

Chapter 2

Key Techniques of Underground Engineering Stable Equilibrium Theory

Abstract Given the current productive efficiency in China, the core of design method is to “basically maintain the original status of strata (surrounding rock)”, so as to achieve the goal of “reasonably exerting the self-bearing capacity of strata (surrounding rock). In another word, the interaction between strata (surrounding rock) and supporting structure will help the system to achieve the “stable equilibrium and deformation compatibility control”. The design theories and construction methods are in consistency in this concern. The underground engineering stable equilibrium theory not only provides the elaboration of mechanics theory but also requirements for construction methods. The theory describes the basic concept of underground structure design and construction, the suitability and consistency of different design theories and construction methods in a better way. In addition, it also emphasize the significance of underground structure design details and reasonable construction process, which reflects the concept of “simplifying complicated issues”. In Timo Shenko mechanics theory, the complicated boundary condition mechanics issues are simplified before solved, in order to better understand the physical interpretation of engineering problems, and thus facilitate the underground engineering design and construction. Based on the underground engineering stable equilibrium theory, four construction techniques are provided and illustrated with cases in this chapter.

2.1 Excavation Energy Control Technique

2.1.1 *Basic Concept of Excavation Energy Control Technique*

For tunnel construction in soil or soft and fractured surrounding rock, the common used construction methods include full-face excavation with face buttress and multiple-heading methods like CD method (mid-wall method), central pillar

method (glasses method) and CRD method (cross mid-wall method) [1, 2]. For such methods, no or very little blasting is required. In general, mechanized or manual excavation construction is adopted. For rock tunnels, drilling and blasting method is mainly used. The energy E consumed in the construction process of both types of methods can be divided into three parts:

$$E = E_1 + E_2 + E_3 \quad (2.1)$$

where E_1 is the energy consumed by machine or human for breaking the rock mass and casting crushed stone in the tunnel, which belongs to the category of effective energy consumption; E_2 is the energy consumed for disturbing the surrounding rock and pre-reinforcement structure, and maintaining the critical stability of surrounding rock deformation, as well as recovering the stability of the damaged or deformed instable surrounding rock; E_3 refers to other energy consumptions, which are subtle and ignorable.

For construction of rock tunnels, two issues of equal importance should be solved in blasting: the first is to break the rock in the tunnel sections to a certain degree using the most effective method, and cast the stones properly; the second is to minimize disturbance to surrounding rock caused by blasting, in order to maintain the original status of the surrounding rock and guarantee the long-term stability of the tunnel. The excavation energy control technique can be described as follows: the construction excavation scheme that consumes the lowest energy E_2 for disturbing the surrounding rock and pre-reinforcement structure will be the optimal scheme, as it results in the least disturbance to the surrounding rock with the blasting effect guaranteed [3–5].

For tunnel construction in soil or soft and fractured surrounding rock, the common used construction methods include full-face excavation with face buttress. The core is to control the deformation of surrounding rock, so as to basically maintain the original status of the surrounding rock; otherwise, localized instability and failure of surrounding rock may induce surrounding rock instability and failure of a larger scale. The basic requirement of minimizing the energy consumption for tunnel construction is to prevent larger-scale failure of surrounding rock. After failure of surrounding rock, the work required for re-achieving stability of surrounding rock is far more than that required to maintain surrounding rock stability by pre-reinforcement. Therefore, for either mechanized or manual excavation, most energy is consumed during excavation and implementation of pre-reinforcement structure. Two problems should be solved during construction: the first is to reduce disturbance to surrounding rock and pre-reinforcement structure during construction, so as to maintain the original status of surrounding rock to the maximal extent and at the same time make pre-reinforcement structure take effect; the second is to prevent large-scale instability of rock/soil mass during construction. Therefore, for tunnel construction in soil or soft and fractured surrounding rock, the excavation

energy control technique can be described as follows: with sequential construction and satisfactory deformation control of surrounding rock by supporting structure, the scheme with the minimum power consumption E_2 for recovering stability of the failed or instable surrounding rock is the optimal scheme.

2.1.2 Application of Excavation Energy Control Technique

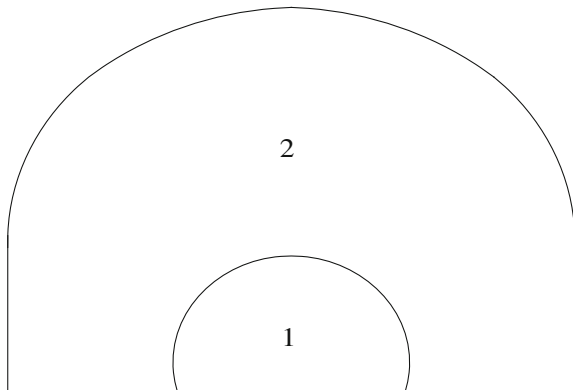
1. Pilot Tunnel Advancing + Expanding Excavation Construction Method

In the construction of big tunnels, a pilot tunnel is usually excavated using the drilling and blasting method or small heading machines as shown in Fig. 2.1. When radial free face is exposed after the pilot tunnel excavation, blasting is adopted for expanding excavation. In such cases, the free face for blasting is wide and the clamping effect is weak, which means less blasting energy consumption and far less disturbance to the surrounding rock.

2. Smooth Blasting is Preferred for Hard Rock than Pre-splitting Blasting

During smooth blasting, blasting the central part poses a small impact on the surrounding rock. Due to exposed free face, the blasting of perimeter poses less impact on surrounding rock. For hard rock, full-section smooth blasting should be adopted. However, the pre-splitting blasting refers to formation of a fissure of a certain width along the design contour by blasting before construction blasting of the tunnel. When blasting the main cross-section, the fissure will reflect the stress waves to reduce the damaging effect of the stress wave on the surrounding rock. Hence, during contour hole blasting, the rock within the section contour line and the surrounding rock will pose the same clamping effect on the blasting. The damaging effect of blasting on the surrounding rock is remarkable; especially when the

Fig. 2.1 Pilot tunnel advancing + expanding excavation construction method



strength of the rock is high and the explosive charge in the contour hole is abundant, the energy consumption is high and the damaging effect is more obvious. If there is joint fissure in the surrounding rock, safety accidents with block falling can be easily triggered. In this case, the pre-splitting blasting is not preferred.

3. Stepwise Construction for Weak Surrounding Rock with Weak Blasting

During tunnel construction, there will often be low-strength, easily weathered and fractured weak surrounding rock, which are classified as class III–V surrounding rock with poor stability. Accidents such as collapse may occur easily when tunneling in such rock. Real cases indicate that the blasting procedure poses a remarkable effect on stability of surrounding rock. Blasting vibration is often a triggering factor of collapse surrounding rock. Therefore, measures should be taken to reduce the vibration intensity of blasting, in order to reduce disturbance to surrounding rock and maintain the original status of surrounding rock to the maximum extent.

For tunnels in weak surrounding rock, the top heading and bench method is generally adopted. For the top heading construction, the smooth blasting is applied to the arch part; the self-weight of rock may help with cracking of the arc rock surface along the periphery hole, which can reduce the explosive consumption and the energy consumption to certain extent. This can guarantee the blasting effect and at the same time lower the intensity of vibration of surrounding rock caused by the explosion in periphery holes. During the bench excavation, to support the surrounding rock in time, the side bench should be excavated and supported first, conforming the order as shown in Fig. 2.2. As the strength of rock mass is low, the disturbance to the surrounding rock of sidewall will be very weak if the weak blasting is adopted.

If there is significant differences in properties of rocks in the tunnel section, the construction scheme should be adjusted. If the heading rock mass is weak while the bench is hard, the bench construction order should be adjusted accordingly as shown in Fig. 2.3. If the excavation order in Fig. 2.2 is adopted, the rock mass on

Fig. 2.2 Construction order of top heading and benching tunneling method. 1 Heading excavation and supporting; 2 left bench excavation and supporting; 3 right bench excavation and supporting; 4 middle bench excavation and supporting

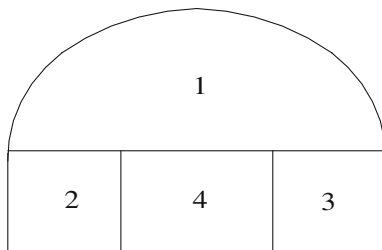
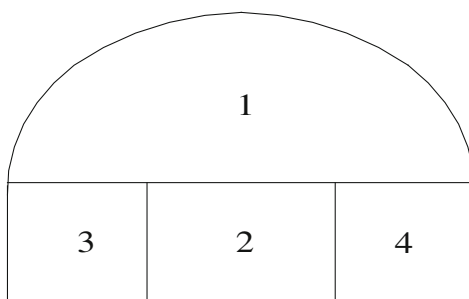


Fig. 2.3 Construction order with hard bench
part. 1 Heading excavation and supporting; 2 middle bench excavation; 3 left bench excavation and supporting; 4 right bench excavation and supporting



the two sides will be subjected to strong clamping effect in the horizontal direction. As the rock is hard, it takes strong blasting energy to break the rock mass, which requires high energy consumption and causes more obvious disturbance to the surrounding rock.

4. Reasonable Selection of Excavation Method

In order to reduce the explosive charge required for excavation of a section and control blasting scale, the top heading and bench tunneling method may be adopted. As most of the city tunnels are of the shallow-buried type and the surrounding rock of top heading is weak, the explosive charge required is low (or even manual excavation is possible). After blasting of the top heading, a free face that is helpful for blasting the bench will be formed, so as to reduce the vibration as shown in Fig. 2.4. For hard strata, the bench and top heading method may be reserved for smooth blasting. The cutting is placed at the bottom to increase the distance from the explosive center of the cutting.

As shown in Fig. 2.4b, the lower face I is excavated at first followed by excavation of the upper face. In this way, there will be a satisfactory free face for blasting the upper section, to improve the smooth blasting effect and reduce the vibration. For the area of the cutting, the advancing step of each blasting cycle should be controlled at about 2.5 m. For the upper section, the step should be controlled as about 2 m. The thickness of the reserved smooth blasting layer should be about 1 m, with the advancing step of about 2 m. Blasting of the cutting hole area + smooth blasting layer and the cutting hole area and interlayer is carried out in turn, so as to ensure that there is about 1 m advancing once the blasting sound is heard from the cutting hole area. The interval between the interlayer and smooth blasting layer is about 2 m.

For instable surrounding rock (half-soil and half-rock in particular), the top heading and bench tunneling method should be used. In another word, the upper soil layer should be excavated manually to form a vibration-isolating slot at the arch, so as to prevent upwards transmission of the vibration wave. Then the blasting method is used to excavate the lower part rock as shown in Fig. 2.4c.

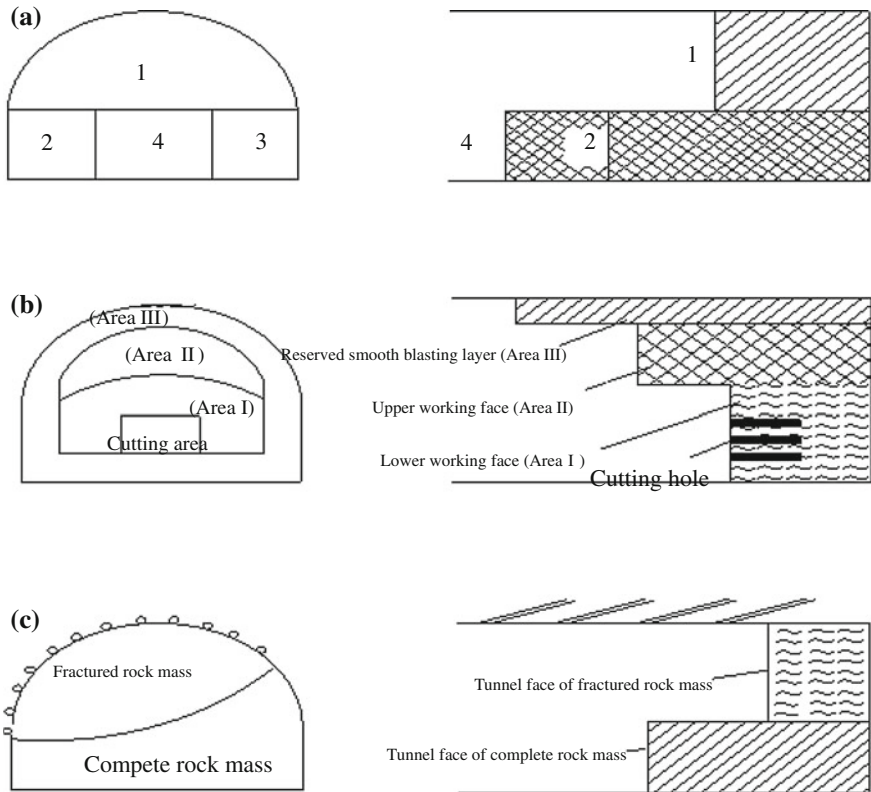


Fig. 2.4 Schematic diagram of blasting excavation method. **a** Surrounding rock with soft upper part and hard lower part; **b** relatively hard surrounding rock; **c** half-soil half-rock surrounding rock

2.2 Strong Pre-reinforcement Technique

2.2.1 Basic Idea and Expression Forms of Strong Pre-reinforcement Technique

The supporting structure of tunnel is used to effectively control deformation of the surrounding rock. Engineers should allow certain deformation (including structural deformation), and should not attempt to stop deformation of surrounding rock, which may induce excessive pressure on the support. Meanwhile, excessive deformation of surrounding rock which may cause collapse should be prevented. A proper supporting structure should be installed at a proper time to prevent any unfavorable stress status in the surrounding rock.

The underground engineering equilibrium stability theory indicated that: the pre-reinforcement force must be strong enough for the tunnel to “basically maintain the original status of strata (surrounding rock)” in any circumstances. The

foundation of “reasonably exerting self-bearing capacity of strata (surrounding rock)” for tunnels is to keep the interaction of surrounding rock and supporting structure in a stable equilibrium and deformation compatibility control status [6].

1. Intact Surrounding Rock with Satisfactory Self-bearing Capacity

Intact surrounding rock has high self-bearing capacity to maintain its stability. As shown in Fig. 1.12a, the surrounding rock self-stabilized without any supporting measure. For tunnel excavation in such surrounding rock, the surrounding rock is allowed to deform to a certain extent. This is because that certain deformation may help reasonably exert self-bearing capacity of surrounding rock, and thus reduce the supporting force required. In many provincial roads or county roads, to reduce construction cost, the self-bearing capacity of surrounding rock is fully utilized to maintain stability of the cavern, and no lining or little initial shotcrete are installed. An example is given in Fig. 2.5. Other examples include Longyou Grottoes, cave dwellings on the Loess Plateau in Northwest China, and tunnels built up for the tunnel warfare during the Second World War. It is similar to the hard rock + shotcrete with anchor for the Norwegian method.

2. Surrounding Rock with Certain Self-bearing Capacity

For surrounding rock with certain self-bearing capacity, the curve of pre-reinforcement principle is shown in Fig. 1.12b. The self-bearing capacity of surrounding rock is greater than the original internal force P_0 in early stages. After tunnel excavation, the surrounding rock will not loosen or collapse immediately, as the surrounding rock pressure is still in the deformation pressure stage. However, the surrounding rock is in an instable equilibrium status. As the deformation increases, the internal structure and stress status of surrounding rock changes constantly. The self-bearing capacity of surrounding rock is exerted, and then decreases. The purpose of supporting is to allow the surrounding rock to shift from the instable equilibrium status to stable equilibrium status, and the timing of

Fig. 2.5 The surrounding rock is able to self-stabilize after excavation



Fig. 2.6 Collapse of a carven after excavation



supporting is very important. As shown in Fig. 2.6, if supporting is provided too early, the self-bearing capacity of surrounding rock cannot be fully exerted. In such a case, it takes very high supporting resistance to allow the surrounding rock to shift from the instable equilibrium status to stable equilibrium status. If supporting is provided too late, the surrounding rock pressure has changed from deformation pressure to loose pressure, and the surrounding rock has shifted from the instable equilibrium status to instability status. In such a case, the surrounding rock will easily loosen, which may result in large-area collapse. Figure 2.6 is the scene of collapse accident caused by late supporting of a tunnel.

3. Fractured or Weak Surrounding Rock with Insufficient Self-bearing Capacity

As shown in Fig. 1.12c, the self-bearing capacity of the fractured surrounding rock is quite low and decreases fast after excavation, and the deformation pressure of surrounding rock will quickly change into loose pressure. The surrounding rock will soon enter the loose status, and immediately shift from the instable equilibrium status to the instability status. Therefore, pre-reinforcement or advance supporting should be provided before excavation, in order to improve the original status of surrounding rock and enhance the self-bearing capacity. Even after treatment of the tunnel, the surrounding rock is still in an instable equilibrium status after excavation. However, its self-bearing capacity has been greatly increased to prevent instant collapse, which can provide time for primary support. The rigid supporting must be installed timely as primary support. Furthermore, after excavation of the tunnel, as the surrounding rock is in a sensitive instable equilibrium status, or the instable equilibrium status with poor anti-disturbance capacity, the order of primary supporting greatly influence the change of status. Proper selection of supporting order is of great significance for shifting the instable equilibrium status to the stable equilibrium status. Figure 2.7 is the failure caused by untimely supporting after excavation, or insufficient rigidity of the supporting.



Fig. 2.7 Surrounding rock failure caused by delayed supporting

According to the strong pre-reinforcement technique, before installation of secondary lining, the primary supporting and the surrounding jointly form the bearing system. The bearing capacity of primary supporting is a key component of pre-reinforcement capacity, and plays a very important role. The STM requires primary supporting to be main bearing structure during construction (resist soil pressure and part of the water pressure). The secondary lining and primary supporting will jointly bear the permanent load.

It is of equal importance to maintain the strength of primary supporting of fractured surrounding rock. For example, the cause of collapse accident of the class-III surrounding rock of Wuzhuling Tunnel was the insufficient strength of primary supporting. The flexible supporting structure was installed for the section. The shotcrete started to crack as shown in Fig. 2.8a shortly after application of



(a) Cracking of primary support



(b) Collapse

Fig. 2.8 Failure of preliminary supporting

primary supporting and later the tunnel collapsed suddenly as shown in Fig. 2.8b. This case indicates that the concept that “the primary supporting should be strong enough to bear part of the water pressure and all of the soil load; for the shallow-buried and subsea tunnels, the supporting will bear all the water load and soil load; the secondary lining is taken as the emergency capacity” should be followed for design of tunnels in weak surrounding rock.

For the fractured surrounding rock of class IV–V, multiple heading method is one of the common construction methods for large-section tunnels, multiple-arch tunnels and small-clearance tunnels. For construction of tunnels in fractured surrounding rock, the surrounding rock may easily fall and drastically increase lining load. The theoretical study shows that the surrounding rock stress concentration can be reduced through multiple heading excavation, so as to reduce the section of single heading and lower the deformation energy absorbed by the surrounding rock. In this way, the surrounding rock can maintain its stability shortly after excavation, so as to create favorable conditions for supporting installation.

4. Excavation of Tunnels in Special Environments

The strong pre-reinforcement technique is used to interpret the uniformity and suitability of various design theories and their related construction methods for general tunneling issues. For tunneling in special environments, certain extension is required based on the existing Terzaghi theory, М. Лромобъяконоб theory and other applicable mechanics theories. The diversion tunnels of Jinping II Hydraulic Power Station are given as an example.

Jinping II Hydraulic Power Station utilizes the natural head difference 150 km of Jinping Bay of the Yalong River, to draw water for power generation through straightening tunnel. The power station has the largest hydraulic tunnel in the world. The major difficulties of the project include the design and construction of four diversion tunnels of about 16.6 km long. The diameter of the diversion tunnels is 12 m, or 11 m after lining. The general buried depth of the tunnels is 1500–2000 m, with the maximum depth of 2525 m. For the section with buried depth of 2525 m, the self-weight of the overlying rock mass reaches 68 MPa in spite of the tectonic stress. According to the elastic mechanics theory, even if tunnel excavation only induces 2 times concentration of the original stress, the maximum stress of the surrounding rock will be 136 MPa. The diversion tunnel surrounding rock is mainly composed of marble, with uniaxial compressive strength (UCS) of only 80–120 MPa. Therefore, the surrounding rock stress will exceed the UCS of the rock, and the rock mass strength will be far lower than that of the rock. In addition, there is external water pressure of more than 1000 m in the diversion tunnel. Therefore, large-scale plastic failures of the surrounding rock caused by excavation will be unavoidable as the buried depth of the power station is excessively deep. It is the inevitable to allow certain plastic failure of the surrounding rock subjected to ultrahigh ground stress field. The basic requirement for supporting design and construction control is that we should make the best out of the environment. Prevention against harmful result of extension of plastic deformation area, utilization of plastic deformation of

Fig. 2.9 Karst cave

surrounding rock to reduce the concentration of surrounding rock stress, and allowing the stress concentration area to move into the deeper may help reduce the stress level on the supporting structures (these measures were adopted by Japanese ocean survey vehicles at the depth of 7000 m under the sea level); as an option, the concept and entire construction process of classical history projects may be referenced, which is also in accordance with the practice of modern mechanics. For instance, Fig. 2.9 is a karst cave; the proven length of Guizhou Shuanghe Karst Cave is about 117 km, which is made of more than one hundred branch caves and many underground streams; the stable status of cave is also in accordance with the law of utilizing the plastic deformation of surrounding rock to move the stress concentration area into the deeper surrounding rock. No matter what construction method is adopted, the target is to stabilize the tunnel surrounding rock with the most economical means, so as to guarantee safe construction of the tunnel.

2.2.2 Application of Strong Pre-reinforcement Technique in Surrounding Rock with Satisfactory Self-stabilizing Capacity

The class I–III hard surrounding rock and class IV surrounding rock with satisfactory stability are usually rock mass with satisfactory integrity. After excavation of the cavern, the overall stability of the surrounding rock will be satisfactory. Strength of the discontinuities is usually a key factor affecting stability of surrounding rock. The anchor-shotcrete supporting is used to stabilize the surrounding rock, control surrounding rock deformation, prevent loosening and collapsing of surrounding rock and generating of “loose pressure”. Depending on different geological conditions of the surrounding rock, there are two types of design concept of anchor-shotcrete supporting: (i) for class I–III hard and intact surrounding rock, the supporting parameters may be determined according to the *Design Code for Highway Tunnels* [7]; (ii) for other class II–III hard rock with average stability and

class IV surrounding rock with satisfactory stability, the interaction theory between the surrounding rock and supporting should be used for stability analysis.

1. Intact Hard Surrounding Rock

Class I–III hard complete surrounding rock falls into the first circumstance of pre-reinforcement principle analysis. The self-bearing capacity of surrounding rock after excavation is higher than the original ground stress, and the surrounding rock can self-stabilize. In such cases, only localized block falling or rock burst should be taken into consideration for stability of surrounding rock. The strong pre-reinforcement technique will be adopted for turning the key blocks into stable blocks through anchoring and shotcreting. The basis for identification of key blocks is the practical experience of engineering and field intuitive judgment ability of field engineers, and the monitoring measures used. The primary supporting parameters can be determined according to the *Design Code for Highway Tunnels*. In situ stress calculation and supporting parameter design are not necessary.

2. Class II and III Surrounding Rock and Class IV Surrounding Rock with Satisfactory Stability

Other Class II–III hard rock with general stability and class IV with satisfactory stability fall into the second circumstance of pre-reinforcement technique analysis. As shown in Fig. 1.5, the pre-reinforcement technique refers to allowance for certain deformation of the surrounding rock and provision of certain supporting resistance for surrounding rock through flexible supporting ($F > P_0$). In this way, the surrounding rock will shift from instable equilibrium to stable equilibrium status. The flexible supporting is mainly realized through anchor-shotcrete supporting. Combination with metal net or steel sets or both may be adopted in the forms of rockbolt-shotcrete mesh, rockbolt-shotcrete frame and etc.

According to rock mass structural control theory, the stability of surrounding rock is mainly controlled by the rock mass structure. Deformation of surrounding rock is predominated by structural-controlled deformation, and surrounding rock failures are also predominated by structural failures. The strengthening mechanism and the power of strengthening effect of rockbolts are closely related to discontinuities of rock mass. Therefore, researches should be carried out for different rock mass structures.

(1) Rock Mass of Blocky Structure

There are several groups of discontinuities in the rock mass. The existence and strength of such discontinuities usually controls the strength and stability of rock mass. The structural instability process of the surrounding rock of blocky structure begins with falling of localized rock blocks on the surface. Without supporting, the blocks one through five would fall successively as shown in Fig. 2.10. This process is just the structural instability process, and mainly depends on the connectivity, orientation, roughness and underground water condition of the discontinuities [8–11]. The purpose of the anchor-shotcrete supporting is to control opening and

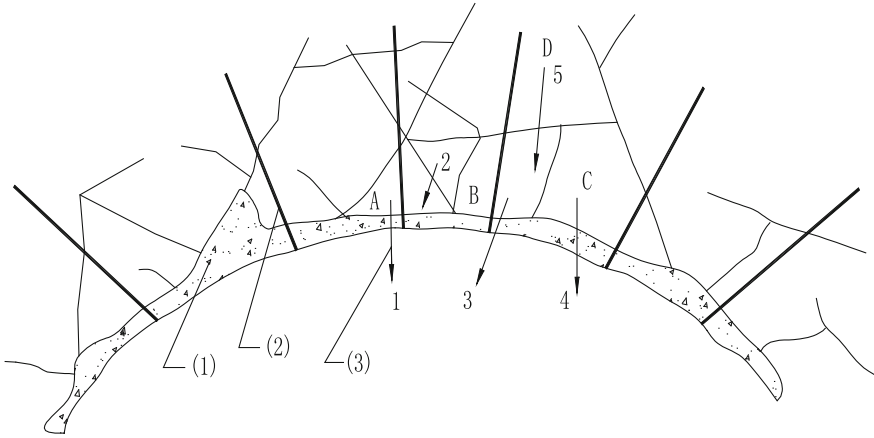


Fig. 2.10 Schematic diagram of anchor-shotcrete supporting technique for blocky surrounding rock. (1) Shotcrete; (2) rockbolt; (3) potential collapse direction of the block; the potential collapsing order of blocks is 1, 2, 3, 4, and 5

sliding along discontinuities through the self-tension and shearing, so as to prevent falling of the blocks, and maintain the original contact relation and original strength of surrounding rock.

(2) Rock Mass of Stratified Structure

Figure 2.11 shows the instability condition of tunnel surrounding rock in rock mass of stratified structure. The layer instability forms are predominated by buckling and instability of hinged arch. Sun Guangzhong proposed the beam column buckling model in 1980s [12]. This model is applicable when the span to thickness ratio is high and the longitudinal stress is high.

The composite beam action theory believes that the stratified rock is anchored by the rockbolt, so as to control structural deformation and instability of the rock mass.

After all, the main function of rockbolts is to control the structural deformation and instability of surrounding rock. According to the study on rockbolt supporting mechanism for different rock mass structures, Fig. 2.12 shows the optimized arrangement form of stratified rock mass rockbolts.

3. Synergistic Action of Rockbolts and Shotcrete

(1) Through combination of internal supporting and surface supporting, rockbolts can be inserted deeply inside the surrounding rock, so as to strengthen the rock mass within the anchoring layer. The shotcrete is the surface supporting of surrounding rock. (2) Combination of local strengthening supporting and general supporting: the rockbolts can directly strengthen and maintain the rock mass of the applied part, while shotcrete is the general supporting of the entire tunnel surface. (3) Combination of faces and points components in terms of geometric forms: rockbolts refer to point supporting while shotcrete refers to face supporting.

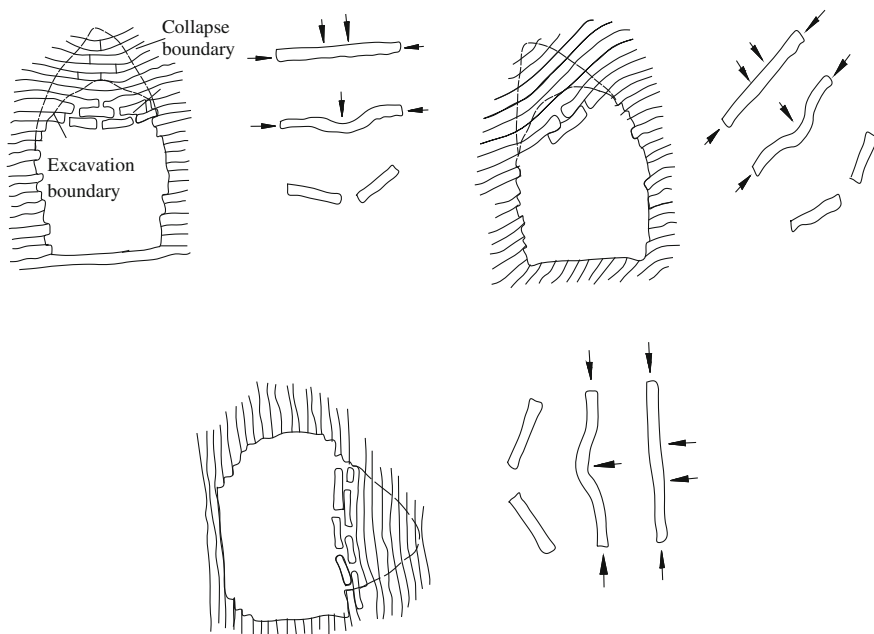


Fig. 2.11 Instability forms of stratified rock mass

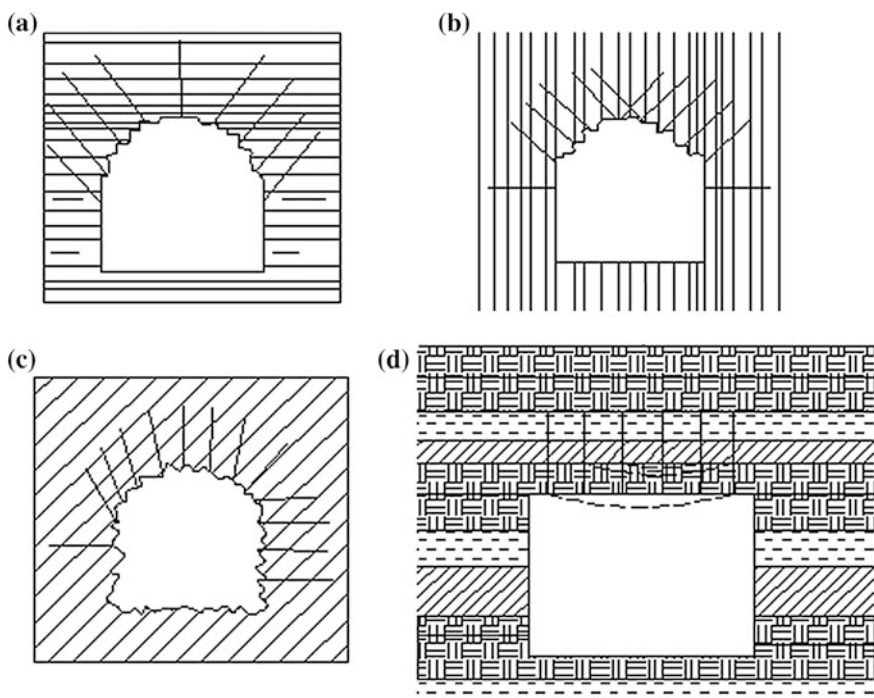


Fig. 2.12 Optimized arrangement of anchor bolts

Rockbolt is used to strengthen the surrounding rock from inside, to allow the surrounding rock to form a bearing structure. The shotcrete can stick closely to the surrounding rock and apply radial pressure and circular shearing force on the surrounding rock, and thus increase the circular pressure of surface rock and prevent falling of surface blocks.

2.2.3 Application of Strong Pre-reinforcement Technique to Deep Excavations in Surrounding Rock with Poor Self-stabilizing Capacity

The deep-buried fractured surrounding rock, especially the surrounding rock at the arch part of a tunnel, has poor self-stabilizing capacity. The self-stabilizing time of the surrounding rock is very short or there is even no self-stabilizing time. After loss of bearing capacity, collapse may easily take place, which may result in engineering accidents. Tunneling safely and economically in such surrounding rock has always been a focus of the engineering practitioners.

Rockbolt, grouted anchor, forepoles, blanks and other steel sets and grouting body in the pre-reinforcement structure jointly form the arched shell along the tunnel longitudinal direction to bear the above fractured rock mass. The surrounding rock is maintained before displacement and the self-bearing capacity of surrounding rock gets improved. Self-stabilizing of surrounding rock is maintained before deformation occurs, and the surround rock can self-stabilize after a certain period of time, which provides conditions for further installation of supporting and lining.

For class III–V fractured surrounding rock with poor stability, pre-reinforcement or step-by-step construction with timely supporting should be adopted. Use weak blasting to minimize disturbance to surrounding rock, and follow the principle “basically maintain the original status of surrounding rock” to achieve the stability of the surrounding rock.

For class III–V surrounding rock with poor stability, one of the effective way is to adopt appropriate pre-reinforcement for the full-face of lower pilot tunnel. This method is technically and economically beneficial. Different pre-reinforcement schemes are adopted depending on the types of fractured surrounding rock, which is also the prerequisite of application of appropriate advance pre-reinforcement for the entire section of lower pilot tunnel. Some of the commonly used pre-reinforcement schemes are described below:

- (1) For class IV hard surrounding rock with satisfactory stability, the advance rockbolt and advance bolt grouting combined with lattice girder arch pre-reinforcement can be adopted before the appropriate construction of the entire section of the lower pilot tunnel.

- (2) For class IV–V soft surrounding rock of satisfactory stability, the advance short forepoles (small steel pipe) or blank (combined with steel sets pre-reinforcement) can be adopted before the appropriate construction of the entire section of lower pilot tunnel.
- (3) For class IV–V soft surrounding rock with poor stability, the advance long forepoles or blank (combined with steel sets pre-reinforcement) can be used before the appropriate construction of the entire section of lower pilot tunnel.
- (4) For the following special cases, the rigid supporting (backboard method) may be used or the strata should be improved before the appropriate construction of the entire section of lower pilot tunnel:
 - (i) The loose rock mass that has not been cemented or artificially filled gravelly soil;
 - (ii) Shallow-buried sections in which open cutting is not suitable;
 - (iii) Expansive rock mass or loose rock mass with expansion factors and dense discontinuities;
 - (iv) Vibrant underground water movement, which results in a large-area water spraying.

Direct use of anchor-shotcrete supporting is not preferred for excavation of tunnels in the above four types of unfavorable geological conditions. The STM or rigid supporting similar to the Shield Tunneling Method in soft soil should be adopted, such as the support system that integrates advance forepoles, small steel pipe or blank, steel sets and shotcrete, and the method that improves strata to strengthen the surrounding rock. The core is to allow only water loss but control or limit the loss of solid particles. The short excavation and strong pre-reinforcement is used to basically maintain the original status of surrounding rock, and thus reduce and restrain harmful deformation of surrounding rock, and reasonably exert the self-bearing capacity of surrounding rock.

Engineering practice shows that, for mountain tunnels with non-cemented soil-like surrounding rock, the soil mass pressure will be reduced to 0 when the procedures are centralized, the back of lining is backfilled in a compact way, and construction quality is good. The old loess cave dwellings in Northwest China and the tunnels built up for the tunnel warfare in North China are examples in this respect. However, no cavity is allowed in the underground tunnel lining under the groundwater level. Compact backfilling and grouting is required (this has also been demonstrated by model tests). Alternatively, the commonly used water-proof plate scheme may be altered by directly spraying waterproof materials onto the primary supporting, followed by secondary supporting with pumped concrete, and the generation of cavity in the secondary lining should be prevented.

For class V–IV surrounding rock, class III surrounding rock with poor stability and surrounding rock with special geological conditions, the use of methods that is similar to shield tunneling or strata improvement method may help to stabilize the surrounding rock, control surrounding rock stress and deformation, prevent loosening, collapsing and generating of “loose pressure”. However, the supporting



Fig. 2.13 Tunnel collapse accident caused by supporting design parameters according to design code

mechanism is different from that of anchor-shotcrete supporting. The design can be conducted based on the M. Лромобъяконоб and K. Terzaghi theories. As the surrounding rock of special geological conditions has very poor stability and is usually subjected to the action of underground water, to ensure safety, the internal friction angle of surrounding rock ϕ and the cohesion c (i.e. $c = \phi = 0$) is not taken into consideration. Only the action of surrounding rock reaction forces P_1 and P_2 induced strong pre-reinforcement is taken into consideration, which will not generated hazardous looseness.

For the design of some failed tunnel cases, the proposed supporting design parameters listed in Table 7.4.3-2 of *Design Code of Highway Tunnels* [7] were taken as reference with only the action of anchor-shotcrete supporting taken into consideration. After completion of tunnel primary supporting, collapse took place before the installation of secondary lining as shown in Fig. 2.13. This indicated that the parameters specified in Table 7.4.3-2 have some limitations in such a case. The tunnel pre-reinforcement mechanics theory that uses methods similar to shield tunneling in soft soil can be used to determine the per-supporting parameters as shown in Fig. 2.14, to avoid or reduce occurrence of similar accidents.

1. Theoretical Calculation of Supporting Structure in Deep-buried Fractured Rock Mass

For tunnel excavation in deep-buried or shallow-buried fractured rock mass, it is extremely important to determine the pressure of surrounding rock during supporting design. Firstly, several methods should be utilized to determine the pressure of surrounding rock, and then the stress of steel sets is analyzed. The M. Лромобъяконоб theory is used for calculation.

(1) Calculation Based on Statistics Theory

The concerned department of the Ministry of Railways has summarized the investigation data of 357 collapse accidents of single-track railway tunnels [6].

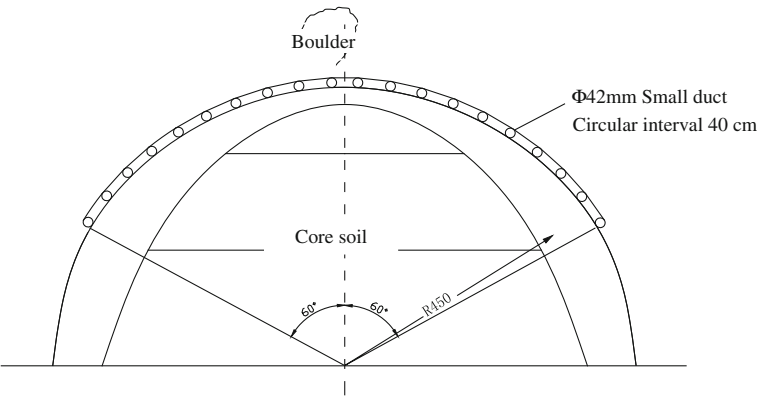


Fig. 2.14 Strong pre-reinforcement to ensure safe excavation of tunnels

Table 2.1 Statistical height of collapse in various surrounding rocks (m)

Surrounding rock classification	I	II	III	IV	V	VI
Collapse height h (m)	0.65	1.29	2.4	4.32	9.6	19.2

The arithmetic mean value is taken as the mathematical expectation, to obtain the statistical height of collapse in various surrounding rocks as shown in Table 2.1.

The excavation height of the underground tunnel is H ; the width is B ; then let $h = n(B + H)$.

The load coefficient $n = 0.043e^{0.64(S-1)}$ (S refers to the type of surrounding rock) can be obtained based on the statistics of data.

The design load of supporting structure is $q = \gamma n(B + H)$.

(2) **Determination of Surrounding Rock Pressure**

When designing actual tunnel support, we should reduce the in situ stress obtained by calculation. In general, for fractured surrounding rock sections, steel sets is installed timely after excavation in order to restrain the expansion of loose zone. The actual pressure on the steel sets is lower than the gravity of the rock mass in the collapsed area given in the statistics. The pressure reduction coefficients for various surrounding rock types from various literature are summarized and listed in Table 2.2. The reduced load $p = \mu q$ will be used for calculation and analysis of internal force of steel sets.

Table 2.2 Surrounding rock pressure reduction coefficient μ of various fractured surrounding rocks

Surrounding rock classification	III	IV	V	VI
μ	0.3	0.4	0.6	0.7

(3) Checking Calculation of Steel Sets Stress

In recent tunnel construction, steel sets are usually used for fractured surrounding rock section or collapsed area. The most predominant characteristic of steel sets is that they are able to bear load immediately after installation, and control further expansion of loose and plastic areas of surrounding rock and rapid development of deformation. The steel sets interval specified in design codes should be no more than 1.5 m, and is generally selected as 1.0 m. For fractured surrounding rock sections, the interval should be further reduced.

During tunnel construction, the combined action of rockbolt and steel sets can be used to form the load-bearing arch, and increase the rigidity of primary supporting. The rock mass can be stabilized by grouting through lengthened bolt. For sections with heavily fractured rock, the intervals between steel sets should be further reduced.

After initial selection of a proper type of steel sets according to codes, the in situ stress can be used for checking calculation. The calculated safety coefficient of initially selected steel sets should fall within the designed allowable range; otherwise, re-selection of the steel sets is required until requirements are met.

2. Full-face Excavation with Appropriate Pre-reinforcement and Lower pilot Tunnels Ahead

Engineering practice indicates that a lower pilot tunnel excavated with short step (1–2 m/cycle) approximately 3–5 m ahead other portions of the face, is economically feasible for tunneling with pre-reinforcement in the class III–V surrounding rock (Fig. 2.15).

(1) Basis for Appropriate Pre-reinforcement of Full-face Excavation with Lower Pilot Tunnel Ahead

- (i) The lower pilot tunnel (3–5 m ahead) plays the role of probe drilling in poor ground, to facilitate early emergency measures to handle geological problems and to eliminate any hazard before accidents occur.

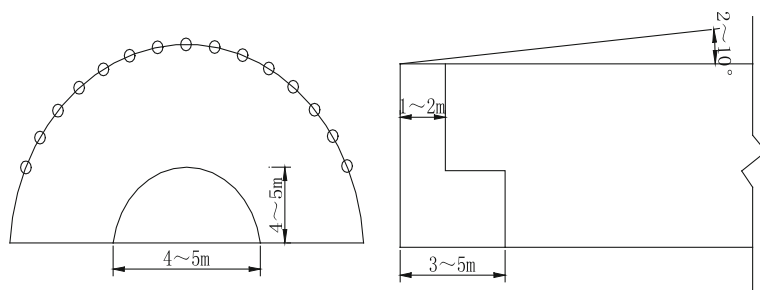


Fig. 2.15 Schematic diagram for appropriate full-face excavation with pre-reinforcement and lower pilot tunnel ahead

- (ii) If there is abundant underground water, the lower pilot tunnel can lower the underground water level satisfactorily; this method is highly suitable for large-span tunnels;
- (iii) As the span of the lower pilot tunnel is small, the self-stabilizing capacity of surrounding rock is increased, which places less demand on temporary support; the invert struts/lining should be installed in time to form an enclosed structure;
- (iv) The method facilitates mechanized operation; muck removal and material transportation can be carried out on the bottom of the tunnel section without secondary handling. Furthermore, the free face for blasting is wide with little clamping effect, and less explosive energy is needed. According to the principle of energy conservation, the least work is required for such tunnel construction. Therefore, the other consumptions are also controlled to the minimum, and the damage to the tunnel surrounding rock is also minimized;
- (v) When tunneling through various geological conditions, the method does not require change of construction machinery and operation platform, which increases the utilization rate of machinery;
- (vi) The circular steel sets are strengthened to increase the overall rigidity. It is like the relation between the railway track and sleeper interval. The key point is to reduce the sleeper interval and increase the sleeper rigidity.

(2) Instructions for Pre-reinforcement of Fractured Surrounding Rock

Pre-reinforcement measures for class IV surrounding rock: for sections in poor quality ground, surround rock usually has joints and fissure development and influenced by underground water, and thus small pre-grouting pipes should be adopted to compact and improve the surrounding rock and prevent water. For sections with underground water activities, it is not practical to use the “anchoring-first, grouting-second” type rockbolts. The best way is to use self-drilling rockbolts with the quick-hardening grouting to improve the surrounding rock.

Pre-reinforcement measures for class V–VI surrounding rock: for artificially backfilled gravel soil and loose rock, rock mass with joints and fissures, and strata with underground water activity, direct use of rockbolt-shotcrete support is not recommended. For sections where the umbrella shell is not easily formed using forepoles, it is difficult to grouting forepoles and “anchoring-first, grouting-second” grouting rockbolt because the effect is not satisfactory. The small pre-grouting pipes (the short forepoles) or steel blank should be used as the longitudinal pre-reinforcement. This is because the steel sets, when used as radial support, has high overall rigidity, which helps with restraining unfavorable deformation of the surrounding rock. For heavy-deformable sections, measures such as ground grouting rockbolt or self-drilling long grouting rockbolt and timely installation of invert struts or advance deep-hole curtain grouting can be adopted to control initial deformation of surrounding rock, in order to mobilize the self-bearing capacity of surrounding rock in poor ground.

2.2.4 Application of Strong Pre-reinforcement Technique to Deep Excavations in Surrounding Rock with Large Deformation

The 19.8 km long New Puru Tunnel I, which was completed in 1906, underwent large deformation. Enasan Highway Tunnel in Japan, Tauern Tunnel in Austria, and Arlberg Tunnel all underwent large deformation of surrounding rock. In China, large deformation of surrounding rock was observed in Guanjiao Tunnel of Qinghai-Tibet Railway, Muzhailing Tunnel and Baoziliang Tunnel of Baoji-Zhongwei Railway, Jiazhuqing Railway Tunnel of Nanning-Kunming Railway, Zhegushan Highway Tunnel of National Road 317, and Tieshan Tunnel [13, 14]. Large deformation causes great troubles for engineering construction. Practical experiences indicate that large deformation usually takes place in areas with high ground stress. In addition, the surrounding rock are soft, loose, fractured or expandable. The large deformation of tunnels induces great hazards, including high rectification cost and long construction period delay. The prevention and control of large deformation of tunnels is one of the worldwide problems. Research on control of large tunnel deformation has become a hot topic in the field of tunneling.

Control of large deformation during tunneling is a complicated and dynamic process. Through supporting measures, displacement compatibility between rock and support is realized. That the surrounding rock and support system act jointly to reach a stable equilibrium status, should be taken as the supporting principle for controlling large deformation. The main supporting techniques include the use of smooth blasting technique to control over excavation of and reduce disturbance to surrounding rock, and thus basically maintain the original status of surrounding rock; the use of strong pre-reinforcement technique to improve the bearing capacity and variability performance of surrounding rock; the forms of primary support can be rockbolt-shotcrete and net frame support system, with the supporting parameters optimized; the supporting should be applied stepwise to control the displacement of surrounding rock; secondary lining should be applied at a proper time to ensure that the interaction of the surrounding rock and support system reaches the stable equilibrium status.

1. Large Deformation Mechanism Analysis

(1) High Ground Stress and Weak and Expansive Surrounding Rock are the Internal Causes

The investigation into the geological conditions of numerous large deformation tunnels indicates that there are two internal causes of large deformation of tunnels. The first is the high ground stress field. The typical cases include Ahlberg Highway Tunnel, Zhegushan Highway Tunnel, Guanjiao Railway Tunnel, Wushaoling Tunnel, Tauern Highway Tunnel, and Jiazhuqing Railway Tunnel, the in situ stress of which reaches above 10 MPa. The second is the poor properties of the surrounding rock, which is specifically reflected by the weakness, joint development (breakage) and expansibility.

Weak rock mainly includes mud rock, clay, shale and carbonaceous shale. If a tunnel penetrates the fault fracture zone or tectonic active zone, discontinuities developed extensively in the surrounding rock, which reduced the strength of the surrounding rock. Examples include Muzhailing Highway Tunnel, Cheyang Tunnel, Wushaoling Tunnel, Huocheling Tunnel, Furongshan Tunnel, Liangfengya Tunnel and Bixi Tunnel. Under the action of water, the volume of expansive surrounding rock may increase after the minerals absorb water; the surrounding rock will squeeze into the tunnel to induce large deformation. For instance, the surrounding rock of Beishan Tunnel, Nakasakuma Tunnel, Iwate Tunnel, Nakaya Tunnel, New Utsu Tunnel, and Baoziliang Railway Tunnel of Baoji-Zhongwei Railway is expansive. The common lithology of such rock includes tuff, silty mudstone, and argillaceous siltstone. Some tunnels have both high ground stress and poor rock properties, such as Enasan Tunnel, Liangfengya Tunnel and Wushaoling Tunnel.

The aforementioned large deformation tunnels can be divided into large deformation tunnels of high ground stress and weak surrounding rock, and those of expansive rock. The ground stress and surrounding rock strength are two major factors that determine deformation of the tunnel. Comparison with regard to the two factors should be made to judge whether the surrounding rock will have large deformation. Therefore, the ratio of surrounding rock strength and ground stress value should be taken as the criteria to judge whether there will be large deformation. The basic characteristics of expansive rock include high hydrophilia, high expansion rate, high expansion pressure and high disintegration. Such characteristics are adverse to the stability and maintenance of tunnels. The hydrophilia of expansive rock is high because it contains clay minerals with high hydrophilia, such as montmorillonite, illite, and kaolinite. There will be strong absorption after they get contact with water. In such cases, the inter-particle cohesive bondage will be weakened, and the particle interval increased and volume expanded.

(2) Engineering Disturbing Force is the External Cause

Before excavation, the surrounding rock is in the 3D stress and a stable equilibrium status. After excavation, a new space is formed with free faces on the face and walls. The rock core was excavated and newly exposed free faces provide no supporting action on the surrounding rock. The original stable equilibrium status of the surrounding rock is interrupted. The surrounding rock starts to deform towards the inside of tunnel section. In this case, the original stress redistributes in the surrounding rock. During stress adjustment, part of the stress is released in the form of deformation energy. The radial stress within the surrounding rock is reduced, even to zero on the rock surface, while the circumferential stress increases to induce the stress concentration.

The magnitude of secondary stress is related to that of the original in situ stress, the cavern shape, the support system and the excavation sequence. Selection of a proper cavern shape can help reduce concentration of surrounding rock stress. Selection of the cavern shape should be based on the type of in situ stress field. The

elastic mechanics analysis shows that when the ratio of vertical axis to horizontal axis of an oval section is equal to the reciprocal of the lateral pressure coefficient, the cavern shape can be regarded as optimal. In this case, the tangential stress in the surrounding rock should be compressive stress, and should be equal everywhere near the surface. For instance, the round section should be the optimal shape for a cavern subjected to hydrostatic stress. After Wushaoling Tunnel advanced into F7 fault zone, the section was revised from the horseshoe shape to the round shape, which is good for maintaining surrounding rock stability. The weak rock mass has very remarkable non-linear mechanical properties. After excavation, the surrounding rock is in the plastic and rheological status, which is against the superposition principle of force during the entire mechanics process. The process of excavation and supporting is actually the loading and unloading process of surrounding rock. The construction process is an irreversible non-linear evolutionary process. The final status is not unique, but depends on the stress path or stress history. With different excavation and support sequence, the presence and degree of stress concentration, the size of the plastic zone, the final deformation of surrounding rock will be different. The stress, the damage area, and the tunnel convergence are greatly influenced within the construction period as the cavern shape and loading mode keep changing. Furthermore, the stress redistribution after excavation and the area of the damage zone will be also impacted. Therefore, reasonable selection of the excavation and supporting sequence of a tunnel plays a very important role in controlling displacement. Generally, under greater in situ stress, the secondary stress will also be greater; the ratio of secondary stress to the strength of surrounding rock determines whether there will be large deformation. With high ground stress and weak surrounding rock, the strength of surrounding rock will be far lower than the secondary stress, and a large-scale plastic zone will be formed in the surrounding rock. The surrounding rock will develop obvious plastic deformation with time.

(3) Improper Construction Measures and Unreasonable Support Structure are Direct Causes

Soft rock subjected to high stress and expansive soft rock both have characteristics such as large deformation, high-rate and long-duration deformation. During the construction of Wushaoling Tunnel, the maximum horizontal convergence and crown settlement reaches 1,034 mm and 1,053 mm respectively, with the deformation rate of 34 mm/d. In addition, the deformation lasted for several months or even several years. The deformation characteristics of large-deformation tunnels are different from those of other tunnels. Therefore, the construction technique and supporting structure of large-deformation tunnels should also be different.

Proper construction scheme and technical measures should be chosen to facilitate maintaining the original status of surrounding rock basically. Measures such as smooth blasting and pre-reinforcement are general technical measures used in construction. The purpose of such measures is to “basically maintain the original status of the strata (surrounding rock)” during construction, in order to ensure the

stability of the surrounding rock. When the mining method is used, the use of smooth blasting aims at reducing vibration of surrounding rock caused by blasting and thus maintaining the original status. For surrounding rock with poor stability, pre-reinforcement method is generally used, to strengthen the surrounding rock before commencement of tunnel construction. For large-deformation tunnels, short-bench or micro-bench construction method should be used to shorten the exposure time of surrounding rock, and close the support to form a loop. In this way, the surrounding rock of the entire section can be satisfactorily supported in time.

For selection of supporting structure, the first important point is to identify the specific controlling factors of large deformation. The controlling factors are causes that lead to large deformation and determine the mechanism of such deformation. They are also the main basis for selection of supporting techniques. The large-deformation tunnels with different controlling factors require different technical schemes. The large-deformation tunnels have different surrounding rock pressure types, including loose pressure, deformation pressure and expansion pressure. For loose pressure, strong pre-reinforcement can be adopted to strengthen the surrounding rock and improve the self-strength of the rock mass; meanwhile, the rigid support is adopted to support the surrounding rock to avoid collapse of fractured rock block. Deformation pressure is the predominant pressure form for soft rock tunnels. For deformation pressure, in addition to the support rigidity, the supporting timing and sequences should be controlled depending reasonably on the rheological characteristics, in order to allow certain deformation and facilitate release of energy while controlling the deformation within a certain extent. In this way, the pressure will not develop into loose pressure. Expansive pressure can be regarded as a kind of deformation pressure. In addition to the measures taken to control deformation pressure, the physical and chemical effect that cause dehydration and drying of surrounding rock should also be prevented. This is because that drying and watering cycles may cause serious expansion and slaking of some soft rock. Large deformation of tunnels results from the co-existence of several mechanisms induced by several factors. In different stages of construction, the dominant factors that cause the deformation are different. Therefore, large-deformation tunnel support is a series of sequences, which cannot be accomplished in an action. Related technical measures should be taken in the design and construction stage, to overcome factors that cause large deformation one by one, so as to achieve deformation compatibility between surrounding rock and supporting structure and allow the rock-support interaction to achieve the stable equilibrium status. For example, the rockbolts used as the first primary support of weak surrounding rock in the high stress area of Wushaoling Tunnel were not long enough to reinforce the surrounding rock, which is one of the major factors that have caused the large deformation; furthermore, the initial bench was too long in the construction scheme, and in this case, the primary support could not be closed early enough. This is also one of the major factors that have caused the large deformation.

2. Common Treatment Measures Against Large-deformation Tunnels

Measures used to treat large deformation tunnels are given in Table 2.3.

As shown in Table 2.3, comprehensive measures should be taken for large-deformation tunnels. The main technical measures include:

- (1) Sufficient deformation allowance should be reserved in the tunnel excavated section to allow certain deformation of the surrounding rock.
- (2) Strong pre-reinforcement or rockbolt-grouting support should be adopted to strengthen the surrounding rock and improve the self-bearing capacity of surrounding rock;
- (3) The short-bench method or ultra-short-bench method should be adopted; temporary invert strut or temporary support is required; shotcrete layer is used to close the surrounding rock in time;
- (4) Strengthen primary support by lengthening or densifying the rockbolts, and thickening the shotcrete layer; add invert lining and foot anchors; use steel sets with sliding joints, so as to allow controllable displacement of the surrounding rock under strong primary supporting action. In this way, unloading and transfer of secondary stress into deeper portions can be realized.
- (5) The secondary lining is strengthened by thickening, reinforcement with steel fibers and timely installation, which provides high-strength support and stabilize surrounding rock. Water control should be enhanced for expansive soft rock, which is also necessary for other types of large-deformation tunnels.

3. Recommended Support Schemes for Large Deformation Tunnels

The problem of the aforementioned excavation and support schemes is that if the secondary lining is applied before the displacement of primary support and surrounding rock converge, the secondary lining will be subjected to high rheological pressure and may crack. The core problem is that the deformation of the lining structure and surrounding rock is not compatible, and the lining cannot be kept in a stable equilibrium status. Therefore, reasonable design of support system and choice of proper support timing for large-deformation tunnels has been a hot topic in the engineering tunneling.

Construction concept: displacement control for large-deformation tunnel is a dynamic process, which aims at guaranteeing deformation compatible between surrounding rock and support structure through construction and supporting measures; in this way, the rock-support interaction will be in a stable equilibrium status.

Main techniques: (i) the smooth blasting technique should be used to improve the section forming quality, to control surrounding rock disturbance and basically maintain the original status of surrounding rock; (ii) the strong pre-reinforcement technique is adopted to improve the self-bearing capacity of surrounding rock; (iii) the primary support is installed stepwise; the supporting parameters should be optimized to allow the surrounding rock displacement to gradually converge; (iv) secondary lining is applied at a proper time to allow rock-support interaction to reach a stable equilibrium status.

Table 2.3 Construction measures for large-deformation tunneling projects

S/N	Project	Strata	Main measures			(Ultra) short bench	Primary support	Timely installation of secondary lining	Inverts	Water control	Others
			Reserved deformation (mm)	Strong pre-reinforcement							
1	Retrofit of Nakaya tunnel	Expansive tuff	300	—	√	Thickened shotcrete layer; 9 m long rockbolt; anchors provided at the foot and bottom; increased rockbolt density	—	√Increase curvature of temporary invert struts	—	—	—
2	Retrofit of New Utsu tunnel	Expansive tuff	—	—	—	Thickened shotcrete layer; 6 m long rockbolt; increased rockbolt density	—	√Add temporary invert struts	—	—	Secondary lining reinforcement
3	Retrofit of Enashan tunnel	Fractured rock subjected to high stress	500	—	—	Thickened shotcrete layer; 9–13.5 m long rockbolt; steel sets with sliding joints	√	—	—	—	—
4	Wushaoling tunnel (China)	High stress of the fault zone	400	√	√	Shotcrete layer thickness of 200 mm; re-spraying of 150 mm; 6 m long rockbolt; 120 section steel sets; feet-lock bolt	√	—	—	—	Enhanced monitoring
5	Primary support of Wushaoling tunnel	Phyllite	—	√	—	Shotcrete layer 250 mm; 4 m (roof) 6 m (wall) grouting rockbolt; H175 section steel sets and metallic net	—	—	√	—	Enhanced monitoring; support by horizontal steel pipes
6	Jiazhuging tunnel (China)	High stress and low strength	Roof: 450; wall: 250	√	√	Shotcrete layer 250 mm + 150 mm; 8 m long rockbolt; steel sets with sliding joints	√25 mm + 55 mm	—	—	—	—

(continued)

Table 2.3 (continued)

S/N	Project	Strata	Main measures			(Ultra) short bench	Primary support	Timely installation of secondary lining	Inverts	Water control	Others
			Reserved deformation (mm)	Strong pre-reinforcement							
7	Primary support for retrofit of Huocheling tunnel	Class VI-V surrounding rock	20-30	-	-	-	6 m long rockbolt; grouting rockbolt; steel sets of 18# I-shaped section	√	-	-	Bar-mat reinforcement for secondary lining
8	Primary support for retrofit of Liangfengya tunnel	High stress and low strength	300	-	-	-	Shotcrete layer 250 mm; rockbolt grouting; 20b section steel sets; feet-lock rockbolt	√	√	√	Thickness of secondary lining 800-1000 mm
9	Primary support for recovery of Bixi tunnel	Low strength	Arch: 400; wall: 250; bottom: 20	-	-	-	Shotcrete layer 250 mm; rockbolt grouting; steel sets of I16 I-shaped section; bottom reinforcement	Increased rigidity and strength of secondary lining	-	√	Addition of temporary support
10	Muzhailing Highway tunnel	Mud stone; fault fracture zone	500-800	√	√	√	Densified 6-8 m rockbolt; U-shaped section steel sets with sliding joints; rockbolt provided at the invert strut	√Double-layered bar-mat reinforcement for secondary lining	√	-	Reinforcement provided by inverts; connected to the wall reinforcement

(1) Highlight Strong Pre-reinforcement

One of the characteristics of large deformation tunnels is the high initial deformation rate. The surrounding rock quickly deforms after excavation of the tunnel, which will cause deterioration of the rock structures and gradual reduction of the self-bearing capacity. One of the reasonable design concepts is to strengthen pre-reinforcement, so as to allow the surrounding rock to be supported by the strong pre-reinforcement immediately after commencement of excavation.

(2) Stepwise Installation of Primary Support

Large-deformation surrounding rock falls into the category of class VI rock according to the *Design Code of Highway Tunnels* (JTG D70-2004). The surrounding rock is predominated by soft-flow plastic clayey soil, backfill, saturated and silty-fine sand layers or soft soil. The rock mass deformation will be large in such strata. The soft-first and rigid-second double-layer primary support should be adopted as shown in Fig. 2.16a. The main technical scheme for the first flexible supporting layer is as follows: lengthen and densify the rockbolts; install of lattice girder; thicken shotcrete layer and lay metal net; add base invert and foot rockbolts. The flexible support allows certain deformation of the surrounding rock and certain development of the plastic area, to exert the self-bearing capacity of the surrounding rock. In this way, the rock mass stress can be released to a certain extent while maintaining stability of the rock mass. The other rigid support layer requires steel sets, shotcrete and metal net to control deformation, in order to make the deformation converge and get prepared for the installation of secondary lining. The specific construction method is to provide rigid support after completion of the first flexible support layer (3–10 m) with a space of 5–20 m according to the measurement feedback. This aims at controlling deformation of surrounding rock, and basically maintaining original status of surrounding rock, and allowing the surrounding rock to stabilize gradually as shown in Fig. 2.16b.

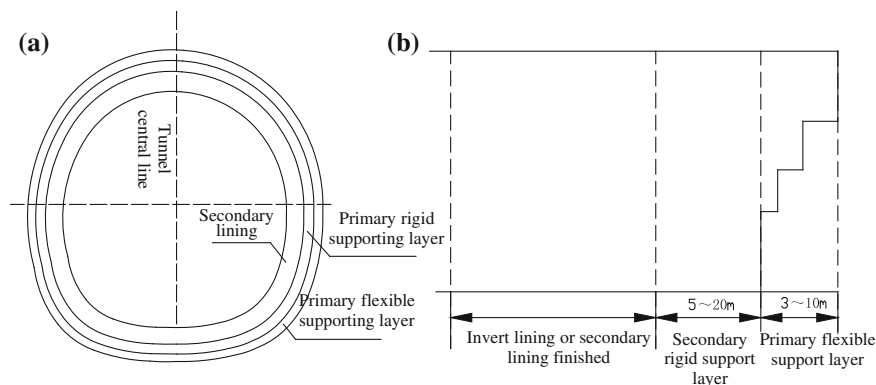


Fig. 2.16 Schematic diagram for construction and lining of class IV surrounding rock

(3) Installation of Secondary Lining at a Proper Time

After the deformation of surrounding rock becomes basically stable, the secondary lining can be installed. The secondary lining bears only a little or no pressure of the surrounding rock, and allows rock-support interaction to achieve a stable equilibrium status.

2.2.5 Application of Strong Pre-reinforcement Technique to Shallow-Buried Tunnel in Strata with Poor Self-stabilizing Capacity

1. The Basic Concept of STM is the Same as that of Strong Pre-reinforcement Technique

The STM is a tunnel construction method proposed by Academician Mengshu Wang and other underground engineering technicians of China Railway Tunnel Group based on the successful experience abroad and the experience in construction of mountain tunnels in China using the NATM [1, 15]. This method is applicable for constructing the portal section of mountain tunnels, metros in urban areas and shallow-buried structures for other purposes, where weak surrounding rock is dominant. The major characteristics of this method: a process is established to feedback the design and construction based on the measurements; use the new “soft-first and rigid-second” composite lining support system; the primary support bears all basic load; the secondary mold-casting lining is used as the emergency capacity; the primary and secondary support will jointly bear the special load. The excavation methods include the positive top heading and bench tunneling method, unilateral pilot tunnel method, and mid-wall method (CD method or CRD method), and the central pillar method (spectacles technique). Through years of constant summarization and improvement, the STM is now widely used in urban metro, utilities, heating power, power pipelines, city underground passages, underground parking lots, and etc. A complete set of supporting techniques have been developed. The open-cut method (cut and cover method), shield method and STM have their respective advantages and disadvantages as listed in Table 2.4. A proper construction method should be selected according to the specific conditions of the project.

Design and construction of tunnels has the following characteristics:

- (i) The engineering analogy is the main basis for design of shallow-buried tunneling technique; before engineering design, the geological conditions of the section should be compared with similar engineering geological conditions, so as to determine the pre-selected design scheme for the project, which is called pre-design.
- (ii) Carry out structure calculation using the load-structure method; the calculation results should be similar to the actual stress condition of the structure.

Table 2.4 Comparison of shallow-buried underground engineering construction methods

Comparison indexes	Method		
	Open cut (cut and cover) method	Shield method	STM (NATM)
Geology	Various strata	Various strata	Special treatment is required for the underwater strata
Places	Occupying a large area of street pavement	Occupying a small area of street pavement	Occupying a small area of pavement
Section change	Suitable for different sections	Not applicable	Suitable for various sections
Depth	Shallow buried	A certain depth required	A certain depth (less than that required for shield method) required
Water proofing	Easy	Difficult	Comparatively difficult
Ground subsidence	No	Remarkable	Ignorable
Traffic obstacle	Huge impact	Moderate impact	No impact
Underground pipelines	Relocation and protection required	No relocation or protection required	No relocation or protection required
Vibration and noise	Heavy	Moderate	Moderate
Ground relocation	High	Comparatively high	Low
Water treatment	Dewatering and draining	Combination of blocking and dewatering	Combination of blocking and dewatering or blocking and draining
Progress	Subjected to serious disturbance of relocation; the overall construction period is short	The advance works is complicated; the length of overall construction is fair	The construction can be commenced very soon; the overall construction period is long

- (iii) Deformation control of surrounding rock is the core problem with STM.
- (iv) Design and construction should be closely related to each other; construction measures should be taken into full consideration during design stage.
- (v) As the geological conditions of shallow-buried tunnels are clear, the pre-design should be as accurate as possible.

In engineering practice, the following principles should be followed for STM:

- (i) The critical ground subsidence should be determined according to the engineering environment conditions and safety requirements for the tunnel; the critical subsidence should not be the strictest values such as 10 or 30 mm;
- (ii) The ground subsidence, construction safety, period and cost must be taken into consideration to choose appropriate excavation technique;

- (iii) The pre-reinforcing measures (such as forepole umbrella, rockbolt and grouting) should be taken;
- (iv) Time-space effect should be considered for tunnel supporting;
- (v) After tunnel excavation, primary support with sufficient rigidity and early strength should be provided as soon as possible to control deformation of the surrounding rock rather than exert its self-bearing capacity to the maximum extent;
- (vi) The invert should be constructed and closed as soon as possible to form a loop; the distance between the invert and working face should be as short as possible and should be no more than 1 time the cavern diameter;
- (vii) Generally, the secondary lining should be installed after the deformation of surrounding rock and primary support basically converges. However, the secondary lining may be installed earlier if the stability requirements have not been met even after assistant measures are taken (as the shallow-buried tunnel load is clearly known, it is possible to install secondary lining earlier);
- (viii) Enhance monitoring and measurement; feedback information in time; adjust the support parameters in time;
- (ix) The composite lining should be adopted; waterproof layer should be provided between layers to isolate water and prevent cracking. Only when there is no shearing force between the two layers, can the cracking of secondary lining be prevented. This method is generally used in the quaternary strata, where the self-bearing capacity of surrounding rock is poor. In order not to damage the ground building and underground structures, the ground subsidence should be strictly controlled. Hence, the primary support should be very strong and be provided in time. The construction key points for such design concept can be summarized as “advance pre-reinforcement; strict grouting; short footage; strong support; early closing; frequent measurement and fast feedback”.

Comparison between the basic concepts of STM and strong pre-reinforcement technique shows that the two are the same. Specifically, the operable measures of STM are more complete and mature. However, the operable measures of STM can also be used for strong pre-reinforcement technique, and the application scope of the latter is more reasonable in terms of physics and mechanics concepts.

2. Design of Pre-reinforcement Structure for STM

(1) Computational Simulation of Excavation

The basic research idea: after excavation of a cavern, the surrounding rock stress will transfer into the deeper portions and redistributed. Finally, the reattributed stress will be applied onto the excavated cavern structure as the equivalent load to study the mechanical behavior of the cavern after excavation. In the calculation, the reinforcing action of primary support on surrounding rock and the interaction between different parts during excavation are studied. Furthermore, the impact of excavation steps and the excavation sequence on the stress-strain status and final stress deformation is also considered.

General conclusion of calculation and analysis:

- (i) There is stress concentration on the inverts and sidewalls, where the bending torque and axial force are high, which results in strong loosening stress. This is closely related to the excavation span;
- (ii) The loosening scope around the bottom and sidewall is wide, which means that the invert must have a high bearing capacity;
- (iii) The lengthened anchors at the bases of steel sets play a very important role to effectively control development of the plastic area; the shortened rockbolt at the roof crown can completely meet the requirements;
- (iv) The amplitude and direction of the shearing stress depends on the excavation sequence;
- (v) After installation of the inverts, the plastic area at the bases will be properly controlled, which means that timely installation of inverts and closing of the lining structure is very important;
- (vi) To expedite construction, the top heading and bench method is preferred than mid-wall method and central pillar method. However, if the surrounding rock condition is poor, and/or there is need to control subsidence and deformation, the mid-wall method and central pillar method should be used; the procedures conversion is considered, the top heading and bench tunneling method and mid-wall method may be exchanged.

(2) Analysis Using the Experience Analog Method

Table 2.5 are the statistical analysis results of cases of large-section tunnels. To facilitate summarizing and analyzing, the spans are divided into 10, 15 and 20 m, and the excavation areas are divided into 100 m², about 140 m² and more than 170 m². According to the excavation faces, the first type is the large section and the last two types are ultra-large sections. The flatness ratio calculated is also listed in the Table.

The analysis shows that:

- (i) The flatness ratio decreases with the increase of the span, which indicates that the designer should take vertical clearance and economical feasibility into consideration for design. If it is economical to reduce excavation area by strengthening the primary supporting and lining thickness as the span increase, it means that the research of flatness rate is also an economic issue.
- (ii) During construction, soft rock has poorer stability compared with the hard rock at the roof crown. When tunneling in soft rock, the loose pressure on the two sidewalls is high, and the base heave severely; a small curvature radius should be used for design.
- (iii) As the flatness ratio of the tunnel decrease, the axial force lining will be smaller, and the negative moment on the lining of two sidewalls will be higher. The positive moment on the roof basically remains the same. This means that the stress on the lining of two sidewalls has increased. In such a case, the thickness of the sidewall lining should be increased to control the lining stress of the tunnel.

Table 2.5 Statistics of support parameters and excavation methods for large-section underground structures (rock class II–III)

Span	10 m	15 m	20 m
Excavation area	Up to 100 m ²	About 140 m ²	More than 170 m ²
Flatness ratio	0.6-0.72	0.49-0.68	0.52-0.64
Item	Used scheme	Recommended scheme	Used scheme
Construction method	(1) Top heading and bench method; (2) Top heading and bench method with temporary closing; (3) CD, CRD method	Deep buried: (1) Top heading and bench method; (2) CD, CRD method; Shallow buried: (1) CD, CRD method; (2) Top heading and bench method with temporary closing	(1) Top heading and short bench method; (2) CD, CRD Method; (3) Central pillar method
		Deep buried: (1) Top heading and short bench method; (2) CD, CRD method; Shallow buried: (1) CD, CRD method; (2) Central pillar method	Deep buried: (1) CD, CRD method; (2) Central pillar method (3) Top heading and short bench or ultra-short bench method; Shallow buried: (1) CD, CRD method (2) Central pillar method
Shotcrete (cm)	5-20	10-15	15-25
Rockbolt (m)/ (circular × longitudinal)	2.5-3.0/ (1.5 × 1.2)	2.5/(1.0 × 1.0)	3.0-4.05/(1.0 × 1.0)
Steel sets model/distance(m)	H150/1.5	—	H200/1.0
Pre-reinforcement	—	—	Small grouted pipes
Lining thickness (m) (roof/invert)	(0.25-0.5)/(0-0.5)	(0.25-0.3)/(0.25-0.3)	(0.3-0.6)/(0.3-0.6)
		(0.25-0.35)/(0.25-0.35)	Small grouted pipes (0.4-0.5)/(0.4-0.5)

(continued)

Table 2.5 (continued)

Span		10 m	15 m	20 m	
Statistics of support parameters and excavation methods for large-section underground structures (rock class IV–V)					
Construction method	(1) Top heading and bench method (short and super short bench); (2) Central pillar method; (3) CD, CRD method	Deep buried: (1) Top heading and short bench method; (2) CD, CRD method; (3) Central pillar method; Shallow buried: (1) CD, CRD method; (2) Top heading and bench method with temporary closing	(1) Top heading and bench method (short and super short bench); (2) Central pillar method; (3) CD, CRD method (4) Top heading and bench method with temporary closing	Deep buried: (1) Top heading and bench method with temporary closing; (2) CD, CRD method; (3) Central pillar method	Deep buried: (1) CD, CRD method; (2) Central pillar method (3) Top heading and short bench method with temporary closing Shallow buried: (1) CD, CRD method (2) Central pillar method
Shotcrete (cm)	5–20	10–15	10–25	15–25	15–20
Rockbolt (m)/ (circular × longitudinal)	2.5–3.5/ (1.0 × 1.0)	3.0/(1.0 × 1.0)	3–6/(0.8 × 1)	3.5/(0.8 × 0.8)	4.5/(1.0 × 0.8)
Steel sets mode/interval (m)	H250/1.5	Replaced with lattice girds	H250/1.0	Replaced with lattice girders	Replaced with lattice girders

(continued)

Table 2.5 (continued)

Span	10 m			15 m		20 m	
Pre-reinforcement	Small grouted pipes; forepole umbrellae	Small grouted pipes; forepole umbrellae	Small grouted pipes; forepole umbrellae; footing grouting; spiral spraying pipes	Small grouted pipes; forepole umbrellae; footing grouting; spiral spraying pipes	Small grouted pipes; forepole umbrellae; footing grouting; spiral spraying pipes	Small grouted pipes; forepole umbrellae; footing grouting; spiral spraying pipes; roof lining	Small grouted pipes; forepole umbrellae; footing grouting; spiral spraying pipes; roof lining
Lining thickness (m) (roof/invert)	(0.3–0.5)/(0.3–0.6)	0.4/0.4	(0.4–1)/(0.4–1)	0.4/0.5	(0.4–2)/(0.4–2)	0.6/0.7	

- (iv) The measures for strengthening primary support such as the use of long rockbolts, footing and base grouted rockbolt may help reinforce the surrounding rock, prevent loosening and deformation of the surrounding rock and guarantee construction safety.
- (v) In terms of construction procedures, for class II rock and the rock of a lower class, invert struts should be provided at first; then the lining of the section should be closed to maintain the stability of the entire structure.

Furthermore, for detailed rockbolt design, small grouted pipes and forepole umbrella design, please refer to *General Theory of Shallow-buried Tunneling Technique for Underground Works* by Academician Wang MS [15].

3. Construction Measures for STM

For tunnel construction in shallow-buried sections, the subsurface excavation method should be adopted considering the influence on the surrounding environment. The STM is a comprehensive construction technique, in which several pre-reinforcement measures are used to reinforce the surrounding rock and reasonably adjust the self-bearing capacity of the surrounding rock. After start of excavation, the support elements are provided in time to form a closed lining structure that function jointly with the surrounding rock. In this way, excessive deformation of surrounding rock can be effectively controlled.










When the STM is used, the typical construction methods include positive top heading and bench method, while other construction methods suitable for special strata conditions include the full-section method, top heading and bench method with unilateral pilot tunnel, central pillar method (spectacles technique) and mid-wall method. Please refer to Table 2.6 for the detailed construction methods.

Note that the selection of construction method for shallow-buried tunneling depend on various conditions of the specific underground works. A designer should select the most economical and ideal design and construction scheme, or a combination of several suitable schemes. The selection is a dynamic process influenced by many factors.

In the shallow-buried tunneling, if the surrounding rock is hard and the rock mass is stable, excavation is usually carried out at first, followed by installation of support structures. Full-face excavation is preferred if possible. For lining installation, the sidewall lining should be built up at first, followed by roof lining, which refers to the “wall-first roof-second” construction method. If the surrounding rock stability is poor, excavation and supporting should be carried out at the same time, to avoid excessive deformation and collapse of surrounding rock. After excavation of the heading, permanent support elements should be installed. In particular, the roof lining is usually installed after excavation of the top heading, and then the bench is excavated under the protection of the roof. This is called the “roof-first wall-second” method.




In weak and loose strata in urban areas, with respect to strata displacement control, the priority order of STMs is as follows: CRD → central pillar → CD → top heading and bench method with temporary closing heading → top heading and bench method.

Table 2.6 Main excavation methods for underground structures using STM

Construction method	Schematic diagram	Comparison of important indexes					Waterproof	Effort for dismantling temporary support	Cost
		Application conditions	Subsidence	Construction period					
Full-section method		Satisfactory strata; Span ≤ 8 m	General	Shortest	Satisfactory	None	Low		
Top heading and bench method		Relatively poor strata; Span ≤ 12 m	General	Short	Satisfactory	None	Low		
Top heading and bench method with temporary closing heading		Poor strata; Span ≤ 12 m	General	Short	Satisfactory	Small	Low		
Top heading and bench method with face buttress		Poor strata; Span ≤ 12 m	General	Short	Satisfactory	None	Low		
Top heading and bench method with unilateral pilot tunnel		Poor strata; Span ≤ 14 m	Relatively large	Relatively short	Satisfactory	Small	Low		
Mid-wall method (CD method)		Poor strata; Span ≤ 18 m	Relatively large	Relatively short	Satisfactory	Small	Relatively high		
Crossed mid-wall method (CRD method)		Poor strata; Span ≤ 20 m	Relatively small	Long	Satisfactory	Big	High		
Central pillar method (spectacles method)		Small span, large span if continuous used	Large	Long	Poor	Big	High		
Central excavation method		Small span; large span if continuous used	Small	Long	Poor	Large	Relatively high		

(continued)

Table 2.6 (continued)

Construction method	Schematic diagram	Comparison of important indexes					Cost
		Application conditions	Subsidence	Construction period	Waterproof	Effort for dismantling temporary support	
Side drifts-support method		Small span; large span if continuous used	Large	Long	Poor	Large	High
Tunnel column method		Multi-layer multi-span	Large	Long	Poor	Large	High
Cut and cover excavation; reverse construction method		Multi-span	Small	Short	Satisfactory	Small	Low

To conclude, various factors and the geological conditions should be taken into consideration to select an effective construction method.

Note that all the aforementioned content related to shallow-buried tunneling technique is obtained from *General Theory of Shallow-buried Tunneling Technique for Underground Works* by Wang MS.

2.3 Comprehensive Stress-Independence Technique

2.3.1 Concept of Stress Independence for Tunnels and Case Study

1. Concept of Stress Independence for Tunnels

Independent tunnels, small-clearance tunnels and multiple-arch tunnels all have been used for highway construction in China. According to the *Design Code for Highway Tunnels*, the highway tunnels should have two independent tunnels with respect to up and down lanes. The minimum clear distance of independent two tunnels should be as shown in Table 2.7. For special sections that connect bridges and tunnels or is restrained by terrain conditions where the minimum clear distance cannot be met, the small-clearance tunnels or multiple-arch tunnels can be used.

Due to the large distance between the two independent tunnels, there is no superposition of secondary stress fields formed after excavation. Therefore, they can be regarded as two independent tunnels for stability study. For multiple-arch tunnels and small-clearance tunnels, the two tunnels are very close to each other, and the secondary stress field of the surrounding rock will overlap after excavation. As the tunnel construction procedures are complicated and the surrounding rock will be disturbed several times, the internal stress distribution on lining structures is even more complicated. Compared with independent tunnels, the two tunnels of multiple-arch tunnels and small-clearance tunnels (especially for multi-arch tunnels) have great influence on the surrounding rocks of each other under poor or complicated geological conditions. In such a case, the lining stress is complicated and unclear, which will result in poor stability. The stress independent structure will be preferred. Therefore, it is important to take measures in terms of design and construction techniques, to improve the independence of surrounding rock and lining stress of the two tunnels and reduce mutual influence. In this way, stability of the surrounding rock can be realized.

The common construction scheme of multi-arch tunnels is the middle pilot tunnel method. According to the traditional section form, as the top of the pillar is

Table 2.7 Minimum clear distance of two independent tunnels

Surrounding rock classification	I	II	III	IV	V	VI
Minimum clear distance (m)	1.0 <i>B</i>	1.5 <i>B</i>	2.0 <i>B</i>	2.5 <i>B</i>	3.5 <i>B</i>	4.0 <i>B</i>

B width of tunnel span

not satisfactorily compacted during the left and right tunnel construction, the tunnel span will be increased and the surrounding rock stability will decrease. The structural stress of the left and right tunnels is unclear and mutual impact exists. To overcome such problems, the section of multi-arch tunnel should be optimized and improved. A good section should ensure the interaction between the pillar and surrounding rock to guarantee the stress of the two main tunnels is basically independent, and thus “maintaining the original status of surrounding rock and exerting full self-bearing capacity of surrounding rock” can be realized. CRD (crossed mid-wall method) technique is usually adopted for small-clearance tunnels. Reinforcing measures will be taken for the mid-wall and bases to ensure the mid-wall and foundation rock is reinforced and stabilized. In this way, the stress independence of the two main tunnels can be basically realized, and the original status of surrounding rock can be maintained.

2. Cases that Are Incompliant with Stress Independence

Figure 2.17 is the scene of continuous collapse of the street pavement caused by improper construction of an underground project, which caused great economic loss and casualty and adverse impact on society.



Fig. 2.17 Continuous collapse of street pavement caused by underground works (mutual impact)

Fig. 2.18 Accident rescue scene



At 15:30 on May 18, 2006, a two-layer cave dwelling collapsed as shown in Fig. 2.18. Some of the workers engaged in dismantling houses on the site were buried underground. The remaining workers immediately called more than 20 fellows working on another construction site to help. They ran into the accident site to rescue their buried colleagues. To their surprise, another disaster occurred about 5 to 6 min later, and injured 26 and killed 5 of them. Among the 26 injured persons, some suffered from visceral hemorrhage, and others suffered damage of brains, lumbar vertebra and fracture of shanks. This accident aroused attention to safety technical issue (stress independence issue) during the process (even dismantling) of multiple-arch tunnels and arch bridges (especially two-way curved arch bridges).

As shown in Figs. 2.17 and 2.18, lessons should be learnt from the safety accidents of continuous collapse of street pavement or dismantling process caused by underground works (lack of mutual independence). Mr. Leonhardt, the famous German bridge expert on stress independence, strongly emphasizes the ideological structural construction.

2.3.2 *Stress Independence for Design and Construction of Multi-arch Tunnels*

1. Influence of Tunnel Span on Surround Rock Stability

Assume the initial ground stress of surrounding rock is P_0 and the radius of continuous medium surrounding rock is a , an then the stress distribution near a round tunnel is expressed as

$$\begin{cases} \sigma_r = P_0 \left(1 - \frac{a^2}{r^2} \right) \\ \sigma_\theta = P_0 \left(1 + \frac{a^2}{r^2} \right) \end{cases} \quad (2.2)$$

Under deep-buried condition, according to the arch theory of М. Протодяконов, the pressure imposed by fractured surrounding rock media on the roof of the multi-arch tunnels is expressed as:

$$P_v = \frac{2a}{3a_1f} (3a_1^2 - a^2) \tag{2.3}$$

where a_1 is half of the tunnel span; and a is the bottom width of the tunnel.

As shown in Eqs. 2.2 and 2.3, for both single tunnel and multiple-arch tunnels, the approximate analytical solution of surrounding rock stress or tunnel structural stress is directly proportion to the square of the span. It is very important to improve the design and construction of multiple-arch tunnel to allow the pillar and the surrounding rock to form an integrity. The original status of surrounding rock can be basically maintained, because the pillar can reduce the span and increase the stress independence.

2. Traditional Section

Now, there are many completed two-arch tunnels with the integral curved pillar sections as shown in Fig. 2.19. The pillar thickness is generally 1.8–2.0 m. The characteristics is that the pillar is not only connected with the primary support of the roofs of left and right tunnel, but also connected with the secondary lining and waterproof layer of the two tunnels. The disadvantages mainly include:

- (i) The top of the pillar is not satisfactorily compacted during construction of the left and right tunnels, and some clearances exist between the pillar and the surrounding rock; in this case of poor independence, the span of the tunnel is increased and mutual impact exists between the surrounding rocks of the two tunnels;

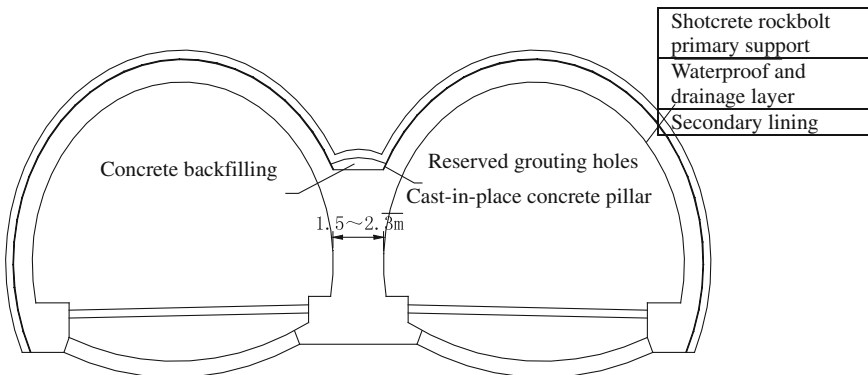


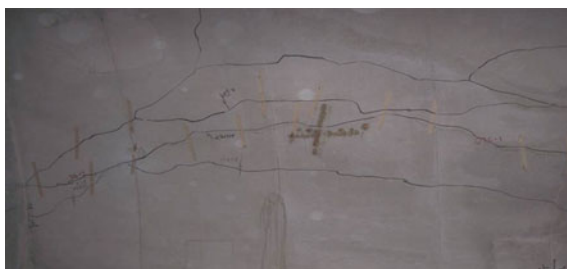
Fig. 2.19 Schematic diagram of traditional section

- (ii) The roofs of the two main tunnels are supported by the pillar. One of the tunnel may be displaced due to unsymmetrical loading, which may result in impact on the internal stress of the other tunnel. Such mutual impact will cause structural stress dependence of the left and right tunnel, and increase the difficulty of structural design;
- (iii) The drainage channels may be blocked after pillar and roof grout injection. This is a main reason for water leakage of the pillar, which will threaten the durability and safe operation of the tunnel;
- (iv) The stress condition of the integral curved pillar is very complicated and many construction procedures are required. The surrounding rock will be disturbed several times. The secondary lining is not applied at the same time as construction of the pillar, and hence construction joints between the pillar and secondary lining of main tunnels are usually introduced.

The main disadvantages of such structures include lining cracking and water leakage. There will be longitudinal or circular cracks on the pillar and water leakage at the connection between the invert and the pillar. If the stress on the roofs of the two tunnel is uneven, the entire structure will fail or will suffer from serious diseases.

Feiyuze Tunnel is a 215-m-long double-span tunnels with the maximum cover depth of 71 m. The tunnel goes through class V surrounding rock. The lithology is predominated by Mid-Triassic system yellow grey and dark grey pelitopsammite sandwiched by thin layers of dark grey fine sandstone and gravelly soil. The rock mass is fractured and now gravel like due to tectonic impact. The rock is severely weathered in most parts and moderately weathered in local parts with fissure developed. The underground water is fissure water in bedrock. According to the original design, the three-pilot-tunnel method would be used for the tunnel. The face of up tunnel should advance ahead of the down tunnel for no more than 30 m. During the actual construction, the up tunnel was excavated at first; the excavation of down tunnel was carried out after secondary lining of the up tunnel was finished. From November 2004, during excavation of down tunnel, many cracks were observed on the secondary lining of up tunnel as shown in Fig. 2.20. The field measurement report presented by the construction contractor and the testing conclusion of the third-party testing organization demonstrated that the development of cracks had become basically stabilized within a short time.

Fig. 2.20 Distribution of cracks on secondary lining



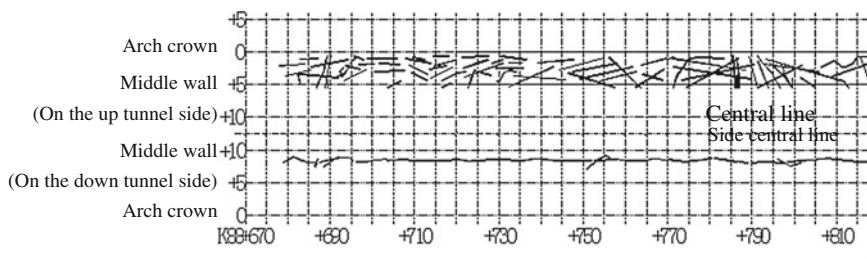


Fig. 2.21 Crack distribution diagram

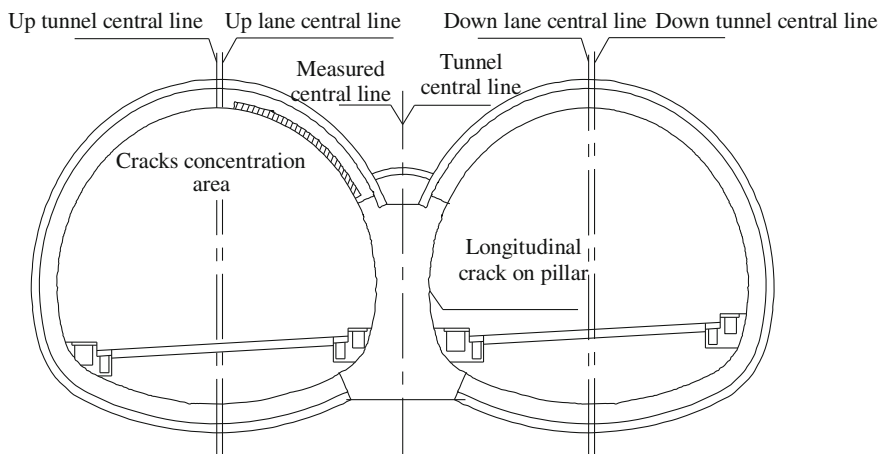


Fig. 2.22 Diagram of crack concentration area

The actual distribution of the cracks was shown in the Fig. 2.21. The longitudinal direction refers to the longitudinal direction of the tunnel (5 m each grid 5 m), while the radial direction is perpendicular to the longitudinal direction (the roof crowns of the up and down tunnels are taken as the 0 points; 5 m each grid). According to the distribution in Fig. 2.21, the cracks concentrated at the connection between the roof of up tunnel and the pillar. There was one longitudinal crack on the down side of the pillar. The cross section is shown in Fig. 2.22.

In fact, the basic cause for longitudinally distributed cracks of Feiyuze Tunnel is the inappropriately application of integral double-span structure under poor geological conditions. In addition, it is not proper to excavate the up tunnel first and then excavate down tunnel after completion of secondary lining of the up tunnel.

3. Improvement of the Sections

The composite curved pillar structure is the improved multi-arch tunnel section form. As shown in Fig. 2.23, the secondary linings of the left and right tunnels dose

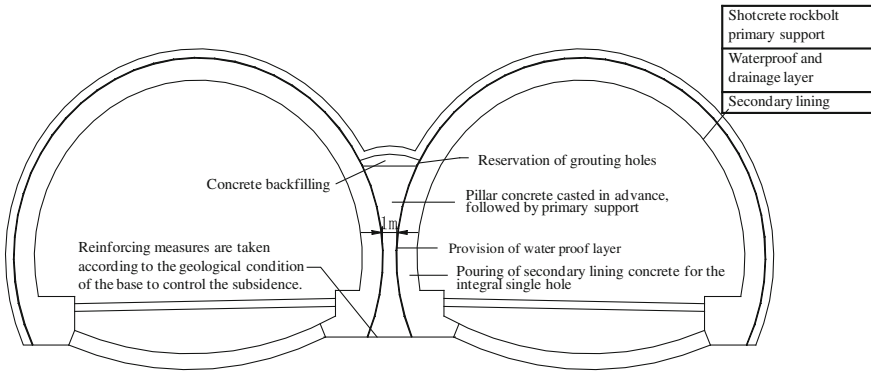


Fig. 2.23 Schematic diagram of improved section

not rest on the pillar, but form their own circles independently. Therefore, the single-tunnel integral section construction method is used for both the left and right tunnels. The main advantages include:

- (i) The waterproof and structural design should be considered together to improve the waterproof effect of the lining;
- (ii) Without weakening the structure, the secondary linings on the two sides should form independent circles. The stress on the left and right tunnel lining is clear and negative mutual effect is reduced.

There are also 3 disadvantages:

- (i) The top of pillar is difficult to compact during excavation and lining installation of left and right tunnels, and hence clearance may exist between the pillar and roof, which results in increased surrounding rock span of the tunnel. The surrounding rocks of the two tunnels have negative mutual impact due to poor independence.
- (ii) Construction procedures of the pillar are very complicated.
- (iii) The drainage channels of the geotextile layer can be easily blocked when inject grout into the pillar, in which case the waterproof effect may not meet the requirements.

The structural form shown in Fig. 2.23 was adopted for Jiulong Tunnel of an expressway. Reinforcement concrete structure was adopted for the secondary lining of the tunnel, on which cracks were observed. During construction of the tunnel, collapse and cavity on the roof were observed at the section K6 + 265–295 of the left tunnel. The surrounding rock in the collapsed sections is of class IV–V. The top headings of the two tunnels were 50 m apart; the top heading was 30–40 m ahead

of the bench. Installation of secondary lining was 50–100 m behind the primary support. There was fault fracture zone in the cracking section of the left tunnel but not in the right one.

Though tunnel deformation monitoring was carried out during construction of Jiulong Tunnel and secondary lining was installed after deformation converged, the precision of monitoring could only solve the stability and failure issue of surrounding rock. However, since the precision was low, the cracking issue (rigidity issue) could not be solved, especially for the secondary lining. If the primary support of multi-arch tunnel is strong enough to allow the deformation of primary support and surrounding rock to be compatible and adjusted automatically (grouting and other measures may be taken for reinforcement during the period), the secondary lining cracking problem can be satisfactorily solved if the secondary lining is provided after completion of proper stabilization.

4. Optimization of the Section

A multi-arch tunnel was located on a first-class highway (a connecting line of an expressway) and was the key project of highway. The lithology that the tunnel went through is predominated by the siltstone, pebbly sandstone, and conglomerate, locally sandwiched with mud stone. The surrounding rock is of class III–IV. The hydrogeological condition in the measured area is simple. The underground water includes only quaternary pore water and bedrock fissure water. The engineering condition of the tunnel is complicated. The surrounding rock has highly developed joints and chaotic distributed attitude. In addition, eight fault fracture zones went through the site, and the widest one was 22 m wide. There is an air-raid shelter 3 m above the tunnel. The underground water is abundant but unevenly distributed.

The tunnel section was optimized according to the multi-arch tunnel as shown in Fig. 2.24. Construction of the middle pilot tunnel was carried out at first. Then rockbolts or small grouted pipes was adopted to reinforce the surrounding rock at

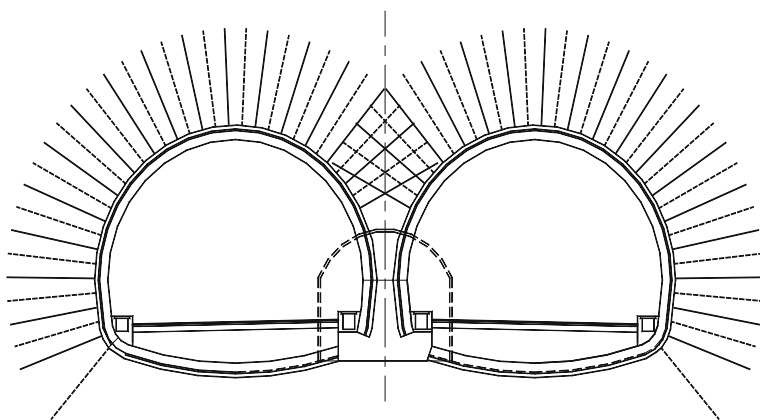


Fig. 2.24 Schematic diagram of optimized section

the roof of the middle pilot tunnel according to the condition of the surrounding rock, followed by the construction of reinforcement concrete pillar. The pillar reinforcement was integrated with the rockbolts or small pipes on the roof to form an integral part as a span-reducing and supporting method. If the surrounding rock was weak, reinforcement such as foundation expansion or base grouting were taken for the pillar, so as to greatly reduce the mutual impact and make the stress of primary support clear during construction of left and right tunnels. In this way, basic independence of stress is guaranteed.

The optimized section of multi-arch tunnels is in accordance with the emphasis laid by Leonhardt, the famous German expert in bridges, on the structure construction. As he addressed in his book titled *Construction Principle of Reinforced Concrete and Pre-stress Concrete Bridges*, “good construction details are more important than complicated calculation for the performance of a bridge”. Good construction details of a tunnel are very important to the stress state of the tunnel.

2.3.3 Stress Independence for Design and Construction of Small-clearance Twin Tunnels

1. Common Construction Measures for Small-Clearance Tunnels

The rock pillar is usually subjected to disturbance several times during construction of small-clearance tunnels in soil or weak loose surrounding rock, which is much worse than that of construction of single tunnel. Therefore, the stress on the rock pillar will be much higher on the side under construction than the other. In particular, when the adjacent partial excavation of the two holes passes a certain unsupported section, the middle wall rock pillar may easily result in wedge-shaped failure. If treatment is not provided in time, there may be disturbance or even collapse as shown in Fig. 2.25. The commonly used engineering measures include:

- (i) Simultaneous excavation of the two tunnels should be avoided. If excavation of the two tunnels is inevitable, a distance of the two faces $L \geq D$ (D is the diameter of one tunnel) should be kept. Furthermore, if the wedge formed above the pillar is composed of soil or weak loose surrounding rock, the outer side of each tunnel should be excavated first; if the wedge is stable soil or rock, then the inner side should be excavated as shown in Fig. 2.25. If the wedge is the stable hard rock, then the middle-pilot-tunnel ahead method should be used as shown in Fig. 2.26.
- (ii) During construction, the tunnel whose face advances ahead should be utilized to inject grout to reinforce the rock pillar and nearby surrounding rock.
- (iii) The construction should be carried out as quickly as possible; the closing of lining structure should be done immediately after construction is done to gain more time.

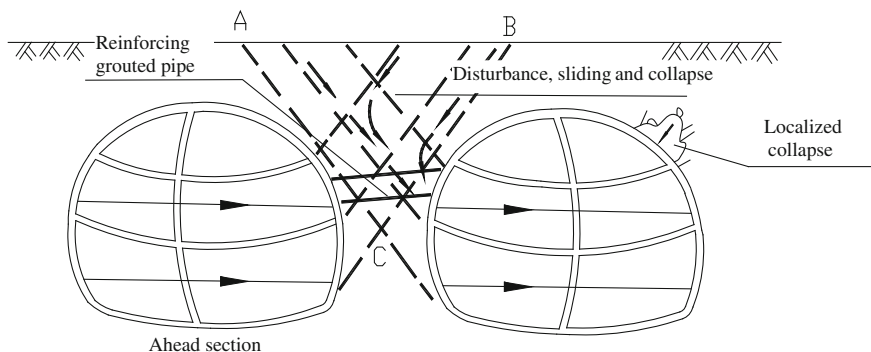


Fig. 2.25 Reinforcement of rock pillar

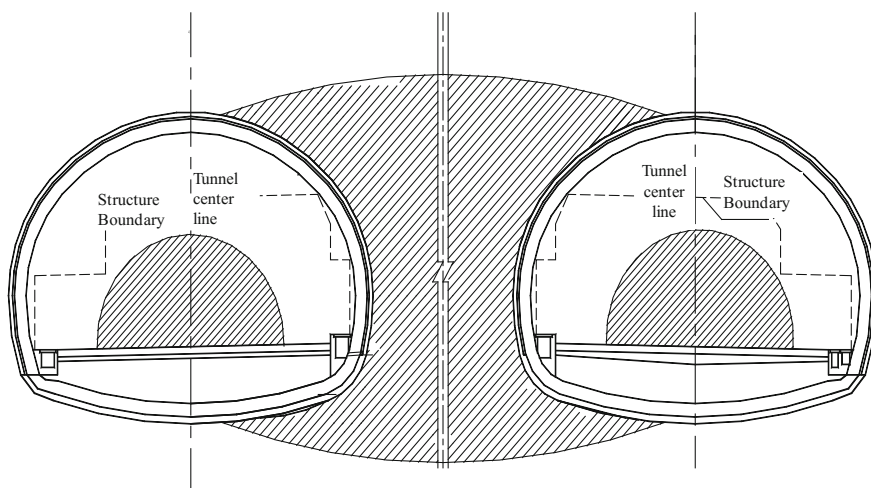


Fig. 2.26 Middle-pilot-tunnel ahead method and reinforcement of rock pillar for small-clearance tunnel in hard rock

- (iv) For small-clearance tunnels in fractured surrounding rock, the unilateral pilot tunnel method and the top heading and bench method with unilateral pilot tunnel method should be used. The shared characteristics of the two construction schemes includes that the rock pillar should be supported and reinforced as soon as possible. Reinforcing measures include rockbolt, grout injection and tensioned cables.

For small-clearance tunnels in good rock, though the rock pillar is subjected to disturbance several times, the failure mode of the rock pillar will be much better

than that of small-clearance tunnels in soil or weak loose surrounding rock. If the middle-pilot-tunnel ahead method is used and rockbolt, grouting and grouted tensioned cables are applied to reinforce the rock pillar, the result will be more satisfactory (see Fig. 2.26).

Therefore, economical and feasible design and construction method, or comprehensive application of several methods, should be chosen according to the comprehensive consideration of the geological condition. However, the comprehensive application of reasonable construction methods or several construction methods should follow the determination principle for construction tunnel methods.

2. Case study of stress independence for small-clearance tunnels

During construction of small-clearance tunnels, mutual effect between the two tunnels should be emphasized to guarantee certain independence of stress. Now, during excavation of small-clearance tunnels, the pilot tunnel with reserved smooth blasting layer is the most common construction method. Please see Fig. 2.27 for the specific excavation sequence. This method applies to class I–III surrounding rock with satisfactory self-stabilizing capacity. In such a case, the full-section excavation is possible. Given the free face of the pilot tunnel, the explosive consumption for successive excavation can be greatly reduced, and the disturbance to the rock pillar caused by blasting is also reduced. Figure 2.28 is the specific application case of such method in small-clearance tunnels.

If the pilot tunnel method with reserved smooth blasting layer is used for excavation of small-clearance tunnels, attention should be paid to the location of the pilot tunnel. A certain distance should be kept between the rock pillar and the location of the pilot tunnel (second tunnel), as indicated by the dotted line in Fig. 2.27, to reduce the impact of blasting to the rock pillar. If the pilot tunnel method is used for short small-clearance tunnels, one tunnel should advance at first, followed by the other. Supporting and construction methods of the first tunnel should be the same as the single tunnel. During the excavation of the second tunnel, the rock pillar should be protected to minimize disturbance. This is because that the stability of the rock pillar poses a direct impact on the stability of the small-clearance tunnels. In many cases of failure of small-clearance tunnels, the failure generally resulted from the wedge instability of the rock pillar. Hence,

Fig. 2.27 Excavation scheme of small-clearance tunnels using pilot tunnel method

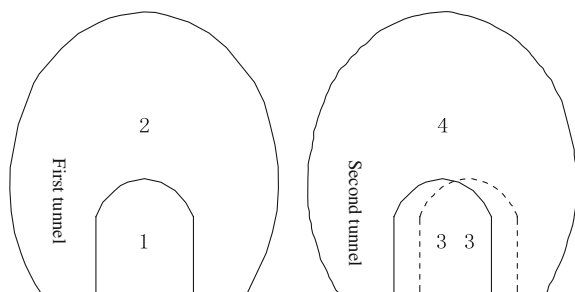




Fig. 2.28 Field photos for excavation of small-clearance tunnels using pilot tunnel method

a certain distance should be kept between the rock pillar and the pilot tunnel during the construction of the second pilot tunnel. The weak blasting should be used to minimize disturbance to and maintain stability of the surrounding rock.

When the design code is not available in 1990s, the middle pilot tunnel was first excavated and reinforcement of the rock pillar was conducted for Zhaobaoshan Tunnel in Ningbo as shown in Fig. 2.29. The main reason is that the integral strength of the surrounding rock is high. It is proper to use the middle pilot tunnel appropriate advance partial excavation method for the small-clearance tunnel in rocky surrounding rock, followed by reinforcement of middle wall rock pillar.

Hence, the core content of stress independence design and construction for small-clearance tunnels is the reasonable construction sequence of small-clearance tunnels. A single one or the combination of several reasonable construction sequences should be used depending on the geological conditions, to “basically maintain the original status of strata (surrounding rock)”, and improve the stress independence of small-clearance tunnels.

Fig. 2.29 Zhaobaoshan tunnel in Ningbo



2.4 Deformation Compatibility Control Technique

2.4.1 *Necessary Conditions for Deformation Compatibility Control*

To ensure satisfactory stress performance of the structure system, it is necessary to properly choose reasonable structure patterns, design methods and construction sequences. Sometimes, whether an innovative structure scheme and structural system can be successful highly depends on reasonable application of structural construction measures, design methods and construction sequences. A structural system is composed of different components, which play different roles in the structural system. If the overall structure has breakthrough, higher requirements will be placed on details of the structure which might be the key points to designs. Generally, the overall innovation of structure will be narrowed down to the innovation of some key details. If certain key details are solved successfully, the overall performance of the structure will be greatly improved. As stressed by Haifan Xiang, the famous Academician on bridges, good construction details of a bridge is more important for the structure performance than complicated calculation" [16].

During structure design, unprecedented structures such as mountain tunnels, shield tunnels and large underground caverns may be created with the assist of the basic mechanics concept. With the development of computer technique and mechanics analyzing measures, the structural engineers may carry out calculation and analysis of stress behaviors of new structural systems using advanced calculating theories and tools. The key point is that the stress behavior of the structure should be accurately evaluated to meet the requirement of design calculation.

The reasonable structural construction measures, design method and construction sequences that meet the deformation compatibility control are the necessary conditions for transfer of the force, deformation and energy of the structural system along the pre-designed path. The mechanics principle of underground works includes "reasonably exerting self-bearing capacity of surrounding rock" and "basically maintaining original status of the surrounding rock". The stable equilibrium equation is P (mutual supporting force of surrounding rock) ($0 \uparrow$ hazard control) + T (supporting force) ($\downarrow 0$) $\geq P_0$ (original internal force). Note that content in the parentheses is just provided to facilitate understanding rather than as supplementation to the expression. If there are strict requirements for ground deformation, then the deformation compatibility control status must be achieved and followed during construction. If there is no requirement for ground deformation, the deformation compatibility control can be properly adjusted to facilitate construction. The mechanics response is closely related to the mechanics transfer path caused by the construction sequences. The essence of tunnel design and construction is the issue of timely and effective control of the stability of weak surrounding rock. The comparison of subsidence mitigation effect of inert grout and low water-cement ratio grout in the shield construction show that the structural construction measures that meet the deformation compatibility control requirements

play a very important role in effective transfer of energy and deformation in a structural system. (1) Inert grout does not meet the deformation compatibility control requirements (except for the stable sealed medium surrounding rock); as the self-bearing capacity of the weak surrounding rock (soil) cannot be effectively recovered or mobilized by refilled medium, it is difficult to maintain the original equilibrium stability status. For instance, the train of Guangzhou Metro cannot run stably in a shield tunnel filled with inert grout in hard rock. Then the operation becomes stable after the sodium silicate was injected and solidified. Water leakage and instable subsidence was induced a shield tunnel of Line I of Shanghai Metro due to usage of inert grout in soft. (2) Grout with low water-cement ratio meets the deformation compatibility control requirements; the filled grout may effectively exert or recover the self-bearing capacity of weak (soil) surrounding rock, or at least maintain the original stable equilibrium status. For example, there was very little water leakage problems after the soft soil was filled with grout with low water-cement ratio in later stages of Shanghai Metro construction; the shield tunnel became stable after filling this grout for Hangzhou Qianjiang Tunnel.

2.4.2 Measures of Deformation Compatibility Control for Mountain Tunnels

For design and construction of mountain tunnels, attention is mainly paid to smooth blasting, rockbolt-shotcrete support and field monitoring of the NATM. However, very little attention has been paid to the effectiveness of bearing structure layer, timeliness of formation of bearing structure layer, and space stability (the time-space effect, the space stability during construction in particular) during construction of underground works. No matter what design calculation method is used, the strata structure method concept (related to “rock bearing theory”) should be adopted for construction methods and process control measures, so as to allow the strata-support interaction to reach the “stable equilibrium and deformation compatibility control” status, and eliminate any risks or potential hazard, and ensure stress safety during construction as shown in Fig. 2.30.

1. When the stability of surrounding rock is poor (soil and fractured rock mass that requires no blasting), excavation and support should be carried out simultaneously; the longitudinal step for an excavation-support cycle should be controlled between 1 and 2 m, to avoid deformation of surrounding rock and collapse. As shown in Figs. 2.30a, b, c, d and e, the key point is the effectiveness of bearing structure layer, especially the space stability during construction (mechanics and deformation control).
2. When the surrounding rock is stable and rock mass is hard (requiring blasting), the tunnel section should usually be excavated at first, followed by support installation. The full section should be excavated at one time if possible.

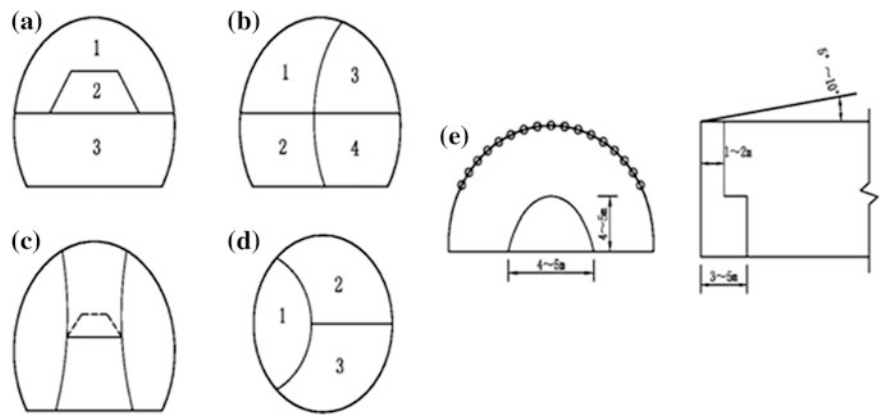


Fig. 2.30 Control of space stability for excavation and support of mountain tunnels under poor geological conditions

For large collapsed cavity in mountain tunnels, if there is no strict requirements for ground deformation, forepole umbrella and steel blank may be used on the outer periphery of the tunnel to form a shell as bearing structure. Light-weight materials should be filled into the collapsed cavity to control deterioration of the cavity or block falling, to reduce load on the support and control adverse effect of collapsed cavity as shown in Fig. 2.31. The above measures, in essence, aim at guaranteeing $\Delta U > \Delta T$ by forming a shell using the forepole umbrella and steel blank as a means of supporting, and thus improve the system resistance working ΔU . Light-weight materials are filled to control deterioration of collapsed cavity and block falling, and reduce supporting load and work of load ($\Delta T - P\Delta S_1 + W\Delta S_2$). In this way, the adverse force, and adverse transfer of energy to and concentration of energy at the weak parts of the structure can be avoided.

Figure 2.32 is the collapse scene of Yongjia Tunnel. The roof collapsed when excavation reached K16 + 183 tunnel face. The longitudinal and horizontal width

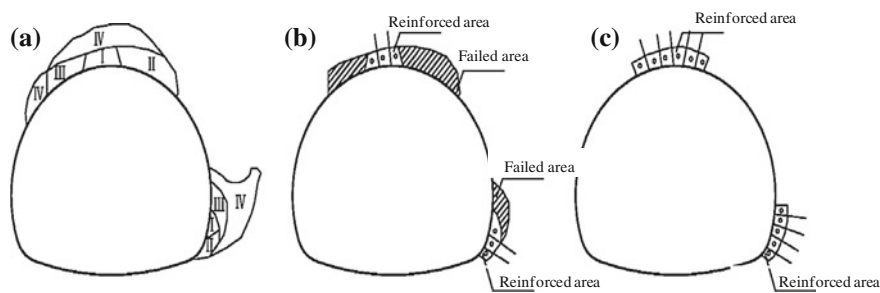


Fig. 2.31 Evolution of space stability of mountain tunnels during excavation and supporting under general geological conditions

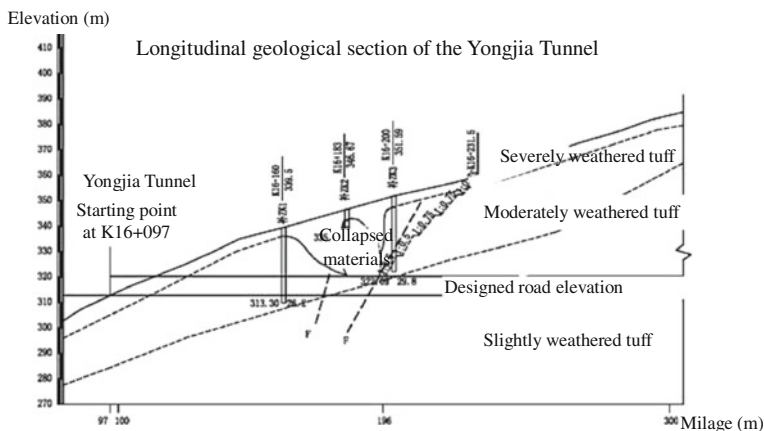


Fig. 2.32 Sudden collapse of about 2,420 m³ in Yongjia Tunnel during excavation

of collapse was about 11–10 m respectively; the height and area were about 22 m and 2420 m³ respectively. The geological survey and investigation showed that there is a buried fault zone near K16 + 183 tunnel face, stretching from north to east and inclining towards northwest. There were many fractured interlayers embedded in the rock and most of the layers were thick. The rock of fractured sections has poor stability. The remaining thickness of the fractured layer exposed at the collapse was 3.5 m underlying another 4.0 m. The collapses materials was located right below the cavity.

The treatment scheme of tunnel collapse is to use forepole umbrella to cross the collapsed area. The cavity behind the umbrella was filled with foam concrete. The next step was started after the concrete gained 70 % of its final strength. After filling with foam concrete, further collapse of fractured rock mass in the cavity was prevented, to reduce load on the forepole umbrella and guarantee safety of surrounding rock and supporting structure.

2.4.3 Measures of Deformation Compatibility Controlling for Shield Tunneling Method

The engineering structure should not only achieve formal equilibrium status in terms of art, but also ensure the system in the stable equilibrium status in terms of “force, deformation and energy”. To ensure “force, deformation and energy” is transferred and transmitted in the designed path is the basic requirements for a reasonable, stable and safe structure, which is also the basis for preventing hazardous process of the design form. According to the specific conditions, the underground structural behaviors can be better controlled based on one or several elements among “force, deformation and energy”. To ensure the “stable equilibrium

and deformation compatibility control” of the structure, it is required to control not only the target status but also the process of achieving structural safety and reasonability, otherwise there will be structure instability or failure. The specific measures that guarantee the target status should be adopted during construction based on the following description of shield tunnel construction cases, in order to ensure stable equilibrium in the entire process of tunnel construction.

Now, most of the design and engineering consultation only focus on load after completion of integral structure. The design and feasibility demonstration can be carried out with reference to the design code. The design measures can be taken to design structure components. Such design approach is only feasible when both the structural form and engineering environment are simple. However, if the structural form and engineering environment are complicated and changing, the impact of load variation on structural stability cannot be taken into consideration by this design approach. The shield tunneling method boasts high-degree technical integration and complicated construction process, with the tunnel line crossing complicated geological environments. The integral structure design of shield tunnel should match the structural construction measures, design method and actual construction process control, in order to guarantee the stable equilibrium and deformation compatibility control of shield tunnels and achieve the target of design and construction. Mechanics response of tunnels is closely related to stress paths induced by construction sequences. Therefore, the essence of tunnel design and construction is to guarantee the timeliness and effectiveness of stability control for weak surrounding rock.

During shield tunnel construction, factors such as over excavation and shield tail void will result in a space formed in the strata which is larger than the space taken by the tunnel segments. In another word, there will be a clearance between the tunnel segments and the strata. The strata deformation and posture stability of tunnel segments can only be controlled by grout injection. The grout can be divided into hard grout and inert group depending on percentage of cement in the grout. The early strength and final strength of hard grout are both higher than those of inert grout. However, better grouting pump and pipelines are required for the former. In China, inert grout, with low early strength and final strength, is usually used for tail void grouting to reduce the grouting equipment and grout expense. As shown in Fig. 2.33, if the grout cannot solidified in time, the grout will flow into the earth chamber via the clearance between the shield and surrounding rock. The liquid grout with high liquidity will result in noticeable pressure fluctuation in the earth chamber because the pressure maintaining effect of EPB(earth pressure balance) shield is not satisfactory. In such a case, significant impact will be pose on the stability control of the excavation face.

Therefore, for tail void grouting, hard grout with high early strength that can solidify quickly should be used to stabilize the segments within a short time, and allow the counter-force of shield jacks that acts on the tunnel segments can be transferred to the strata and thus restrain deformation of the segments ring (the oval deformation will render the circular segments into an unfavorable stressed status, and may even cause ground deformation). In this way, the designed alignment can

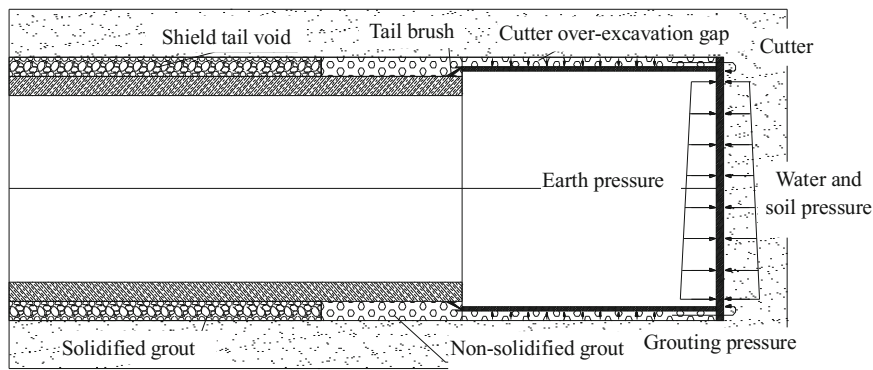


Fig. 2.33 Schematic diagram of shield tail void and grouting

be guaranteed with the assist of the protective layer outside the tunnel segments. In addition, the clearance will be filled to eliminate disturbing factors of the above equilibrium status, which enable the strata-segments interaction to reach the stable equilibrium and deformation compatibility status. The quality and safety of the shield tunnel can be guaranteed using the construction sequences.

Depending on whether the deformation is compatible as illustrated in Figs. 2.34 and 2.35, we can regard the tunnel segments as rigid material, and the soil mass as the semi-flexible material. The low early strength of inert grout makes it flexible material. If the rigidity of these materials are not compatible, the shield tunnel segments may be instable and the soil mass deformation can hardly be compatible. The early strength of hard grout is high, which is similar to that of the semi-rigid and semi-flexible materials. In this way, the shield-strata interaction can easily reach the stable equilibrium and deformation compatibility control status. Now, the tail void filling of inert grout followed by segment injection of quick-hardening grout can be used. The principle is similar to that of injection of hard grout.

There are always unfilled voids after excavation of soft strata, which changes the original stability status and stress condition of the strata. To basically maintain the

Fig. 2.34 Schematic diagram of segment injection of quick-hardening grout

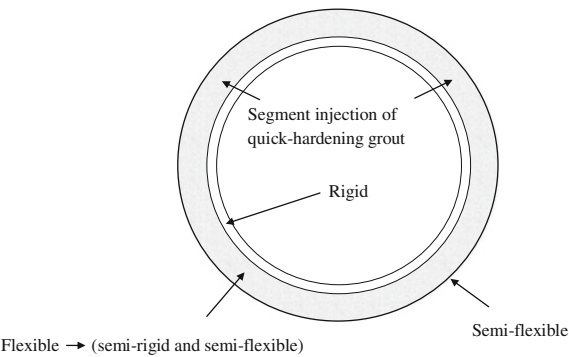


Fig. 2.35 Even elastic tires can better control deformation compatibility and even stress of wheel hubs than the rubber pneumatic tires



original stress and deformation conditions of the strata, pre-reinforcement must be provided. In order to control strata deformation, tail void grouting with strength and rigidity higher than those of the undisturbed soil is required to meet the requirements of complete filling and solidification. For relatively stable strata with no confined water, the concrete pump may be used to inject fine sand mixture with satisfactory pumpability and specific gravity basically equal to that of the undisturbed soil to fill the shield shell and tail voids. In addition, for muddy soil or clay strata with low permeability coefficient, soils will have consolidation after the shield pass. In such a case, refilling of grout is required via the segments to eliminate effect of the strata consolidation, so as to maintain the equilibrium stability of soil-structure interaction and meet the stable equilibrium and deformation compatibility control requirements for structures.

2.4.4 Segment Grouting with Quick-Hardening Grout

Generally, the grout can be divided into the following three types depending on the lasting time of their plastic state: the special plastic grout whose plastic state can last 5–30 min; the inert type with extremely low early strength and hydration time longer above 30 s; the quick-hardening type with high early strength and hydration time below 20 s.

For tunnel segments grouting, the special plastic grout should be used. The grout is injected in gradually with expansion of the filling region. Though the filling pressure is low, the grout may also fill a large area. The plastic state grout gradually moves forward till the entire shield tail voids are filled up. As the viscosity of the grout is quite high, the grout can hardly diffuse into surrounding soil mass. Such grout can effectively fill up the voids at higher locations. During shield construction in China, inert grout (slow solidification type) is usually adopted for tail void grouting. With abundant underground water, the filling behavior, filling range behavior and solidification strength of the grout can hardly be satisfactory. The strata filling effect may not be as ideal as expected.

If the segment injection of quick-hardening grout is used, there will be no loss of liquidity. The grouting is only provided within the limited scope. In another word, quick-hardening and plastic ingredients can be added in the grout so that the grout injection can be controlled within the limited area. After the tail voids are filled with grout, the grout is expected to solidify quickly with strength similar to that of surrounding rock/soil mass. Therefore, according to regulations on early strength of quick-hardening grout, the compressive strength after 1 h should be roughly 0.1 MPa. If the hydration time of quick-hardening grout is too short, the liquidity will be lost before completion of filling, which will result in poor filling effect.

According to whether the deformation is compatible as shown in Fig. 2.34, the segments are classified as the rigid material and the soil mass as semi-flexible material. The early strength of inert grout is low and therefore classified as the flexible material. Non-compatible rigidity may result in instability of tunnel segments or incompatible deformation of soil strata. The early strength of quick-hardening is quite high, as that of the semi-rigid and semi-flexible material. In such a case, the system may easily reach the stable equilibrium and deformation compatibility control status.

After tail voids grouting, quick-hardening grout is refilled at an interval of several rings, so as to form a grout vein framework similar to a bucket hoop. This may facilitate even and balanced stress on the hinged segments, and improve stability of segments. This can also improve the stability through overall application of pre-stress. However, the stability issue cannot be solved by theoretical analysis. The balanced and unbalanced stress on the shield tunnel, and whether the tunnel is restrained by the radial water and soil pressure is quite similar to the stress principle of an empty wood bucket with or without a hoop. When a vertical force is downward applied to the upper part of an horizontally-laid empty wood bucket with or without a hoop, load bearing of the bucket is remarkably different. This model can be used to simulate the unbalanced force applied on the perimeter of the tunnel. The bucket hoop applies a radial constraining force on the bucket, which is very much like the even water and soil pressure on the tunnel in the strata. Similar with a wood bucket without a hoop, if a tunnel is free from constraining force on the perimeter, the deformation after loading cannot be compatible, like the localized stress. Through adjusting the tightness of the hoop, the value of constraining force on the perimeter can be adjusted. Due to such constraining effect, the bucket deformation is controlled and all wood plates form an integral part. Since the outer side is bigger than the inner side of the wood bucket, the contact sides of the wood plates and the connecting chisels jointly provide counterforce, so that the wood bucket in stable and deformation compatible status can bear high pressure. The wood plates of a hoop-less wood bucket are not constrained, and cannot generate compressive pre-stress. In addition, the deformation of each wood plate is not constrained or limited. Once a wood plate develops non-compatible deformation and comes off the bucket, the overall stress equilibrium system will be destroyed. Take the tunnel loop formed by segments as an example (Fig. 2.36), the size of each segment ring is: inner diameter: Φ 5.5 m; outer diameter: Φ 6.2 m; length: 1.2 m; thickness: 0.35 m, with the self-weight of 19.3 t. In another word, the

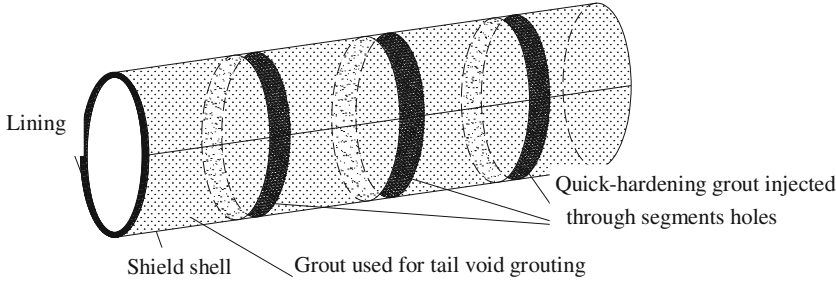
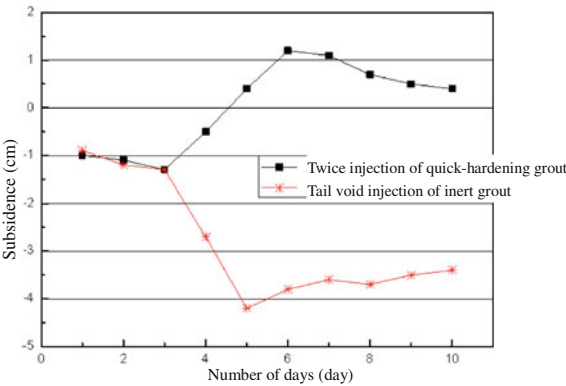


Fig. 2.36 Stability of hinged segments in weak strata

Fig. 2.37 Comparison between controlling effects of inert and quick-hardening grout on ground deformation



self-weight of the upper half of segment ring is 9.65 t. If there is no effective radial force to support the lower segments and constrain the overall deformation, then it will be difficult for the segments to form a loop vertically even when they are only subjected to the self-weight of the upper segments.

If the shield crosses important pipelines or buildings, the quick-hardening grout is usually additionally injected through the segments to control ground deformation and impact on buildings/structures. The shield tail voids can be satisfactorily filled up by adjusting the solidification time of the grout, and then ground deformation can be controlled. Figure 2.37 shows the comparison between controlling effects of inert grout and quick-hardening grout on ground deformation.

References

1. Wang MS (2010) Tunneling and underground engineering technology in China. China Communications Press, Beijing
2. Yu SH (2000) Tunnel construction. China Communications Press, Beijing

3. Zhu HH, Sun HY, Yang JH (2007) Highway tunnel surrounding rock stability and supporting technology. Science Press, Beijing
4. Zhu HH, Zhao Y, Shang YQ (2011) Underground engineering equilibrium stability theory. Chin J Undergr Space Eng 7(2):317–328
5. Mortazavi A, Katsabanis PD (1996) Modelling of blast hole expansion and explosive gas pressurization in jointed media. Int J Rock Mech Min Sci Geomech Abstr 35(5):497–498
6. Zhu HH, Shang YQ, Yang JH et al (2011) Equilibrium stability of underground works during entire process. China Communications Press, Beijing
7. Chongqing Communications Technology Research and Design Institute Co. Ltd (2004) Design code for highway tunnels. China Communications Press, Beijing
8. Lin CT, Amadei B, Jung J et al (1996) Extensions of discontinuous deformation analysis for jointed rock mass. Int J Rock Mech Mining Sci Geomech Abstrs 33(1):671–694
9. Ke TC (1993) Artificial joint-based DDA. In: Proceedings of the first international forum on discontinuous deformation analysis (DDA) and simulations of discontinuous media. TSI Press, Albuquerque, pp 326–334
10. Kim Y, Amadei B, Pan E (1999) Modeling the effect of water, excavation sequence and rock reinforcement with discontinuous deformation analysis. Int J Rock Mech Min Sci 36(7): 949–970
11. Zhao SL, Salami MR, Rahman MS (1997) Discontinuous deformation analysis simulation of rock slope failure processes. In: Proceedings of 9th international conference on computer methods and advances in geomechanics. A. A. Balkema, Rotterdam, 473–477
12. Sun GZ (1988) Rock mass structural mechanics. Science Press, Beijing
13. He MC, Jing HH, Sun XM (2002) Soft rock engineering mechanics. Science Press, Beijing
14. Duan YS, Liu XR, Huang LW, Wu Q (2010) Monitoring and analysis of soft rock tunnel large deformation catastrophe area before and after reinforcement. J Highway Transp Res Dev 9:17–22
15. Wang MS (2004) General theory on shallow-buried tunneling technology for underground works. Anhui Education Publishing House, Hefei
16. Xiang HF (2011) Bridge concept design. China Communications Press, Beijing

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