-3	1-4	▽1-5	17.92	9.28	130.57
②	▽2-1	2-2	2-3	2-4	◎2-5	18.26	6.32	111.59
③	◎3-1	3-2	3-3	3-4	※3-5	17.58	5.69	86.58
④	※4-1	4-2	4-3	4-4	☆4-5	13.98	6.20	63.76
⑤	☆5-1	5-2	5-3	5-4	5-5	14.22	5.88	74.80

### 3. Construction of 3D numerical model

When modelling the tunnel, the direction along the tunnel is the y-axis and in the horizontal plane perpendicular to the tunnel direction is the x-axis and direction vertically upwards is the z-axis. The influence of the tunnel excavation is considered. The radius of influence range is more than 3 times one tunnel span. So in the width direction, about 100 m extends respectively to outside the left and right tunnel. So if we add in the span itself, width direction calculation range is 280 m. The size of the 3D numerical model is 280 m×165 m×32 m. The 3D numerical geometric model and construction conditions are shown in Fig.2.

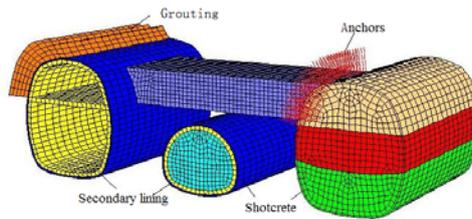


Fig.2 3D geometric model and construction condition

The displacement boundary conditions adopted in the numerical model. The bottom border is constrained with vertical displacement and upper border is free border. Both left and right border are constrained by the horizontal displacement. The same boundary conditions are applied to both the front and back borders along the y-axis<sup>[2-4]</sup>.

The mechanical parameters for the numerical analysis were provided by geotechnical engineering investigation data and they were combined with the national criterion need and parameters discount request in the numerical simulation. The surrounding rock mainly consists of medium grain adamellite with huge phenocrysts, and the mechanical parameters of the surrounding rock and the surrounding rock relevant seepage flow parameters are listed in Table 2 when the coupling problem is considered in numerical simulation. The surrounding rock intensity criteria adopted is the elastic-plastic criterion of Mohr-Coulomb and the concrete intensity criteria adopted is the elastic model. The effect of secondary lining is considered last in numerical simulation.

Tab.2 Mechanics and relevant seepage flow parameters of rocks

Weathered condition	Modulus (GPa)	Friction angle(°)	Density (kN•m <sup>-3</sup> )	Permeability coefficient(m•s <sup>-1</sup> )
Miscellaneous fill	1.0	25	18.0	2.2×10 <sup>-6</sup>
Intensely weathered	5.0	35	23.0	7.1×10 <sup>-6</sup>
Weakly weathered	7.0	40	24.0	1.3×10 <sup>-7</sup>
Slightly weathered	14.0	45	24.8	2.2×10 <sup>-7</sup>

#### 4. Discussion on calculation results

The underground water of surrounding rock tends to seep into the tunnel, and the pore water pressure around the tunnel apparently decreases after excavation. With the passage of time it leads to decreases in ground water tables, and, at the same time, the shape of precipitation funnel starts to show. Tunnel excavation in this way may cause water resources, loss and have a negative effect on the ecological environment. Consequently the reinforcement of rock mass should be considered to maintain the restriction of surface water to the ground and to secure tunnel stability. In general, the cement milk grouting has proved to increase Young's modulus of soil and weathered soil by as much as 20~40 times its original value. But in this numerical model, Young's modulus can be increased as much as 10 times its original values considering conservation<sup>[5]</sup>.

Fig.3 shows the vectors of groundwater flow direction and part contour diagrams of pore water pressure without reinforcement of rock mass. The main seepage parts of the tunnel are centered on the top arch and wall of two side tunnels and on the top of connection tunnel, especially the common part of the top two side tunnels and the connection tunnel. Because of the greater possibility of seepage flow and reduced stability in these parts, there are several other issues and questions to be considered, like the waterproof method and the control reinforcement of the rock mass.

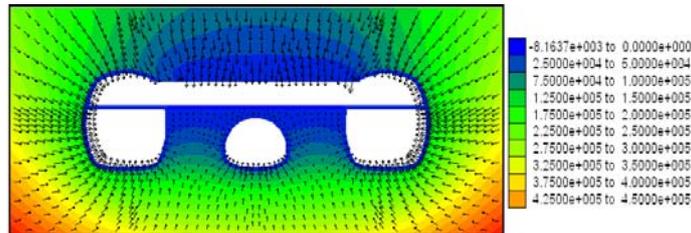


Fig.3 Vectors of groundwater flow direction and pore water pressure (unit: Pa)

The secondary lining is constructed merely as a safety storage of the tunnel in this numerical analysis, so the stress of secondary lining is small. The primary lining stress of the top arch(A), the archspringing(B) and the bottom arch(C) of the side tunnels are monitored. As a result, based on those statistics, the maximum principal stress( $\sigma_1$ ) of primary lining, intermediate principal stress( $\sigma_2$ ) of primary lining, minimum principal stress( $\sigma_3$ ) of primary lining, and anchor force are shown in Table 3. The maximal compression stress value of the primary lining is 8.37 MPa at the archspringing with underground water seepage flow, but there is a 14.1% decrease without underground water seepage flow. Because the excavation of the connection tunnel had a great influence on the building construction of the two side tunnels, the maximal tensile stress of the primary lining is 1.02 MPa at the flank wall behind the two side tunnels and the connection tunnel. The entirety stability of primary lining is better with grouting reinforcement. After the excavation of connection tunnel, stress value of the primary lining of the two side tunnels becomes larger. The maximal anchor force is 24.6kN, and the closer the grid points of an anchor away from the heading face, the larger the anchor force. This is as if should be, because the rock mass is seriously disturbed if it is close to the heading face.

Tab.3 The stress value of primary lining and anchors force

Analysis ways	Monitoring sites	$\sigma_1$ (MPa)	$\sigma_2$ (MPa)	$\sigma_3$ (MPa)	Anchor force (KN)
Without underground water seepage flow	A <sub>1</sub>	-6.07	-0.79	0.12	24.9
	B <sub>1</sub>	-6.78	-1.21	0.30	28.6
	C <sub>1</sub>	-2.21	-0.13	0.51	—
With	A <sub>2</sub>	-6.20	-1.52	0.14	23.3

underground water seepage flow	B <sub>2</sub>	-8.37	-1.82	0.25	31.2
Coupling analysis with grouting reinforcement	C <sub>2</sub>	-3.41	-0.26	0.66	—
	A <sub>3</sub>	-5.13	-1.01	0.32	15.2
	B <sub>3</sub>	-7.19	-1.52	0.19	19.6
	C <sub>3</sub>	-2.39	-0.25	0.42	—

The grid points along the center line on the same level as the top of the two side tunnels were monitored. The maximal vault settlement calculation value is 6.96mm without considering underground water seepage flow, and the maximal calculated value is equal to 10.85mm at the same part when taking it into consideration. Additionally, the closer the grid point's position away from the tunnel excavation region, the larger the difference in settlement value when comparing values with or without underground water seepage flow. For example, on the middle upper grid point of left side tunnel, its final difference settlement value is smaller than the grid point in the center line of the 3D numerical model. Fig.4 shows the time process curves of z-displacement of monitored grid points around left side tunnel vault, the rate of change of settlement is larger at the early stage of tunnel excavation, but it obviously changes at step 17 of excavation(the connection tunnel is excavated at this step). The maximal vault settlement calculation value at the top arch of side tunnel is 6.96mm before the excavation of the connection tunnel, and this value increase by 23.5% after the excavation of connection tunnel. This is to say, the excavation of the connection tunnel has a great influence on the building construction of the two side tunnels. Obviously it can be seen that in coupling analysis, the size of arch subsidence quantity is larger and the entire region is wider when the water seepage functions are considered than when they are not.

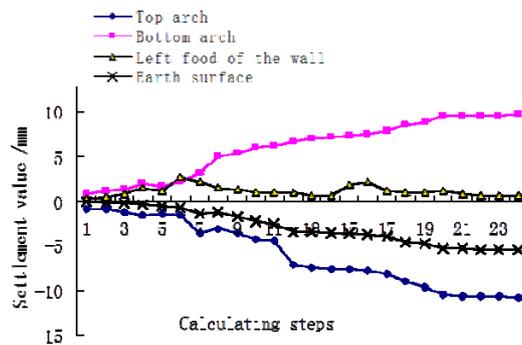


Fig. 4 Time process curves of Z-displacement of monitored grid point

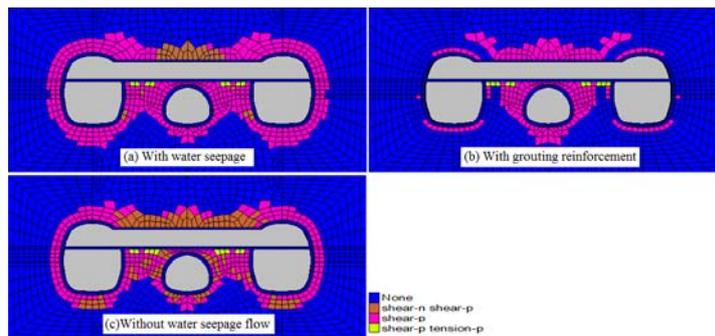


Fig.5 Distributions of plastic zone for surrounding rock in different ways

The distributions of the plastic zone in three ways for surrounding rock at 15-16m section are shown under three situations in Fig.5. It should be pointed out that the plastic zone covers the region between the two side tunnels, and the thickness of the plasticity zone around the three tunnels is about 3~4m. Because of the influence of the seepage field on the stability of the surrounding rack, the compression stress of rock cells become smaller. At the same time the tensile stress of them become larger. Consequently, the volume of the plastic zone is smaller when the coupling analysis is not carried out.

## 5. Summaries

The results of the analysis indicate that the influence boundary of disturbed stress is smaller than that of the seepage disturbed boundary. The seepage forces caused seepage to increase stresses and deformations in

the tunnel ground. According to the interaction principle between ground and support structure, the seepage effects need to be considered when designing tunnel support structures. The excavation of the connection tunnel had a great influence on how the construction of the two side tunnels should be built. The main seepage parts of the tunnel are centered on the top arch and wall of the two side tunnels and at the top of the connection tunnel, especially in the common part of the top of the two side tunnels and the connection tunnel. Because of the greater possibility of seepage flow and greater instability in these parts, there are several other issues and questions that must be considered, like the waterproof method and the control reinforcement of the rock mass.

## 6. References

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