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Stability Assessment for Underground Excavations and Key Construction Techniques

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To practitioners of tunneling and underground engineering.

Foreword

The engineering mechanics is established based on determined material properties and microstructure. However, for some engineering structures (e.g., tunnels in soft rock), material properties and microstructure continuously change, and how they change is still unknown. In addition, the constitutive relation, integration, and force transfer path of the structures are also changing. Therefore, the design and construction of engineering structures should satisfy the structural rationality and deformation compatibility control conditions when applying engineering mechanics to solve these problems. This book inherited the technical essence of loose load theory and rock bearing theory and their value in guiding the modern tunneling and underground engineering, and proposed the stable equilibrium theory of underground engineering construction and four innovative techniques. During the design and construction of underground engineering, the focus of mechanics control conditions changed from force equilibrium into the dual control of force and deformation involving both structural stable equilibrium and structural deformation compatibility. This can not only interpret the laws of structural stability and cope with the uncertainties encountered during the construction but also provide a useful attempt to solve the new-type underground engineering problems.

This book is provided as a reference for underground engineering practitioners.

Hangzhou, China

Hanhua Zhu

Preface

According to the statistics of engineering safety accidents, among the collapse accidents, the deaths in underground engineering construction account for 32.6 %, and those in pit excavation and retaining wall account for 23.9 %. Even in some developed countries, the deaths in collapse accidents caused by structural defects of traffic engineering account for as high as 11 %. The statistical result shows that the projects with a high-efficiency structural organization have fewer problems, while the projects with a low-efficiency structural organization have more problems; in the latter, the material property and microstructure change, which is not conforming to the conditions of the deformation compatibility theory, so only a reasonable structure shall be selected to meet the structural deformation compatibility control. It means that the deformation compatibility control method to solve the engineering structure stability and balance problems shall be innovated, or the engineering structure will have a big difference between the calculated result and the actual behaviors, and even safety threats will be brought about.

Through exploration of nearly two centuries, the underground engineering construction techniques have been remarkably improved. Now, for underground engineering structures, the traditional loose load theory (similar to the load-structure method) or the numerical analysis method is used for the calculation and analysis; the modern rock bearing theory (similar to the strata-structure method) is mainly used for the reasonable structure construction, construction method, and procedures. Though existing theories or methods imply the assumption of "deformation compatibility control," it is often ignored in engineering practice. The structural deformation compatibility control issue is emphasized, and the "deformation compatibility control" is taken as the explicit boundary condition, so as to allow the interaction between underground engineering structures and surrounding rock and achieve the "stable equilibrium and deformation compatibility control" status. As for simple geological environments with good surrounding rock, the underground engineering stable equilibrium can be achieved more easily. It only requires mastering physical concepts, and the selection of theories and construction methods does not matter that much. For underground engineering constructions with poor geological conditions and complicated engineering environment, engineering measures should be taken to guarantee the deformation compatibility control of structure; otherwise, the reasonable transmission or transfer path of force in the structural system will be harmed, and the structural stable equilibrium status may be changed, or even a hazardous equilibrium status may be caused. The underground engineering stable equilibrium theory together with excavation energy control technique, strong pre-reinforcement technique, comprehensive stress-independence technique, and deformation compatibility control technique can effectively solve this kind of problems.

If you have any opinions on any content of the book, please do not hesitate to contact us.

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Contents

Abbreviations

CTM	Conventional tunneling method
EPB	Earth pressure balance
FEM	Finite element method
NATM	New Australian tunneling method
STM	Shallow tunneling method

Part I
Stable Equilibrium Theory and Key Techniques for Underground Excavations

Chapter 1
Formation and Development of Underground Engineering Stable Equilibrium Theory

Abstract The underground engineering surrounding rock exists in certain geological environment, and is vulnerable to influences of tectonic and weathering process. The discontinuities formed inside the surrounding rock intersect the complete rock mass into rock blocks of different shapes and sizes. Before starting underground excavation, the rock mass is in a stable equilibrium status. Excavation will cause unload rebound deformation and stress redistribution on the surrounding rock. If the strength of surrounding rock is stronger than the redistributed stress action after excavation, the surrounding rock will not be obviously deformed or damaged. In this case, no supporting is needed, or only localized supporting is enough to maintain the stability of surrounding rock. If the strength of the surrounding rock is weaker than the redistributed stress action, the surrounding rock may be drastically deformed, which may result in instability and failure. In such a case, strong pre-reinforcement measures should be taken to ensure that the proper interaction of the surrounding rock and supporting structure meets the stable equilibrium and deformation compatibility control requirements, and thus truly "reasonably exert the self-bearing capacity of surrounding rock". In this chapter, the advantages and disadvantage of traditional load theories (loose load theory and rock bearing theory) were summarized and discussed. The physical significance of stable equilibrium and deformation compatibility control was interpreted using a simple ball-ropes model. Then these concepts were introduced into underground engineering.

1.1 Introduction

The principle of using engineering structure deformation compatibility control method to solve engineering structure safety issues is as follows:

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The engineering structure design and construction should solve the practical problems according to the structure function: *Focusing on strategy and highlighting tactics*:

1. **Strategic issues [implying the influence domination, regulation requirements routinization and hazards prevention]**
   (i) Determine the situation (determine the moving status and tendency of the object; convert uncertain factors into certain ones);
   (ii) Control hazards (least energy assumption principle, reasonability of force transmitting medium, and deformation compatibility control)
2. **Tactics issues [the faced issues are lack of connection and may not be comprehensive]**
   (iii) Equilibrium calculation (the structural stable equilibrium and deformation compatibility control are the key points; analogy to classical engineering structures [there was no mechanics calculation for engineering construction before the 18th century]; the force equilibrium issues in engineering structure design and construction [*auxiliary calculating and verification*])

The Newtonian Mechanics ($F = ma$: the structural gravity center, like the mass point, does not change) is the source of engineering mechanics. The following two conditions should be met if engineering mechanics ($F = P + T = P_0$) is used to solve the summation process $\Sigma F_i = \Sigma P_{0j}$ in engineering structure mechanics and deformation issues: (i) the *stability and environmental suitability* of stressed structure mass (construction) m and movement (deformation1) a; (2) the *reasonability* of stressed structure force transmission F; *otherwise, the engineering structure construction control measures should be taken. Conforming engineering mechanics is required after the condition that the material property and microstructure are determined.* However, for some engineering structures, the *material property and microstructure may change during stressing in an unknown way according to an unknown law*. The prerequisite for the application of engineering mechanics to solve engineering structure issues is that the engineering structure design and construction meet the *structural deformation compatibility control conditions*. The two methods for civil engineering design and construction can be described as follows: (1) *If the engineering structure stable equilibrium theory is used to solve relatively mature engineering issues, the precise analysis method ($F = P_0$, calculating equilibrium equation) may be used*; (2) *if the engineering structure stable equilibrium and deformation compatibility control method is used to solve comparatively complicated engineering issues, the studying equilibrium equation ($P + T = P_0$, studying equilibrium equation) effectively utilized for mechanics in the engineering structure design and construction can be used, so as to convert the "leaf issue" into "apple issue". Then, the precise analytical method can be used to solve the engineering issue ($F = P_0$, calculating equilibrium equation)* [1, 2].

The studying equilibrium equation effectively utilized for mechanics in engineering structure design and construction (*leaf method: overall control and detail mastering*). In the engineering mechanics analysis formula ($P + T = P_0$) corresponding to the engineering structure stressed deformation status (1) and (2) as shown in Fig. 1.1, P_0 is the sum of internal and external loads related to the structure status; T is artificial load that should be kept to the minimum; P is the mutual supporting force between structures transferred sufficiently by measures such as reasonable structure construction and construction procedures, and thus should be kept to the maximum.

The comparison of several equilibrium equations and their engineering application conditions is as follows:

$$F = ma \quad \begin{array}{l}\text{The application of Newton's second law usually}\\ \text{implies two constraint conditions}\end{array} \tag{1.1}$$

$$F = P + T = P_0 \quad \text{Engineering structure equilibrium equation} \tag{1.2}$$

$$F = P_0 \quad \begin{array}{l}\text{Engineering structure calculating equilibrium equation}\\ (\textit{apple method}: \textit{logistical analysis and precise calculation})\end{array} \tag{1.3}$$

$$P + T = P_0 \quad \begin{array}{l}\text{Engineering structure studying equilibrium equation}\\ (\textit{leaf method}: \textit{overall control and detail mastering})\end{array} \tag{1.4}$$

Given that there are constraint conditions for the application of Newtonian Mechanics Eq. 1.1, there are also deformation compatibility control conditions for engineering structure equilibrium and calculating equilibrium equations Eqs. 1.2–1.3. Therefore, for the engineering structure design and construction, only when the studying equilibrium equation Eq. 1.4 is used to analyze the deformation compatibility control conditions of the engineering structures and these conditions are achieved (the leaf issue has been converted into apple issue to solve the engineering problems), can the engineering structure stressed deformation status calculated using calculating equilibrium equation Eq. 1.3 match the actual stressed deformation status and the practice engineering issues be actually solved. *In fact,*

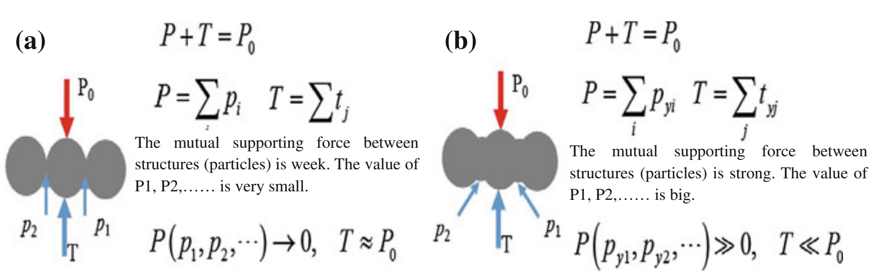


Fig. 1.1 Schematic diagram for the engineering mechanics equation

Eq. 1.3 *is the analytical method while Eq. 4 is the comprehensive method; the two are supplements for each other.*

The main tasks of study on the relation between structure and mechanics in the scope of engineering mechanics include: (1) equilibrium (mechanical characteristics); (2) physical properties (constitutive characteristics); (3) compatibility (geometric characteristics). For the first item, equilibrium, there are already sufficient researches; for the second item, physical property, the existing researches mainly focused on specimen test and theoretical deduction; for the third item, compatibility, the researches mainly focused on deformation compatibility theory and deformation compatibility assumptions. However, there are only a few deformation compatibility control methods adopted to study the relation between reasonable structure construction and mechanics in a systematic way. It is therefore hard to guarantee suitability of the structure force transmitting medium and avoid structure meta-stable equilibrium issue. The reasonable structure construction meets the structural deformation compatibility control requirements, and the influence of stress redistribution on the structural stress safety and deformation compatibility control is within a allowable scope; otherwise, the unreasonable or hazardous transfer of force may cause new or even hazardous equilibrium status, which may pose an impact on the structural stress safety and deformation compatibility control.

The key to quality and safety control during underground engineering construction is the effectiveness of the underground engineering bearing structural layer, and the timeliness of the formation of such layer, as well as the space stability (the time-space effect), that is, structural mechanics is the foundation. When we prefer to use the mathematical method to solve practical issues, we tend to ignore the fact that the physical concept may be far more important than those math expression. The researches of underground works lay much emphasis on theoretical analysis or analogies of experience or combination of the two, and satisfactory results have been achieved with any of the methods.

The authors have conducted statistics analysis on the number of successful and failed cases of underground engineering construction, and studied the scientific essence of the underground engineering theories such as loose load theory, rock bearing theory and etc.*, the related techniques, and their significance and values when they are used to guide modern underground engineering practice, based on the force-deformation status and stability of underground engineering structures.* The underground engineering equilibrium stability theory and key techniques have been explored and established as a beneficial attempt to solve the new-type underground engineering issues.

In fact, the engineering structure equilibrium and stability analysis is in an effort to solve the tendency of stability of engineering structure force-deformation status based on the precise mathematical logic analysis. The key point is to find out the controlling methods and measures to maintain the stability of the engineering structure force-deformation status through engineering structure deformation compatibility control under system control, i.e., "basically maintaining the original status of strata (surrounding rock) and the stability of engineering structure designed force-deformation status". For example, the "plane cross-section assumption" in the material mechanics simplifies the structural stress equilibrium equation. Therefore, the basic prerequisite for the use of existing engineering mechanics analysis method in solving engineering issues is to control the structural deformation compatibility and to ensure the engineering structural safety.

There are "three threats" for underground works: (1) water; (2) softness; and (3) deformation. The rectification measures are as follows: (1) water control and seepage protection; (2) collapse and instability prevention; (3) construction deformation prediction and control. The core of the three issues is also the structural deformation compatibility control. Therefore, the key issues for underground engineering design and construction include: (1) the self-bearing and self-stabilizing capacity of surrounding rock should be utilized to the maximum; (2) interaction with surrounding rock should be kept to the minimum during construction to prevent degradation or even damage of the self-bearing and self-stabilizing capacity; (3) measures should be taken to improve and strengthen the self-bearing and self-stabilizing capacity of surrounding rock.

Why did collapse accidents or even safety accident easily occur during underground engineering construction? The investigation and analysis of a lot of collapse and safety accidents during underground engineering construction shows that: (1) it is easier to determine whether the underground engineering structure system can meet the structural deformation compatibility control requirements based on "basically maintaining original status of strata (surrounding rock)" than on "reasonably utilizing self-bearing capacity of strata (surrounding rock)". If the structural deformation compatibility control requirements are met, the actual force-deformation status of the structure should be basically consistent with the designed force-deformation status. Otherwise, there is difference between the actual force-deformation status and designed status of the structure, which may causes various issues, or even unexpected issues or safety issues. (2) In design, the structural mechanics stable equilibrium during construction is often ignored; it is hard to meet the structural mechanics stable equilibrium requirements, let alone the new Austrian tunneling method (NATM). *In fact, it is required to study not only the overall stability of the structure in design, but also the stability of sub-structures during construction.* (3) Some engineers do not have enough experiences of similar construction; though they are familiarized with the NATM, they tend to ignore the structural mechanics stable equilibrium issue, which results in improper application of methods; the requirements for the NATM are actually not met. Please note that the *NATM is the concept and structural mechanics is the foundation.*

Many engineers know how to carry out engineering structural design and construction according to corresponding code. However, their study on the relation between structural and mechanics that caused structure diseases mainly focuses on mechanics equilibrium, and their study on compatibility mainly focuses on the deformation compatibility theory and deformation compatibility assumptions. However, there are only a few deformation compatibility control methods used to study the relation between reasonable structure construction and mechanics in a systematic way. It is therefore difficult to guarantee the suitability of the structural force-transmitting medium and avoid the structural meta-stable equilibrium issue.

The "*meta-stability solution*" (the design codes are met, but the structural construction is not sufficiently reasonable, and the durability and risk management may not be satisfactory) can only be avoided by comprehensively studying the relation between structure and mechanics, so as to improve the "*usable solution*" (the design codes are met; it may be suitable but conservative and even have risks or defects in special cases) to the "*optimal solution*" (cutting-edge research results or outstanding solutions at present that are better than the design codes; however, measures should be taken to avoid special defects or risks), and constantly *seek for* the true meaning of scientific issues such as study on structural design and construction. *To sum up, it should be guaranteed that each construction step should meet the stable equilibrium and deformation compatibility control requirements in the engineering structural design stage; the core indexes of engineering structural stable equilibrium and deformation compatibility control should be strictly implemented and monitored during construction process, and controlled within the allowable design range*.

In fact, the NATM is, in its essence, the design construction concept and method of "reasonably utilizing self-bearing capacity of strata (surrounding rock)". If the structural construction is not reasonable enough, there may still be special risks or even safety accidents despite the monitoring measurement or prediction control; the structural stable equilibrium and deformation compatibility control requirements should be met throughout the process of underground engineering construction, in order to "reasonably utilize self-bearing capacity of strata (surrounding rock) and meet the safety and quality requirements during entire process of construction and operation". For example, (1) if the self-bearing capacity of the strata (surrounding rock) is sufficient to guarantee the structural stable equilibrium during underground engineering construction, there is no difference whether the NATM or Conventional Tunneling Method (CTM) is adopted; (2) if the self-bearing capacity of the strata (surrounding rock) is not enough to guarantee the structural stable equilibrium during underground engineering construction, especially when the strata (surrounding rock) is in a very poor condition, the structural stable equilibrium and deformation compatibility control requirements can be met during underground engineering structure construction if the NATM is used (the excavation is carried out at the same time or after supporting measures are taken). However, if the CTM is used (the excavation is carried out before supporting measures are taken), collapse of strata (surrounding rock) may easily occur. *The advantage of underground engineering stable equilibrium stability theory is that: in the design, either loose load theory or modern mechanics (such as finite element method) is used to identify the self-bearing capacity of the strata (surrounding rock) or calculate the structural load, as well as the difference between soft rock (soil) strength value and initial rheological stress value, and the effective utilization of high-limit rheological stress value of soft rock (soil)* [3, 4], *to work out a satisfactory supporting and excavation scheme and emergency plan. The supporting and excavation are carried out during construction using the NATM. The monitoring measurement or prediction control are adopted, and the design and construction scheme is dynamically adjusted, so as to "reasonably exert the self-bearing capacity of the strata (surrounding rock)" and* *meet the structural stable equilibrium and deformation compatibility control requirements, and thus guarantee structural quality and safety of underground works.*

1.2 Physical Significance of Engineering Structure Stable Equilibrium and Deformation Compatibility Control

To understand the physical meaning of the relation between structure stable equilibrium and deformation compatibility control, the diagram for an object as shown in Fig. 1.2 can be used to straightforwardly illustrate the influence of deformation compatibility control on the stable status of structural equilibrium. The object shown in Fig. 1.2 is suspended on n ropes; there is a combined action of the forces of the ropes P_1, P_2, …, P_n and the self-weight W of the object, which results in an equilibrium status. The stable equilibrium status highly depends on the deformation compatibility control status of the combined action of acting rope force P_i and object W. The specific description is as follows:

1. When object W is subjected to static load, the combined action of P_1, P_2, …, P_n and W is in an equilibrium status. And if P_1, P_2, …, P_n are within the allowed strength limit of each corresponding rope, the system is in a stable equilibrium status. If any P_i exceeds the strength limit of the corresponding rope, which results in failure of the rope, the stress on the remaining $n - 1$ ropes will be redistributed. During the internal stress redistribution, there might be the following two cases: If the internal stress can be transferred reasonably, the stress on the remaining $n - 1$ ropes will still be within the strength limit and the system will be again in an equilibrium status; if the system structure design is improper, the system internal stress may not transfer reasonably; in such a case, the internal stress of the remaining $n - 1$ ropes will again exceeds the strength range, which may result in breakage of a rope. Repeated occurrence of this process will result in progressive failure and overall instability of the system. From the aspect of energy transfer, the above phenomenon may be interpreted as follows: due to the action of object W, the strain energy accumulated within each rope is U_i (i = 1, 2, …, n), when the structure is in a stable equilibrium status. If the internal stored energy U_i of any rope reaches the energy absorption limit and rope breakage occurs, U_i will be completely released. As the total energy of the system remains the same, the structural deformation will be redistributed. There may be two different cases: if the energy may be transferred reasonably, the remaining $n - 1$ ropes can effectively absorb all the deformation energy and the system will be again in an equilibrium status; if the structural design is improper, the work by the external force will again exceed the structural energy storage limit, which may result in breakage of the remaining $n - 1$ ropes, progressive failure and overall instability of the system.

2. The object W, when being disturbed, will deviate from its original position, and the values of P_1, P_2, …, P_n will be redistributed. Only if P_1, P_2, …, P_n is deformation compatible (i.e., the internal stress among P_1, P_2, …, P_n can be reasonably transferred), all of them being within the allowed strength range with the stable equilibrium and deformation compatibility control, can the system recover to its original position. Otherwise, if the structural design is not reasonable, the internal stress of the system cannot be reasonably transferred, and the redistribution of P_1, P_2, …, P_n may possibly cause the action force P_i of a certain rope to exceed the limit, which may result in progressive failure and overall instability of the system. This phenomenon can be better understood from the aspect of energy analysis. The disturbance from the environment will add certain energy to the structure, in this case, object W. Only when the structure is deformation compatible (i.e., the overall deformation can be transferred reasonably among the ropes), can the system recover to its original position, considering the internal energy consumption mechanism of air and structure (such as friction and damping). Otherwise, if the structural design is not reasonable, the internal energy of the system cannot be transferred reasonably, the redistribution of U_1, U_2, …, U_n may cause a certain rope not able to absorb due energy, which may result in failure or even progressive failure and overall instability of the system.
3. When object W is subjected to dynamic load action, the stress non-uniformity of P_1, P_2, …, P_n becomes even more evident, and the stable equilibrium and deformation compatibility control for the combined action of P_1, P_2, …, P_n and W becomes more complicated. This is because that the magnitude of structure input energy will vary with the form of external dynamic load action. Considering the internal energy consumption mechanism (such as friction and damping) of air and structure, certain energy will be consumed during vibration of the system. The energy added to the structure system through the work of external force and the energy dissipated by the system are in a dynamic equilibrium status when the structure is deformation compatible, and there will be no constant energy accumulation in the system. If the structure is not properly designed and the deformation is not compatible, the energy added to the structure system through the work of external force will be more than the energy dissipated by the system in general, and energy will constantly build up in the system, which may result in divergent system vibration. Eventually, energy will accumulate at week points of the system, which may finally result in localized failure, or even progressive failure and overall dynamic instability of the system.

The general engineering structural mechanics analysis implies the suitability of load and deformation; in fact, the engineering structure should firstly be in a stable equilibrium status, and the structure measure that guarantee the structural deformation compatibility is also required. This is often ignored during actual work.

To facilitate understanding of the internal relation between engineering structural stable equilibrium and stable equilibrium with deformation compatibility control,

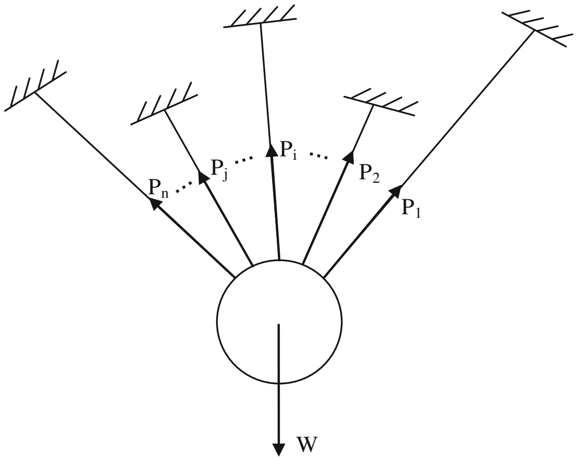


Fig. 1.2 Diagram of relation between stable equilibrium and deformation compatibility control of an object suspended on multiple ropes

the two concepts related to the engineering structure stress analysis and their theoretical application conditions are summarized in Table 1.1.

Therefore, the engineering structure may only exist when it is in the stable equilibrium and deformation compatibility control status.

For underground engineering structures, it is of great importance to "basically maintain the original status of the strata (surrounding rock)" and "construction process control".

The core of modern construction techniques, represented by the NATM, is to "fully exert the self-bearing capacity of surrounding rock". The aim of this idea is to maintain the stability of surrounding rock from the aspect of mechanics. The key to the stability of the surrounding rock is that the interaction of the surrounding rock and support system can reach the "stable equilibrium and deformation compatibility control". In practical engineering, the stress in surrounding rock will be redistributed after underground excavation. In particular, the stress of surrounding rock

Table 1.1 Relation between deformation compatibility control and structural equilibrium status

Content status	Engineering structural stress analysis	Application conditions
(i) Stable equilibrium	The precise analysis is adopted to solve engineering structure issues	Implies or naturally meets the deformation compatibility control requirements
(ii) Stable equilibrium and deformation compatibility control	First follow the overall-control and detail-mastering principle; then precise analysis is adopted to solve engineering structure issues	Establish reasonable structure system and reasonable construction method or process, as well as effective process controlling measures, so as to guarantee effective transfer or transfer path of forces

will transfer if plastic deformation zones appears. Therefore, it is very difficult to locate the zones of stress concentration or catastrophe points of instability in surrounding rock. It is also difficult to judge the stability of surrounding rock using stress-based measures. In order to assess the stability of the surrounding rock, the key is to control abnormal deformation. For class-I, class-II and superior class-III rock (Chinese rock classification system), control measures should focus on structural failures and stable equilibrium of blocks; For inferior class-III, class-IV, class-V and class-VI rock, control measures should focus on deformation compatibility, to prevent hazardous deformation that may cause collapse and loss of stable equilibrium.

The target of the concept of "basically maintaining the original status of strata (surrounding rock)" is the "stable equilibrium and deformation compatibility control". From the aspect of overall stability, it aims at maintaining the interaction of original surrounding rock and support system and achieving stable equilibrium and deformation abnormality control. It is also the necessary and sufficient condition for "reasonably exert the self-bearing capacity of strata (surrounding rock)". Therefore, in engineering practice, the concepts of "reasonably exerting self-bearing capacity of strata (surrounding rock)" and "basically maintaining the original status of strata (surrounding rock)" are actually the same. However, the concept of "basically maintaining the original status of strata (surrounding rock)" can facilitate practical application and control of surrounding rock stability.

Different construction procedures of underground works may often result in different force-deformation status of the rock/soil mass. However, in underground engineering design, only the status of the force in the completed engineering structure is taken into consideration, while the force-deformation status during construction tends to be ignored. For the surrounding rock with satisfactory self-bearing capacity, stable equilibrium may be achieved through re-adjustment after stress redistribution. The influence of excavation method on surrounding rock and supporting structure can be ignored. However, for rock/soil mass with poor self-bearing capacity, the construction procedures should be reasonably controlled during construction, and *process control* should be emphasized to get hold of the force-deformation status of rock/soil mass and supporting structure during construction; because only then can the force-deformation meet the design requirements, and then achieve the "basically maintaining original status of strata (surrounding rock)" for underground engineering rock/soil mass and "stable equilibrium and deformation compatibility control" for the structure. Therefore, the study on force-deformation status of rock/soil mass shows that the research and practice of underground engineering *process control* and *time-space effect* are equivalent. The *process control* should be implemented with emphasis on *time-space effect* of underground engineering construction. The "*timely supporting, timely sealing, timely measurement and timely feedback*" should be guaranteed.

1.3 Traditional Load Theory and Understanding

1.3.1 Loose Load Theory

In 1920s, Haim, Rankine and ИНик proposed traditional "loose load theory". The core content is that: the stable rock mass has self-stabilizing capacity, and does not generate load; the instable rock mass may cause collapse, and should be supported using supporting structures. Therefore, the vertical load acting on the supporting structure is the weight γH of the relaxation of the overlying rock/soil layers within a certain range. The theories of the three are different as follows: Haim, based on the theory of granular materials, believed that the lateral pressure coefficient was one; Rankine believed that the coefficient was $\tan^2(45°- \Phi/2)$; while ИНик, based on the theory of elastic mechanics, believed that the coefficient was $\mu/(1 - \mu)$. Though this kind of theory is suitable for shallow-buried tunnels, there will be more and more unreasonable points of the theory for tunnels in deeper depth. For tunnels with great depth, the pressure calculated will be too high. According to the observed collapse of surrounding rock and results of laboratory model tests, M. Лромобъяконоб and K. Terzaghi [5–7] improved the theory.

The loose load theory has once played important role. As an approximate calculation method of surrounding rock pressure, it can be applied easily. Because the method still provided reasonable pressure estimation for fractured rock mass and shallow-buried tunnels, it is still being widely used in some countries. The code recommended values from statistical analysis of underground engineering surrounding rock collapse based on the loose load theory are basically feasible; however, if no deformation compatibility control measure is taken to correct its excavation and supporting construction method, there may be engineering risks and excess supporting for bad and good surrounding rock respectively.

1. **Finite Failure Area of Surrounding Rock—M. Лромобъяконоб Theory**

The M. Лромобъяконоб theory is an early-stage rock pressure calculating theory. Though the calculating results may, in many cases, deviate remarkably from the actual conditions, the concept of finite failure area of surrounding rock is still of great significance.

The M. Лромобъяконоб theory is also called the natural equilibrium dome theory. The basic point of the theory is that: (i) as there are many joints, fissures and weak interlayers in the rock strata, the integrity of rock mass is harmed; the rock blocks formed by the discontinuities are usually of very small geometric size; the rock strata may be regarded as granular materials such as sands. However, as there is still cohesive force between rock blocks, the rock blocks should be regarded as loose blocks with certain cohesive force; (ii) after excavation of a cavern, a pressure arch will be formed at the top; the in situ stress acting on the lining is only the weight of the failed rock mass between the pressure arch and the lining, and is not related to the rock strata out of the arch or the buried depth of the cavern. The load calculation diagram for rectangular cavern based on M. Лромобъяконоб theory is

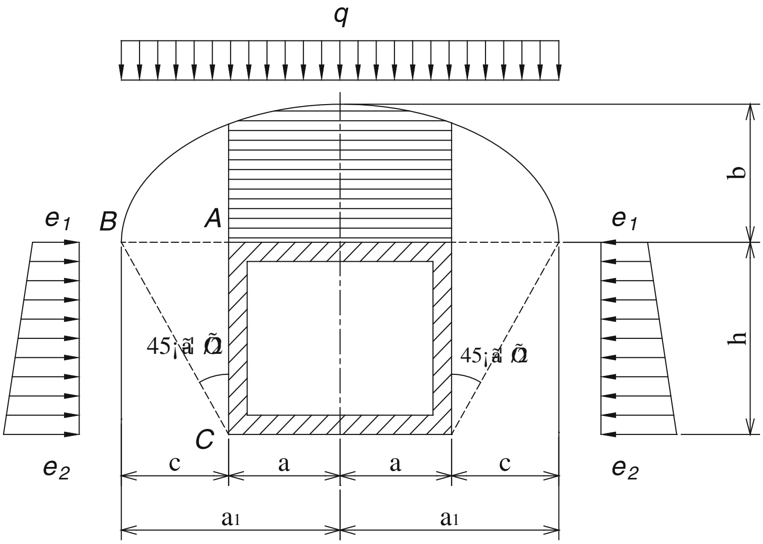


Fig. 1.3 Theoretical calculation chart of pressure arch

shown in Fig. 1.3. In the figure, φ is the internal friction angle of the rock mass; q is the vertical uniformly-distributed load on the pressure arch; e_i is the horizontal load; b is the maximum height of the natural equilibrium arch; a_1 is the maximum span of the natural equilibrium arch.

2. **Stable Equilibrium Theory of Surrounding Rock Failure—Terzaghi Theory**

In Terzaghi theory, the strata are regarded as granular materials. However, the vertical pressure acting on the lining is deduced based on the stress transfer concept. The core idea is the equilibrium of force.

Assume that a rectangular cavern with a span of 2b is excavated in the rock mass at the depth of H, and the side walls are stable after excavation, but the dome is not stable. Possible slippage along faces AB and CD is shown in Fig. 1.4. The pressure of surrounding rock on the dome equals to the weight of the overlying rock pillar minus the shearing resistance of side walls.

If the side walls and the dome are both instable after excavation, then the possible slippage along faces AB and CD is shown in Fig. 1.5, and the surrounding rock pressure can be calculated in accord with this chart.

The assumptions of deformation failure patterns of surrounding rock in the Terzaghi theory may be over simplified or not reasonable enough. However, the analysis concept of force equilibrium is of great significance in tunneling practice.

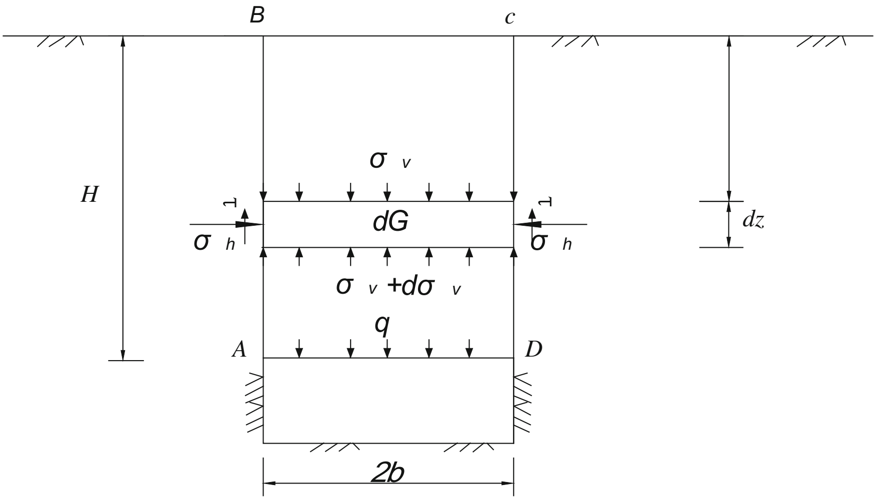
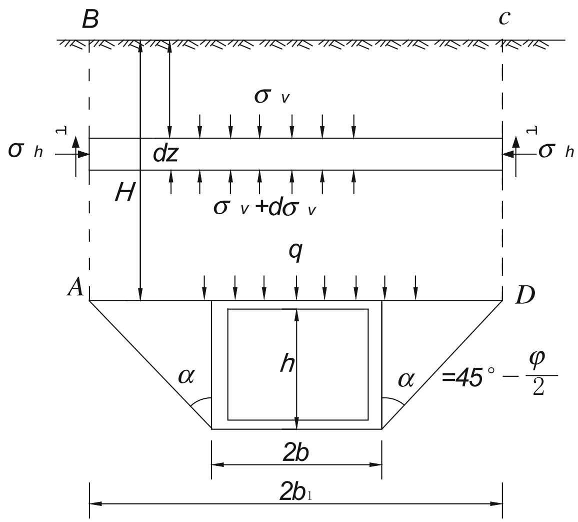


Fig. 1.4 Calculating chart of surrounding rock pressure with stable side walls

Fig. 1.5 Calculation chart of surrounding rock pressure with instable side walls



3. **Representative Construction Method**

The conventional construction method of a tunnel is called the mining method. It is thus named because it is firstly used for underground mining excavations. In the mining method, excavation is realized through drilling and blasting in many cases; therefore it is also called the drilling and blasting method. According to the development tendency of tunneling, the blasting and drilling method will be still the most common excavation method used in China.

The mining method can be further divided into the supporting construction method that uses the steel and wood members and the method that uses the drilling and blasting excavation and anchor-shotcrete. The former one is called the "conventional tunneling method" (CTM) and the latter is called "new Austrian tunneling method" (NATM). The theoretical foundation of the CTM is the "loose load theory" and the theoretical foundation of NATM is the "rock bearing theory".

The design methods based on loose load theory such as traditional Terzaghi theory or the M. Промобъяконоб theory is the mechanical method established based on the statistics analysis of shallow-buried soft layers and deep-buried rock/soil mass. The force deformation status in simple circumstances meets the "deformation compatibility control requirements"; therefore, there is no concept of deformation compatibility control and process control, and it does not reflect the basic concept of "reasonably exerting the self-bearing capacity of strata (surrounding rock) and "basically maintaining the original status of strata (surrounding rock)". For the comparatively fractured rock mass or soft soil strata, the "reasonably exerting self-bearing capacity of strata (surrounding rock)" can only be achieved when the condition of "basically maintaining original status of strata (surrounding rock)" is met. Hence, for shallow-buried tunnels in loose layer or shield tunnels in soft soil, the simplified calculation methods, i.e., the load-structure method (corresponding to the loose load theory), can be used. The deviation generated is within the allowable range of the supporting structure strength. However, the stratum-structure concept (corresponding to the rock bearing theory) is used as the construction method and process control measures, to ensure that the positive interaction of strata and supporting structure reaches the "stable equilibrium and deformation compatibility control" status, and thus eliminate risks and potential hazards, and guarantee the stressing safety. All in all, the theories are worth understanding.

1.3.2 Rock Bearing Theory

1. **Basic Principles**

The modern supporting theory, or "rock bearing theory", was proposed in 1950s; the core content of the theory is that: the stable surrounding rock has self-bearing capacity; it takes time for the instable surrounding rock to loose its stability. If necessary help or constraint is provided for the surrounding rock during this process, the surrounding rock may become stable again. The representative researchers on this theory include K.V. Rabcewicz, Miller-Fecher, Fenner-Talobre and H. Kastener. This theory is comparatively modernized, and it is out of the design concept for the structures on the ground, and more closely to the actual situation of underground engineering. For this reason, it has been widely used in the past few decades.

In "loose load theory", the results of the final status and the treatment of the results are emphasized. However, in modern "rock bearing theory", the construction process and process control, i.e., fully exert the self-bearing capacity of the surrounding rock, are emphasized. Thus it can be seen that the two theories are different in terms of principles and methods. The construction methods using the two theories as guidance are totally different, and therefore pose different impacts on the stability of surrounding rock of excavations.

Compared with the traditional steel and wood component supporting, the anchor-shotcrete supporting is different in terms of means, and more predominant difference is the engineering concept, which represents people's further understanding of tunneling and underground engineering. With application and development of the anchor-shotcrete supporting technique, the theory of tunneling and underground engineering has entered a new era of modern theory. Moreover, the tunnel and underground engineering design and construction can better meet the actual needs of underground engineering, and realize the consistency among the design theory, construction method and structural (system) working status (results). The relation between the supporting and displacement of surrounding rock based on the "rock bearing theory" is shown in Fig. 1.6. Within a certain range, a smaller supporting force is required if allowable deformation of surrounding rock is greater; and vise versa. To prevent failure of the surrounding rock, the development of deformation must be constrained.

The core concept of the NATM is widely used in tunneling and underground engineering practice. However, there is also one important issue with supporting design based on the characteristics curve of surrounding rock displacement and supporting: the D point theoretically exists on the curve, but it is actually impossible to locate the point in practice. Though the stress equilibrium problem of surrounding rock and structures in tunneling can be solved using the characteristics curve, the stable equilibrium problem is still difficult to deal with. The NATM

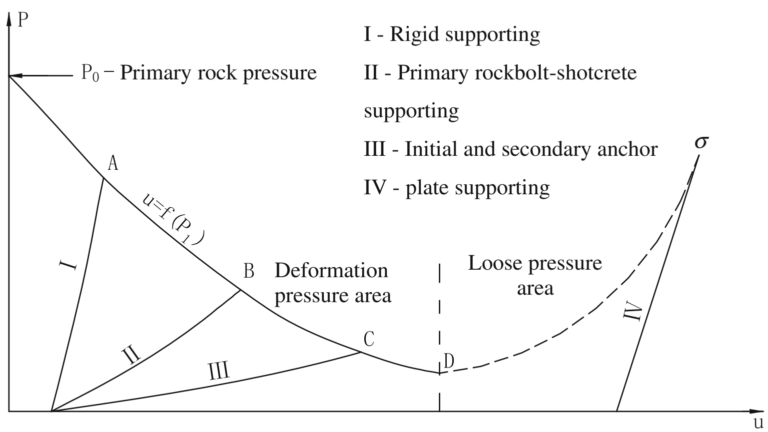


Fig. 1.6 Surrounding rock displacement supporting characteristics curve

based on rock bearing theory originate from hard rock. Though the difference between the soft rock and hard rock is addressed, the stable equilibrium status of interaction between the surrounding rock and supporting structures can hardly be guaranteed, especially for the bifurcation instability problem with poor geological conditions. The lining fracture and construction safety issues of underground engineering demonstrate this kind of risks.

2. **Representative Construction Method—NATM**

The NATM was created by Rabcewicz and Müller, two Austrian civil engineers, in the 1960s based on summary of the practical experience in tunnel construction [8]. After the method was proposed, its theoretical foundation has been continuously improved, and the tunnel supporting technical measures have been constantly enriched. It is widely used in tunnel design and construction all over the world. Now it has become one of the most important methods used in China and other countries for tunnel structural design and construction. The theoretical foundation of NATM is “reasonably exerting the self-bearing capacity of surrounding rock”. The features of NATM, e.g. measurement technique, shotcrete and anchor reinforcement aim at maintaining the original strength of surrounding rock while allowing deformation of the surrounding rock, without any serious looseness and failure. The tunnel excavation and supporting principle that emphasizes deformation trend of the surrounding rock and supporting structure is adopted to ensure the dynamic equilibrium between the external load that deforms the surrounding rock and the structural supporting resistance that restrains the deformation. This construction method is widely feasible and economically efficient.

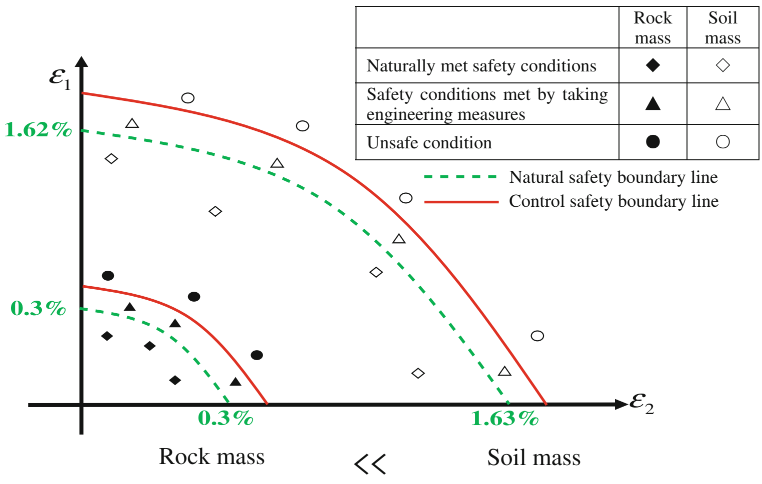
Rock bearing theories such as the NATM, the New Italian Tunneling Method, the Norwegian Tunneling Method and convergence-confinement method were proposed based on the experimental rock mechanics. The structural mechanics analysis method is quite ambiguous, and is often ignored, which results in unreasonable design of supporting structures. To reflect the concept of “reasonably exerting self-bearing capacity of strata (surrounding rock)” and “basically maintaining the original status of strata (surrounding rock)”, abundant theoretical knowledge of mechanics and practical experience of similar projects are needed, so as to master the essence of the rock bearing theories and ensure the positive interaction between the surrounding rock and supporting structure in the “stable equilibrium and deformation compatibility control” status. Hence, it is the basic requirements for underground engineering structural design and safety analysis to establish the generalized stable equilibrium equation that is applicable to various rock/soil characteristics based on the concept of “reasonably exerting self-bearing capacity of strata (surrounding rock)” and “basically maintaining the original status of strata (surrounding rock)”, or “deformation compatibility control”. Furthermore, emphasis should be laid on study on reasonable excavation method, supporting measures and construction process control, to ensure the interaction of the surrounding rock and supporting structure in the “stable equilibrium and deformation compatibility control” status. Under poor geological conditions, stable equilibrium and deformation compatibility of the resultant force of strata (surrounding rock) *and supporting structure may not be met during underground engineering construction. In these cases, it is also difficult for the construction organization to control the deformation and stability, and accidents often take place. The requirements should be taken into consideration in the planning and design stages to avoid similar accidents.*

1.4 Stable Equilibrium Theory for Underground Engineering

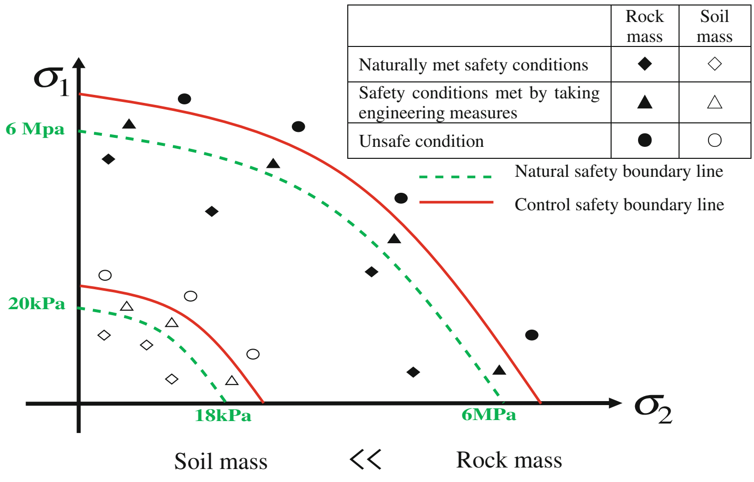
1.4.1 Establishment of Stable Equilibrium Theory for Underground Engineering

There are advantages, deficiencies or even potential safety hazards for application of both loose load theory and rock bearing theory in practical underground engineering construction. The authors have carried out a great number of underground engineering experiments for rock, soil and rock-soil mixture. The results indicate that the curved section $P_0 - D$ in the displacement—supporting characteristics curve as shown in Fig. 1.6 is in accordance with the control safety boundary of underground structures in rock/soil mass as shown in Fig. 1.7. The section $D - \sigma$ has no practical significance in the design and construction of underground engineering. It is of great practical significance to establish the stable equilibrium theory for underground engineering by effectively utilizing the high rheological stress limit and inheriting the essence of loose load theory and rock-bearing theory within the control safety boundary.

In Fig. 1.7, the muddy soil and sandstone are taken as examples to compare the engineering mechanics property of stress control method and strain control method in engineering application for soil mass and rock mass. The dotted line is the safety control boundary of original status of rock or soil mass, which is also called the natural safety boundary line. The stress or strain status inside the dotted line is the safety status (diamond symbol). If the dotted line is exceeded, the failure of rock or soil mass will be expected. The status may be improved to a certain extent through engineering measures, such as grouting in the soft soil and addition of anchor bolt and cable in the rock mass. If the engineering structure meets the deformation compatibility control requirements, the allowable safe range will be expanded to the solid line range. In such cases, some originally unsafe status will turn into safe status (triangle symbols). For Fig. 1.8, engineering measures may be taken to effectively utilize the high rheological stress limit; otherwise, excessive localized stress or strain may be induced, which may result in structural instability. Only by this way can the reasonable transfer or transmission of stress, stable equilibrium and deformation compatibility control of underground engineering structures be guaranteed.



(a) strain boundary



(b) stress boundary

Fig. 1.7 Safety boundary line for underground structures control in rock/soil mass (**a**) strain boundary (**b**) stress boundary

It is also shown in Fig. 1.7 that the strain decreases with the increase of the stress in rock strata. Therefore, structural deformation compatibility control is of greater significance for underground engineering control in poor rock/soil strata.

The difference between the original rheological stress and strength of rock/soil mass should be pointed out. The original rheological stress of soil sample such as the muddy soil in Fig. 1.8 is about 10–25 kPa, and the strength is about 50–100 kPa; the

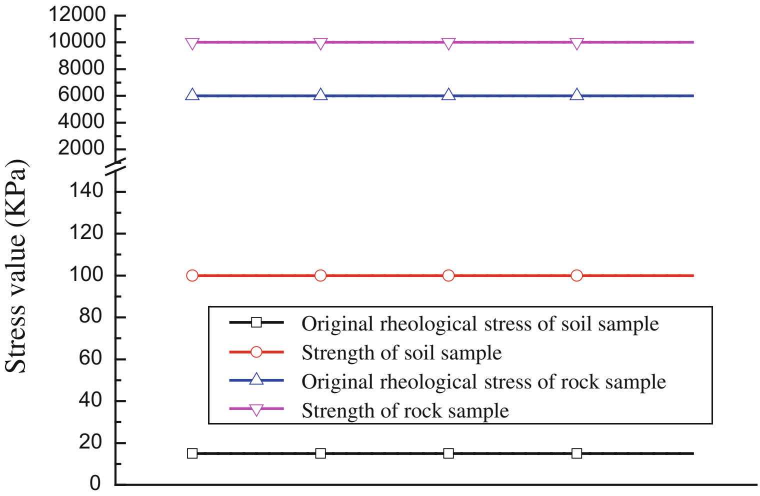


Fig. 1.8 Comparison between original rheological stress and strength value of rock/soil mass

original rheological stress is about 20 % of the strength. Cracks appear when the original rheological stress of rock sample (sandstone) reaches about 6–9 MPa; the original rheological stress value is about 60 % of the strength, and the ratio is much higher than that for soil sample. However, in practical engineering construction, if the stress level exceeds the original rheological stress, the rock/soil mass will non-recoverably deform despite that the stress is within the strength, which will pose a direct impact on the equilibrium status of the underground structural force. Hence, engineering measures should be taken if the stress level exceeds the original rheological stress. It is not wise to wait until complete failure of the rock/soil mass occurs, which may influence the normal working status of the underground structures.

As shown in Figs. 1.9 and 1.10, the original status of strata usually meets the stable equilibrium and deformation compatibility control requirements prior to underground engineering construction irrespective of the hydrological and geological conditions of the strata. Furthermore, the stress-strain status of any point in the strata meets the control safety boundary area requirements. During construction, the physical status of the strata should basically remain in the original status. The natural safety boundary area may be exceeded to a certain extent to reach the control safety boundary area (Fig. 1.7), if the strata deformation conditions are effectively controlled to utilize the high rheological stress limit (Fig. 1.9) by increasing the safety range reasonably. There are two possible usage of high stress limit: (1) the strata (surrounding rock) stress basically remains unchanged, but it is transferred into the deeper portion; (2) stress attenuation and increase of deformation may be evident at the discontinuities in the strata (surrounding rock); the stress may also be transferred into the deeper portion when conditions are met to control further collapse automatically (Fig. 1.10). The difference between rheological stress and strength should be identified as shown in Fig. 1.8 in structural

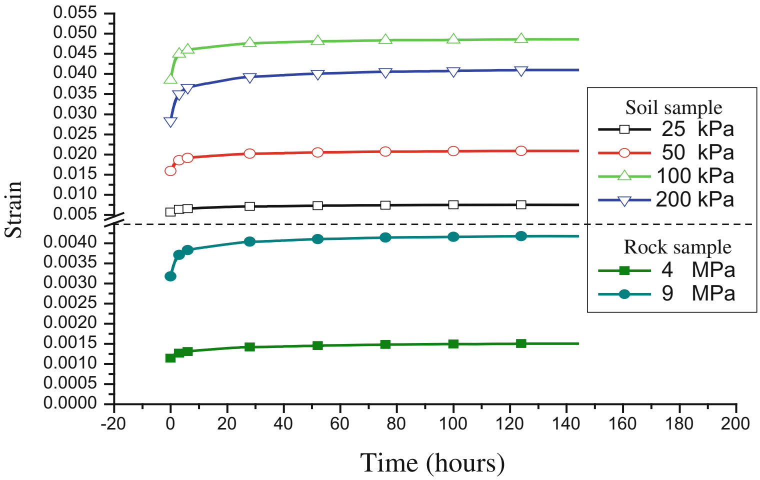


Fig. 1.9 Rheological mechanics characteristics of rock/soil mass

calculation. Then, any point in the strata can meet the stable equilibrium and deformation compatibility control requirements. Otherwise, if the deformation of strata is not effectively controlled during construction, the high rheological stress limit or strength (Fig. 1.9) may not be used in structural calculation. If the recommended rock/soil strength in the existing codes or manuals are used for structural calculation (≥original rheological stress), the designed safety coefficient range in Fig. 1.8 may be easily exceeded. In such a case, irregular structural mechanical behaviors, or even accidents, may occur during construction.

For rock strata or strata with other geological structures, similar mechanical behaviors of structures can be observed during underground engineering construction.

As shown in Figs. 1.7 through 1.10, the characteristics curves of the NATM or convergence-confinement method from mechanics experiments of integral rock mass cannot satisfactorily reflect the mechanical behaviors of underground structures in poor strata, such as soft soil, fractured rock mass or soil-rock mixture. Only when the strata deformation is effectively controlled (Fig. 1.7) can the high rheological stress limit (Figs. 1.9 and 1.10) be effectively utilized, and the characteristics curves of NATM or convergence-confinement method be met. Hence, it is more reasonable to use the "basically maintaining the original status of strata (surrounding rock) to effectively control strata deformation" instead of "reasonably exerting self-bearing capacity of strata (surrounding rock)" to identify the mechanical behaviors of underground structures.

The underground engineering stable equilibrium status abstractly reflects the mechanics relation among the self-bearing capacity P of surrounding rock, supporting resistance T and original internal force P_0 of the surrounding rock, and truly reflects the mutual interaction process and effect between surrounding rock and

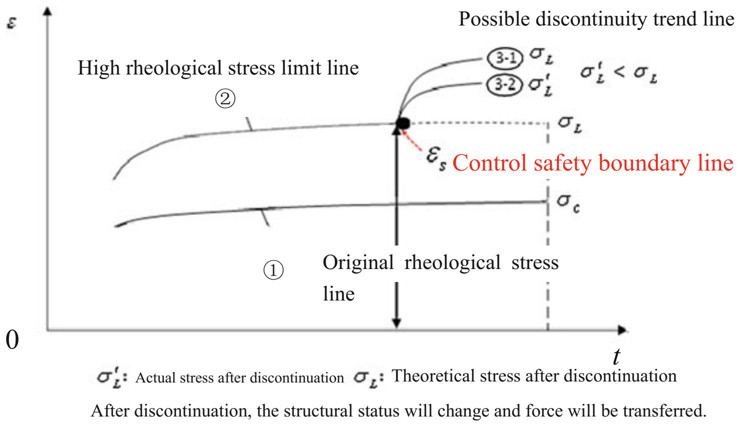
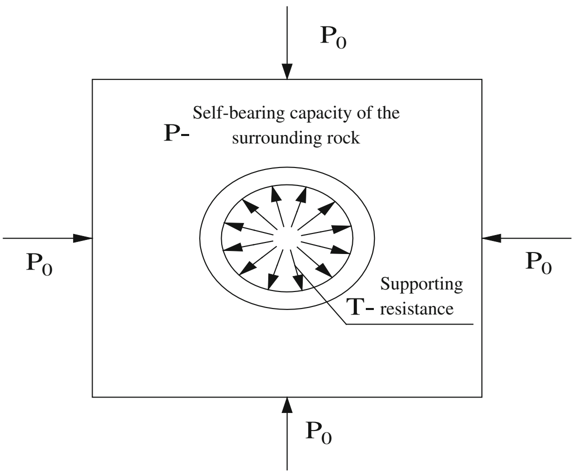


Fig. 1.10 Characteristic diagram of control of discontinuity and conditional utilization of high rheological stress limit of rock/soil mass

lining of the structure system during excavation and supporting (including pre-reinforcement). Establishment of the engineering stable equilibrium theory for underground engineering construction may help engineers with reasonable determination of design method and construction scheme.

Tunnel excavation will introduce a new free face and radial stress release on the surrounding rock, while the stress status of the strata away from the unlined segment remains the same. Generally, considering the homogeneous stress field of ground, P_0 is used to indicate the initial ground stress, as shown in Fig. 1.11. According to the principle of statics, P_0 is balanced by the bearing capacity of the "surrounding rock-supporting" structure system.

Fig. 1.11 Diagram of combined action of surrounding rock and supporting



The surrounding rock pre-reinforcement force F equals to the bearing capacity of the “surrounding rock-supporting” structure system, i.e.

$$F = T + P \tag{1.5}$$

where F is the pre-reinforcement force; T is the supporting resistance; P is the self-bearing capacity of surrounding rock.

Hence, the tunnel pre-reinforcement force is not only the acting force of the supporting structure on the surrounding rock. It is the resultant force of the supporting resistance provided directly by the supporting structures and the self-bearing capacity of the surrounding rock. The self-bearing capacity of the surrounding rock can be maintained through pre-reinforcement measures and reasonable excavation sequences.

If the pre-reinforcement force is greater than the force that will excessively deform or damage the surrounding rock, the surrounding rock is in a stable equilibrium status. This is called the pre-reinforcement technique of tunnels. According to the general principle of surrounding rock stability, the ground stress is the basic power that induces deformation and failure of the surrounding rock, and is the “force source” of surrounding rock instability. Hence, the tunnel pre-reinforcement technique can be further described as follows: the pre-reinforcement force F should be always greater than the original internal force P_0 that maintains the original rock mass stable equilibrium before tunnel construction, and the surrounding rock is in a stable equilibrium status. i.e.:

$$F > P_0 \tag{1.6}$$

Equation 1.6 is generally applicable to underground engineering stable equilibrium issues. Although the expression form of the theory may change with the “specific situation”, Eq. 1.6 always remains the same.

Before excavation, the tunnel is in a three dimensional stress status, and the self-bearing capacity of the surrounding rock is greater than the original internal force. Therefore, the surrounding rock is in a stable equilibrium status. After tunnel excavation, due to the free face, the stress status of the surrounding rock is adjusted and the radial stress decreased. Furthermore, the surrounding rock will move towards the tunnel section when driven by the gravity, water stress, expansive stress of rock, tectonic stress and engineering deviator stress. In the meantime, the internal structure of the surrounding rock starts to deteriorate, which results in a lower self-bearing capacity of the surrounding rock as shown in Fig. 1.12. The excavation is the process of unloading of the surrounding rock and redistributing of surrounding rock stress. The self-bearing capacity of the surrounding rock is determined by the secondary stress status, and is the reaction force that induces movement of surrounding rock and destructive load. The self-bearing capacity of surrounding rock is a function of space and time, and has the *time-space effect*.

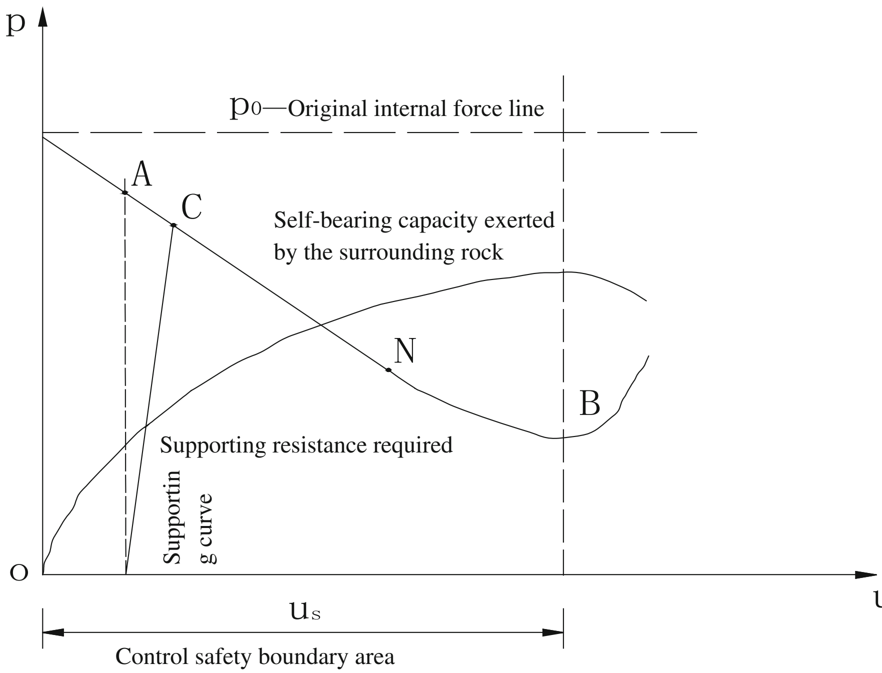


Fig. 1.12 Force-displacement characteristic curve graph of underground engineering equilibrium stability theory

To sum up, (1) intact surrounding rock with good self-bearing capacity is able to provide the bearing capacity required to maintain the stability of surrounding rock as shown in Fig. 1.13a. Such surrounding rock can self stabilize without any supporting measures. The system rockbolts can be omitted, and Eq. 1.6 can be met naturally. (2) For surrounding rock with certain self-bearing capacity, the curve of pre-reinforcement principle is shown in Fig. 1.13b. The NATM can be used to satisfy the Eq. 1.6; or P_0 is initially calculated using finite elements methods, and P can be converted based on the geological conditions, and then the value of T can be initially estimated. In this case, Eq. 1.6 can also be satisfied. (3) For fractured surrounding rock or weak surrounding rock with low self-bearing capacity as shown in Fig. 1.13c, the self-bearing capacity of the fractured surrounding rock is quite low and decreases dramatically after excavation of the tunnel, and the deformation induced pressure will quickly turn to loose pressure. The surrounding rock will soon enter the loose status, and shift very from the instable equilibrium status to the instability status. Therefore, pre-reinforcement or advance supporting should be provided before excavation, so as to improve the original status of surrounding rock and enhance the self-bearing capacity. In this case, the self-bearing capacity of surrounding rock P is about 0, and the system rockbolt doesn’t work, so it is required that $T > P_0$. (4) Under other circumstances, the

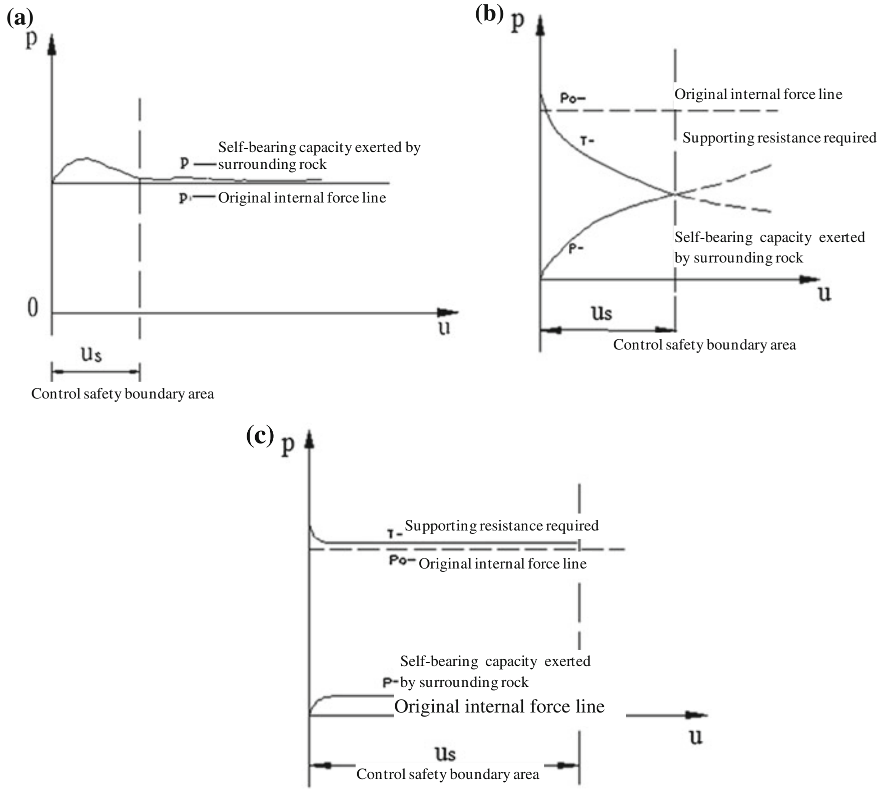


Fig. 1.13 Three examples of force-displacement characteristic curve of underground engineering stable equilibrium theory (**a**) intact surrounding rock; (**b**) surrounding rock with fair self-bearing capacity; (**c**) surrounding rock with poor self-bearing capacity

underground engineering stable equilibrium theory can be implemented by referring to cases of (1), (2) and (3).

Hence, the underground engineering stable equilibrium theory reflects the basic content of traditional "loose load theory" and modern "rock bearing theory", and at the same time expands the existing contents such as stable equilibrium, which provides the new understanding and concept of underground engineering stable equilibrium. The outcomes are based on design codes but provide wider vision than codes. Based on the existing underground engineering construction theory, applying stable equilibrium theory to underground engineering construction help us to establish a more comprehensive theory of underground engineering stable equilibrium. At the same time, it can be used to better interpret the reasonability of many construction methods and ideas, such as removal of system rockbolt, reasonable excavation and supporting techniques, in order to guide the underground engineering design and construction in a better way.

1.4.2 *Extension and Embodiment of Underground Engineering Stable Equilibrium Theory*

The stable equilibrium theory is suitable for general tunneling issues, and can be used to interpret the uniformity and applicability issues of various design theories and their construction techniques. For tunnel engineering construction in special environments, certain extension is required based on the existing M. Лромобъяконоб theory and K. Terzaghi theory and other applicable mechanics theories.

1. **Transfer the Stress of Surrounding Rock into Deeper Portion**

Gradually transfer the stress of surrounding rock to deeper portion through proper excavation and supporting, in order to reduce stress concentration on the surface of surrounding rock, which may cause localized deformation or failure. As shown in Fig. 1.14, the natural cave 200 m high and 150 m wide. The stability status of the cave demonstrate the utilization of plastic deformation of surrounding rock to transfer the stress concentration area to deeper portion.

2. **Transfer Process of Surrounding Rock Equilibrium Status**

The stability of the tunnel and surrounding rock is the result of positive interaction between the tunnel surrounding rock and support system. If the excavation and supporting process is not reasonable, the equilibrium status will change. When the pre-reinforcement is adopted, and the surrounding rock remains in the original status:

$$P_1 \cos \alpha_1 + P_2 \cos \alpha_2 + T = W \tag{1.7}$$

where P_1 and P_2 are the mutual supporting force between surrounding rocks; W is the gravity; T is the supporting resistance (T should be as small as possible).

The self-stabilizing time in special geological conditions such as fractured surrounding rock is very short after tunnel excavation, and roof will easily fall. Through

Fig. 1.14 Natural Cave (height: 200 m; width: 150 m)

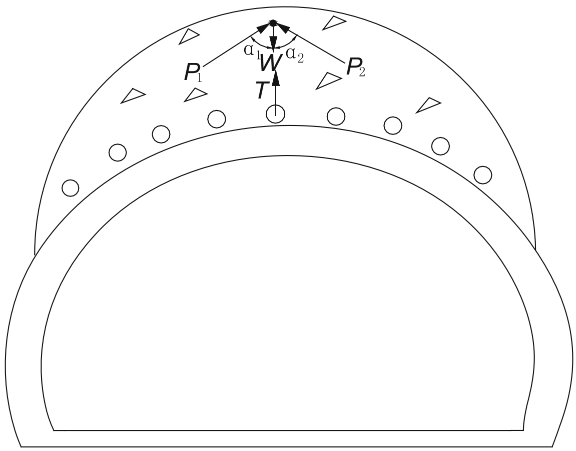


Fig. 1.15 Pre-reinforcement technique for surrounding rock of special geological conditions

pre-reinforcement, a supporting system including pre-reinforcement structure, primary supporting and secondary lining will bear the load together. The pre-reinforcement based on the shallow tunneling method (STM) or shield tunneling principle can be used to prevent loose collapse and collapse-induced loose pressure. However, its mechanism is different from that of the anchor-shotcrete supporting. The design can be conducted according to M. Лромобъяконоб theory and K. Terzaghi theory. As the self-stabilizing capacity of fractured surrounding rock and surrounding rock of other special geological conditions is weak, and always influenced by underground water, the contribution of internal friction angle Φ and internal cohesive force c ($c = \Phi = 0$) is not taken into consideration for sake of safety. Only pre-reinforcement induced mutual supporting forces P_1 and P_2 in the surrounding rocks without harmful looseness (Fig. 1.15) is taken into consideration [9].

If pre-reinforcement is not adopted for the tunnel surrounding rock and serious looseness or collapse happens, then the relation $T \leq W$ holds. In another word, pre-reinforcement measures must be taken for fractured surrounding rock or surrounding rock of special geological conditions, in oder to ensure that the pressure on supporting structures is deformation pressure rather than loose pressure. At the same time, for pre-reinforcement, the rigidity of pre-reinforcement structures and the timing of shotcreting after excavation, i.e., the time-space effect, must be taken into consideration. These factors will influence the deformation of surrounding rock of special geological conditions, i.e., the distribution and magnitude of surrounding rock pressure. Therefore, proper selection of rigidity of pre-reinforcement structures and the timing of shotcreting is very important.

3. **Application of Basic Mechanics (theoretical mechanics, structural mechanics and energy incremental method)**

When basic mechanics (theoretical mechanics, structural mechanics and energy incremental method [10]) is used to solve complicated issues, the object must be understood in a systematic way, and the entire system must be highlighted. Grasp the relation between the entirety and locality, and consider various factors in an overall manner, and analyze the mechanics or energy incremental modes of interaction between surrounding rocks, environment and support systems in details ($F > P_0$ or $\Delta U > \Delta T$). The main status should be in accordance with the basic mechanics or the energy incremental relation. Or, if the "basically maintaining the original status of strata (surrounding rock)" and "stable equilibrium and deformation compatibility control" are achieved through a support system with reasonable rigidity to convert the complicated mechanics issues into simple ones, the basic mechanics can be used to solve underground engineering issues.

To conclude, the underground engineering construction concept and effective method are studied respectively but uniformed systematically, in order to extend the research and application scope. For instance, the "loose load theory" can be combined with the modern construction method; the "rock bearing theory" can be combined with both traditional construction method and many modern construction methods; the "loose load theory" and "rock bearing theory" can be extended or mutually integrated (to improve the shield tunneling method in soft soil) to better suit the complicated environment and structure; combination of the stable equilibrium theory with traditional construction and more modern construction methods will allow further extension of its application scope. Practical development relies on theoretical innovation. Similar to a PC mother board with with different hardward slotted and integrated, the underground engineering stably equilibrium theory is used to analyze and study the mechanical status of strata supporting system based on the overall mechanics. The basic content of traditional "loose load theory" and modern "rock bearing theory" is integrated into a system based on the stable equilibrium theory, in order to extend contents on various aspects, enrich the underground engineering stable equilibrium theory, and comprehensively reflect the basic insight of the underground engineering stable equilibrium (i.e. $F > P_0$ or $\Delta U > \Delta T$).

1.4.3 Application of Underground Engineering Stable Equilibrium Theory

The way to determine whether the underground structure system satisfies the "deformation compatibility control" conditions: stress is used for the existing failure criteria of engineering materials, including the classical Mohr-Coulomb criterion, twin shear stress yield criterion and unified strength criterion. However, for complicated underground works, the conditions seem to be satisfied theoretically according to the engineering mechanics analysis and stress criterion. However, in practical engineering, there will still be some problems or even potential safety hazards. Therefore, it is not enough to judge safety of a structure based on the traditional failure criteria. Deformation control conditions should also be applied. For simple structures, the system deformation is definite without any distortion; however, the system deformation may easily distort from the expected status for complicated structures when the construction is sometimes not reasonable. To sum up, for simple structures or homogeneous materials with no deformation compatibility issues, there will be no problem with the existing judging criteria, since the deformation compatibility conditions are implicitly contained. However, for complicated structures, the deformation compatibility control conditions are not necessarily satisfied. In such cases, there will easily be deformation distortion if the existing judging criteria are used. Moreover, the stress redistribution will be influenced and actual stress will deviate from the design status, which may result in accumulated damage to the structure or even trigger failure.

For stable equilibrium and deformation compatibility control issue of complicated engineering structures, though the essential element is to judge the actual stress, deformation, and stability status of the structure or analyze whether the structure exceeds the design stress deformation status, the specific and easily measurable variables (such as deformation and natural vibration frequency) are used to measure the deformation and stability status in practical engineering.

Therefore, for underground structures, whether the structure system meets the deformation compatibility control requirements can be judged based on whether the surrounding rock satisfies the “basically maintaining the original status of rock/soil mass” requirements or whether the status of surrounding rock is effectively controlled. If the deformation compatibility control conditions are met, then the actual stress-deformation status of the structure and the design status will be basically the same; otherwise, there will be difference between the actual stress-strain status of the structure and the design status. In such cases, unexpected problems or safety issues may occur.

In the design process of engineering structures, the construction characteristics of classical engineering structures should be taken as reference to initially design reasonable engineering structure system. This can generally meet the “deformation compatibility control” conditions; in complicated or special cases, the model or large scale test may be used for further verification.

“Force and energy require related material carriers, and related transmission or transfer paths.” The structural “stable equilibrium and deformation compatibility control” and “reasonable conversion of energy” are unified. The deformation compatibility control is the prerequisite of structural analysis using the energy method, force method or displacement method. Ensuring that “force, deformation and energy” are transferred along the designed paths and converted according to the designed mode is the basic requirement for reasonable, safe and stable structure, and is also one of the prerequisites to ensure that the design does not generate hazardous process during construction. The engineering structural behaviors can be better controlled by optimizing one or several factors among “force, deformation and energy" under specific situation. To ensure the "stable equilibrium and deformation compatibility control", not only the object control but also the process control for structural safety and reasonability is required. Otherwise, the structure will be instable or may fail. The following conclusion is drawn based on careful investigation of classical historical projects all over the world, and logic analysis and mechanics judgment: the interaction between surrounding rock and supporting structure should meet the stable equilibrium and deformation compatibility control conditions during the entire process of construction and operation, in order to ensure that the stressed composite structure systems can be converted from composite or single stressed body into integrated composite stressed body; otherwise, the original mechanics equilibrium form may be altered, or a new mechanics equilibrium form may appear unexpectedly, which may even result in instability. For the composite structure system with several independent stressed units for which the deformation compatibility control conditions cannot be met, the key is to prevent cracking of the connecting part. For example, for multiple-arch tunnels and small-clearance tunnels, as well as multiple parallel continuous-beam bridges, their stress independence should be improved. Therefore, the equilibrium and stability theory is extended from the stable equilibrium to stable equilibrium and deformation compatibility control. This includes "reasonably exerting self-bearing capacity of strata (surrounding rock)", $F > P_0$ or $\Delta U > \Delta T$, and the extensions: "basically maintaining original status of strata (surrounding rock)", excavation energy control technique, strong pre-reinforcement technique, comprehensive stress-independence technique and deformation compatibility control technique, in order to probe into the underground engineering construction safety issues in a more comprehensive way.

In order to achieve stable equilibrium status of underground structures, the modern construction techniques (such as NATM, shallow tunneling method, Norwegian method, new Italian tunneling method) and traditional construction techniques (such as mining method, M. Лромобъяконоб theory and K. Terzaghi theory), as well as other effective aiding methods for special environments are necessary. Furthermore, technical measures for tunnel engineering construction, such as excavation energy control technique, strong pre-reinforcement technique, comprehensive stress-independence technique and deformation compatibility control technique should also be adopted. Only when the practitioners master the guiding ideology and technical measures can the underground engineering stable equilibrium theory be satisfactorily applied.

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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

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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:

$$E = E_1 + E_2 + E_3 \tag{2.1}$$

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

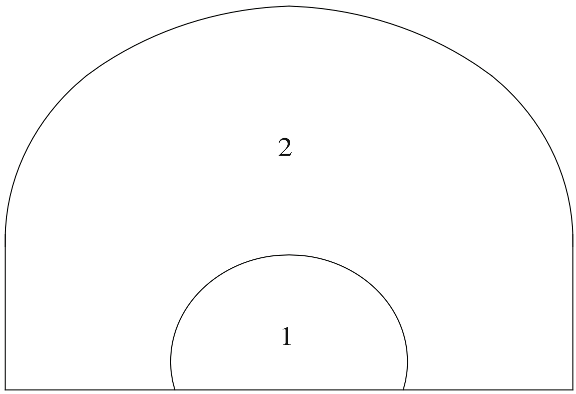


Fig. 2.1 Pilot tunnel advancing + expanding excavation construction method

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

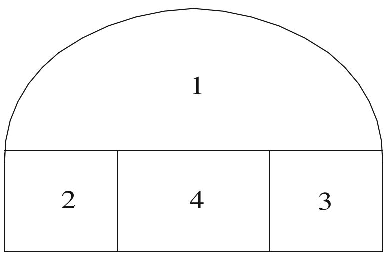


Fig. 2.2 Construction order of top heading and benching tunneling method. *1* Heading excavation and supporting; *2* left bench excavation and supporting; *3* right bench excavation and supporting; *4* middle bench excavation and supporting

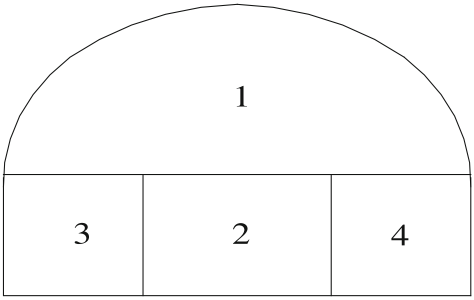


Fig. 2.3 Construction order with hard bench part. *1* Heading excavation and supporting; *2* middle bench excavation; *3* left bench excavation and supporting; *4* right bench excavation and supporting

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.

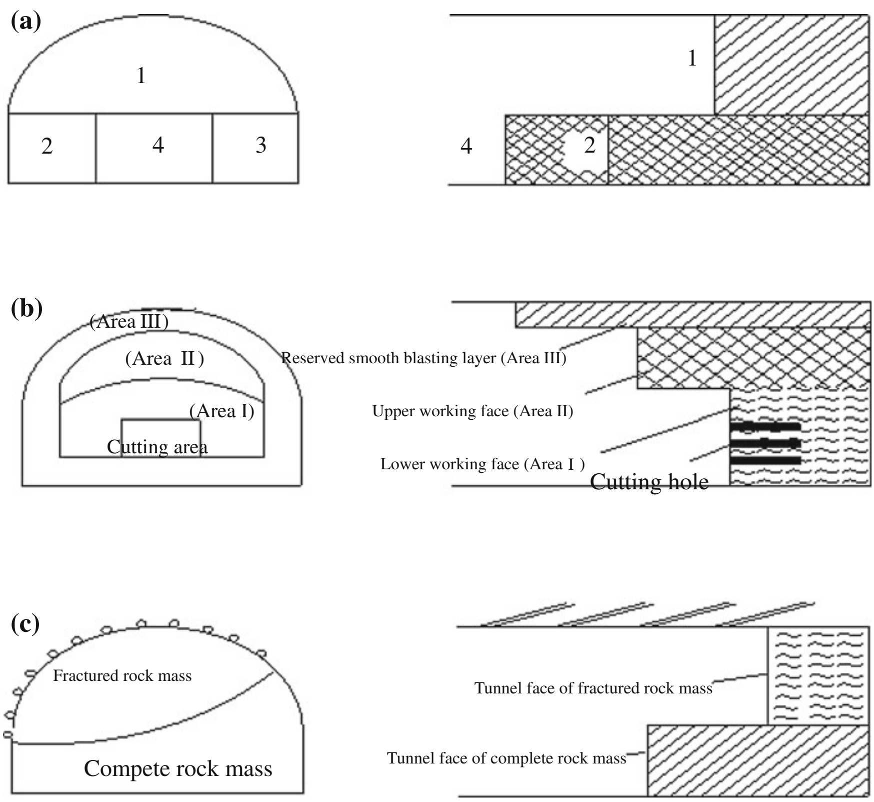


Fig. 2.4 Schematic diagram of blasting excavation method. **a** Surrounding rock with soft upper part and hard lower part; **b** relatively hard surrounding rock; **c** half-soil half-rock surrounding rock

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.

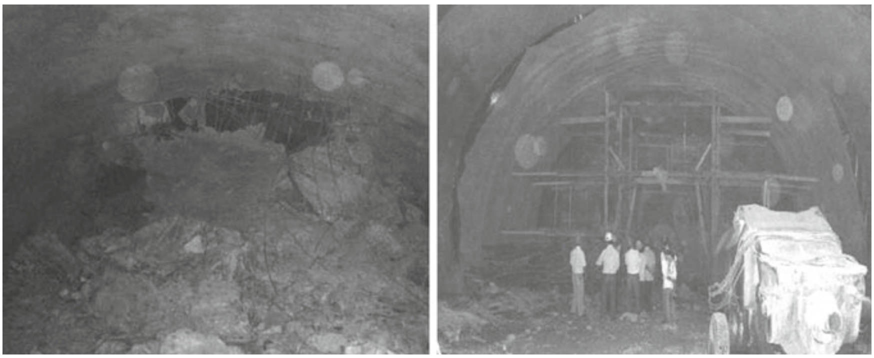
3. **Fractured or Weak Surrounding Rock with Insufficient Self-bearing Capacity**

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, M. 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

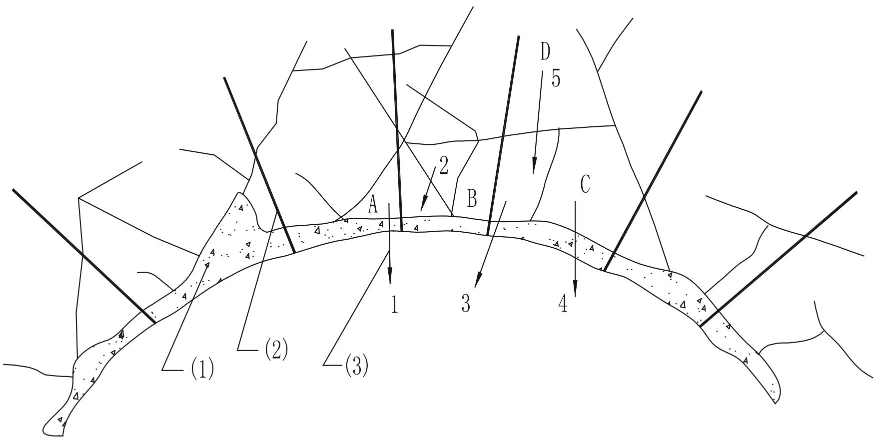


Fig. 2.10 Schematic diagram of anchor-shotcrete supporting technique for blocky surrounding rock. (*1*) Shotcrete; (*2*) rockbolt; (*3*) potential collapse direction of the block; the potential collapsing order of blocks is *1*, *2*, *3*, *4*, and *5*

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.

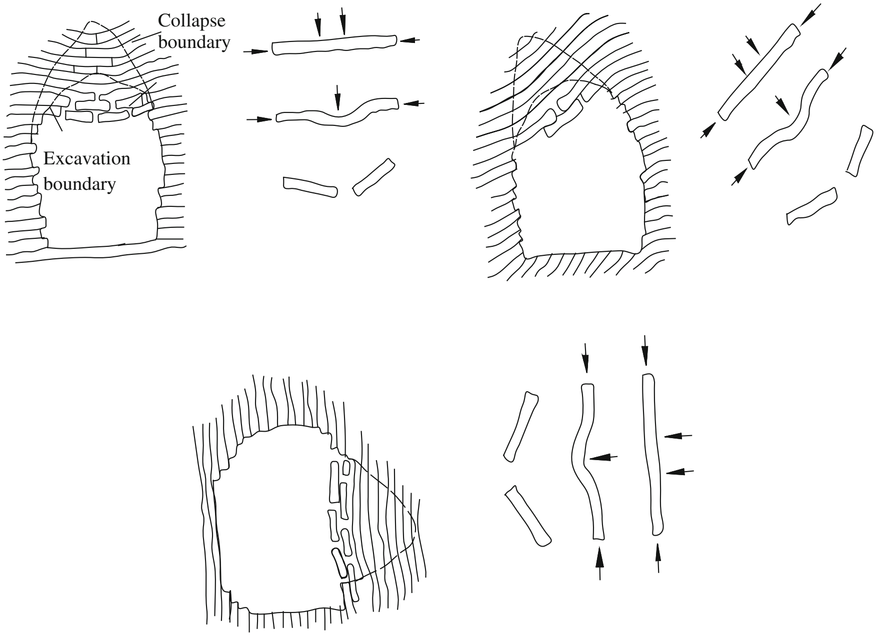


Fig. 2.11 Instability forms of stratified rock mass

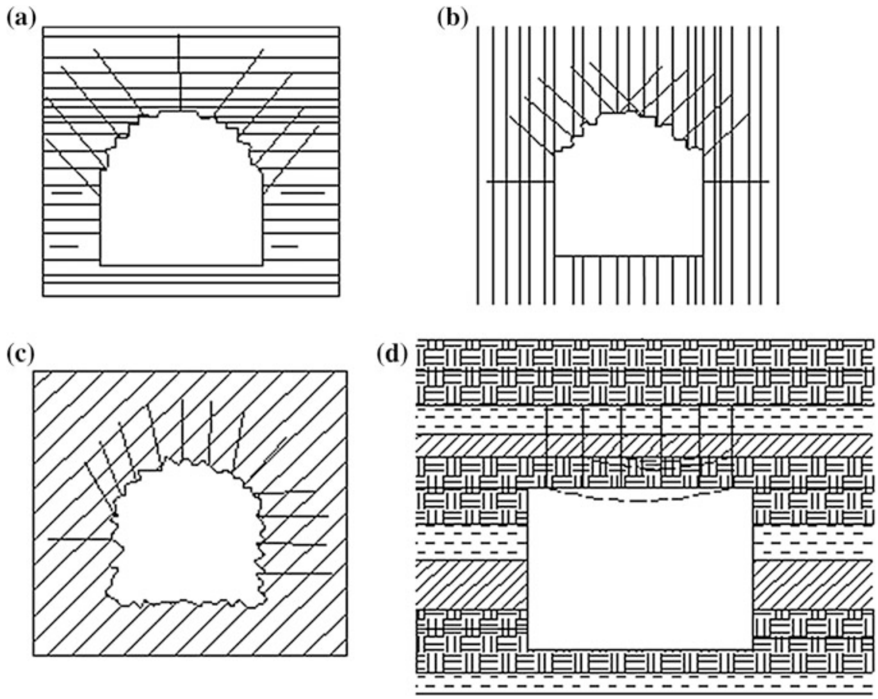


Fig. 2.12 Optimized arrangement of anchor bolts

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].

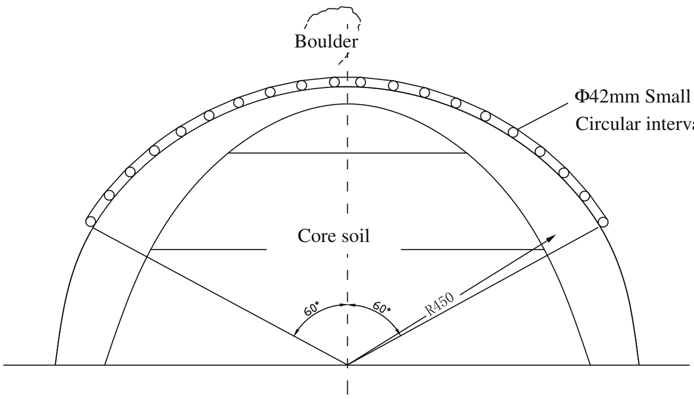


Fig. 2.14 Strong pre-reinforcement to ensure safe excavation of tunnels

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 stresscan 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.

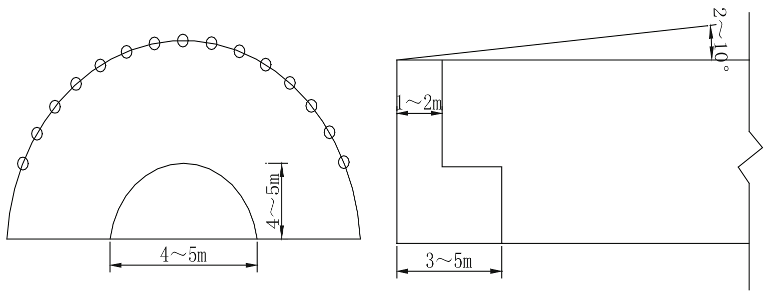


Fig. 2.15 Schematic diagram for appropriate full-face excavation with pre-reinforcement and lower pilot tunnel ahead

(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 increases 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 determines 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							
			Reserved deformation (mm)	Strong pre-reinforcement	(Ultra) short bench	Primary support	Timely installation of secondary lining	Inverts	Water control	Others
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; I20 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	Jiazhuqing 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							
			Reserved deformation (mm)	Strong pre-reinforcement	(Ultra) short bench	Primary support	Timely installation of secondary lining	Inverts	Water control	Others
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.

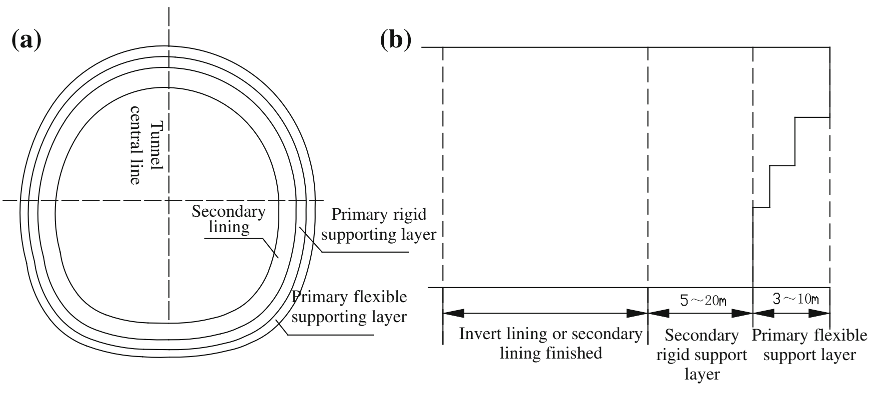


Fig. 2.16 Schematic diagram for construction and lining of class IV surrounding rock

(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 inverts 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 m^2, about 140 m^2 and more than 170 m^2. 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 increae, 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 descrease, 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	Recommended scheme	Used scheme	Recommended 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 bench method; (2) Central pillar method; (3) CD, CRD method (4) Top heading and bench method with temporary closing	Deep buried: (1) Top heading and short bench method; (2) CD, CRD method Shallow buried: (1) CD, CRD method (2) Central pillar method	(1) Top heading and short bench method; (2) CD, CRD Method; (3) 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	10–25	10–15	15–25	15–20
Rockbolt (m)/(circular × longitudinal)	2.5–3.0/(1.5 × 1.2)	2.5/(1.2 × 1.2)	2.5–3.5/(1.0 × 1.0)	2.5/(1.0 × 1.0)	3.0–4.05/(1.0 × 1.0)	3.0/(1.0 × 1.0)
Steel sets model/distance(m)	H150/1.5	—	H200/1.5	—	H200/1.0	—
Pre-reinforcement	—	—	—	—	Small grouted pipes	Small grouted pipes
Lining thickness (m) (roof/invert)	(0.25–0.5)/(0–0.5)	(0.25–0.3)/(0.25–0.3)	(0.25–0.5)/(0–0.5)	(0.25–0.35)/(0.25–0.35)	(0.3–0.6)/(0.3–0.6)	(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; Shallow buried: (1) CD, CRD method (2) Central pillar method;	(1) Top heading and bench method with temporary closing; Top heading and bench method (short and super short bench) (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	10–15	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)	4.5–6.5/(1.0 × 0.8)	4.5/(1.0 × 1.0)
Steel sets model/interval (m)	H250/1.5	Replaced with lattice girds	H250/1.0	Replaced with lattice girders	H250/0.8	Replaced with lattice girders

(continued)

Table 2.5 (continued)

Span	10 m		15 m		20 m	
Pre-reinforcement	Small grouted pipes; forepole umbrella	Small grouted pipes; forepole umbrella	Small grouted pipes; forepole umbrella; footing grouting; spiral spraying pipes	Small grouted pipes; forepole umbrella; footing grouting; spiral spraying pipes	Small grouted pipes; forepole umbrella; footing grouting; spiral spraying pipes; roof lining	Small grouted pipes; forepole umbrella; 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.

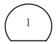
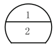
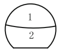
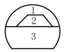
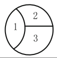
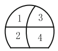
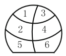
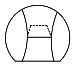
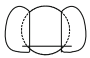
When the STM is used, the typical construction methods include positive top heading and bench method, while other construction methods suitable for special strata conditions include the full-section method, top heading and bench method with unilateral pilot tunnel, central pillar method (spectacles technique)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

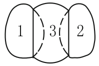
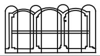
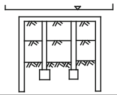
In weak and loose strata in urban areas, with respect to strata displacement control, the priority order of STMs is as follows: CRD → central pillar → CD → top heading and bench method with temporary closing heading → top heading and bench method.

Table 2.6 Main excavation methods for underground structures using STM

Construction method	Schematic diagram	Comparison of important indexes					
		Application conditions	Subsidence	Construction period	Waterproof	Effort for dismantling temporary support	Cost
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					
		Application conditions	Subsidence	Construction period	Waterproof	Effort for dismantling temporary support	Cost
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 ***B***	1.5 ***B***	2.0 ***B***	2.5 ***B***	3.5 ***B***	4.0 ***B***

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, lessens 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

$$\begin{cases} \sigma_r = P_0\left(1 - \dfrac{a^2}{r^2}\right) \\ \sigma_\theta = P_0\left(1 - \dfrac{a^2}{r^2}\right) \end{cases}. \tag{2.2}$$

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

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

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 main 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;

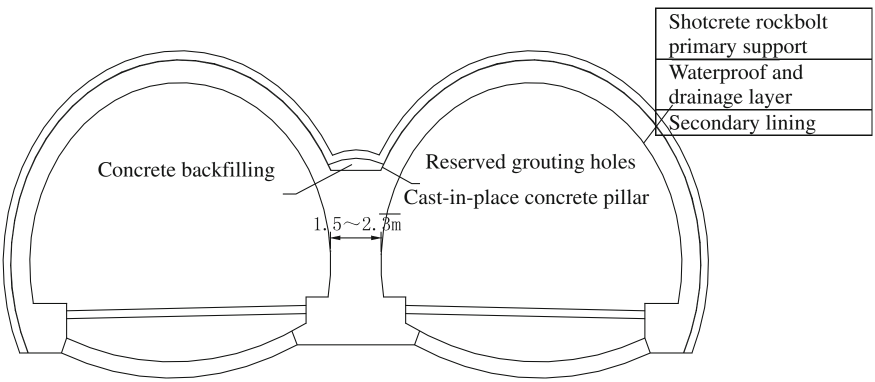


Fig. 2.19 Schematic diagram of traditional section

(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.

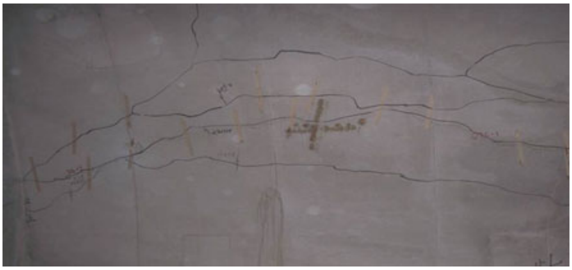


Fig. 2.20 Distribution of cracks on secondary lining

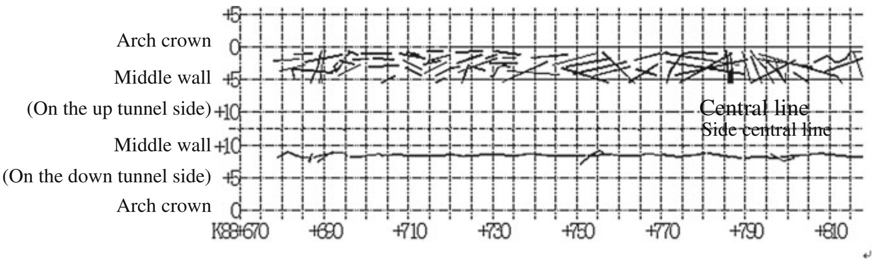


Fig. 2.21 Crack distribution diagram

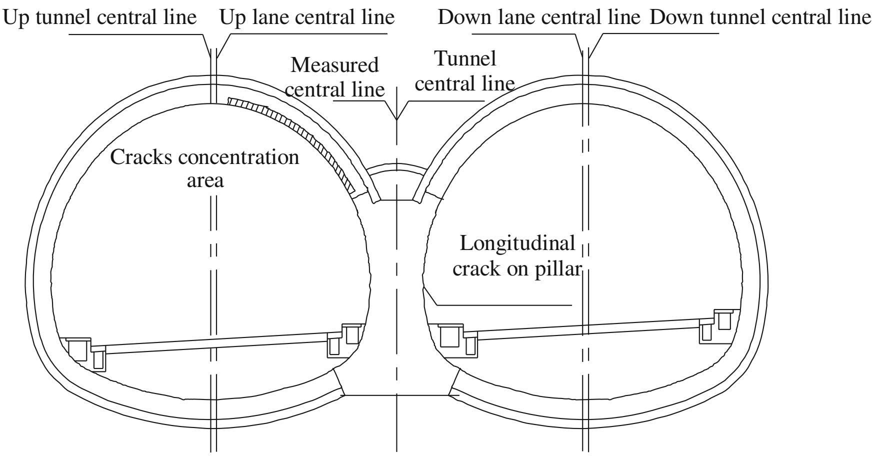


Fig. 2.22 Diagram of crack concentration area

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

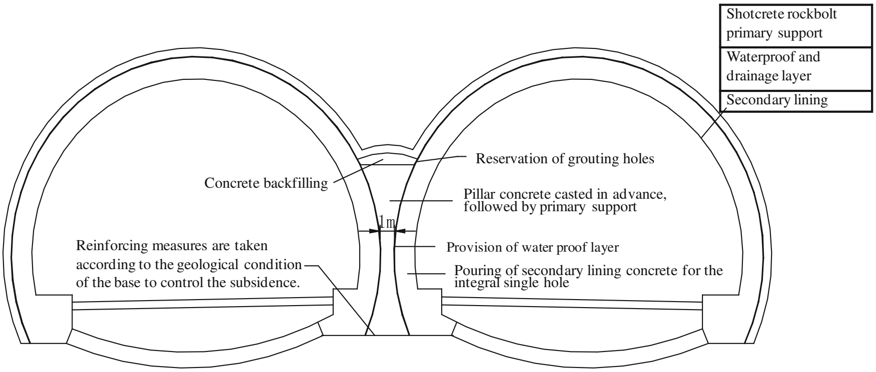


Fig. 2.23 Schematic diagram of improved section

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 geotexile 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

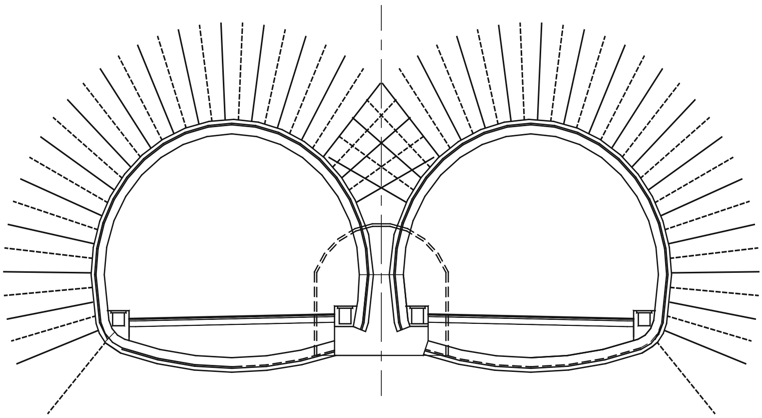


Fig. 2.24 Schematic diagram of optimized section

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 adressed 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.

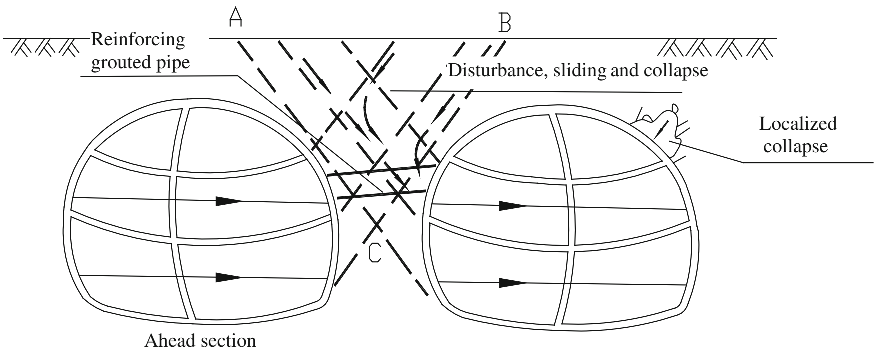


Fig. 2.25 Reinforcement of rock pillar

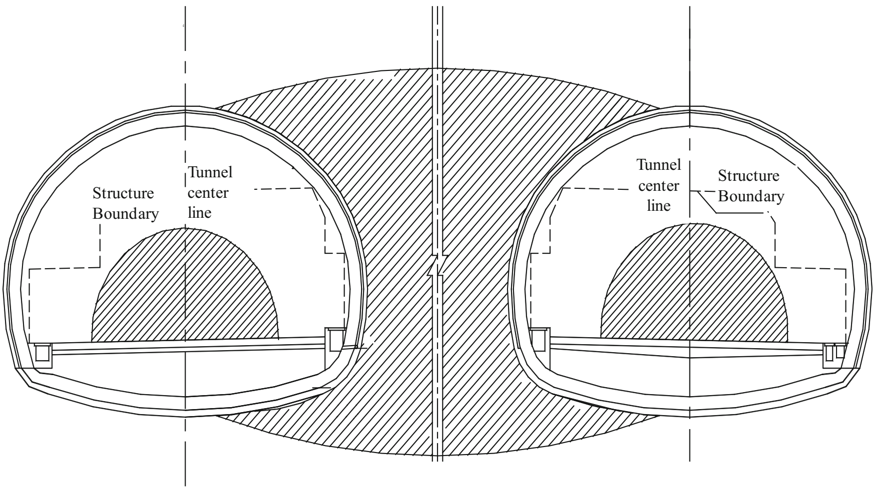


Fig. 2.26 Middle-pilot-tunnel ahead method and reinforcement of rock pillar for small-clearance tunnel in hard rock

(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,

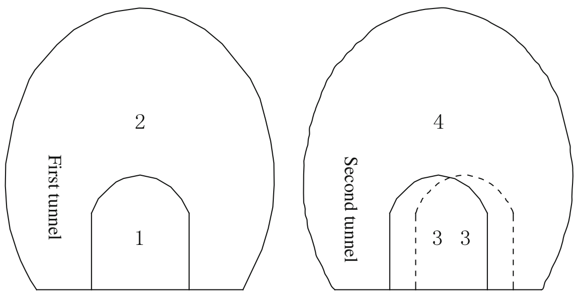


Fig. 2.27 Excavation scheme of small-clearance tunnels using pilot tunnel method

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 ↑ hazard control) + T (supporting force) (↓ 0) ≥ 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.

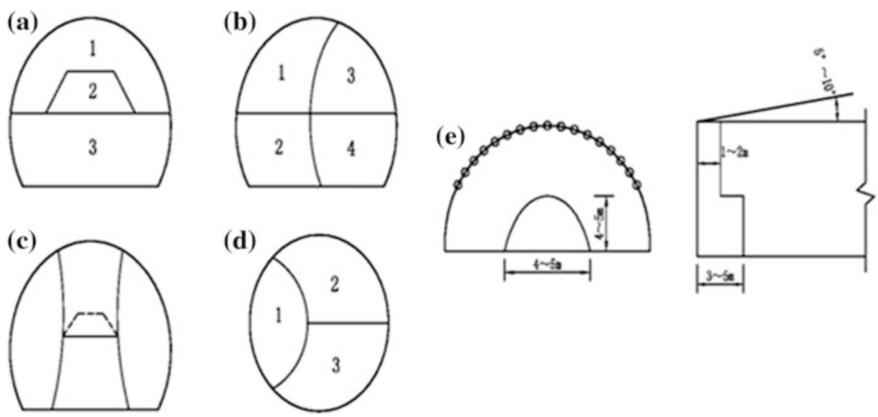


Fig. 2.30 Control of space stability for excavation and support of mountain tunnels under poor geological conditions

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

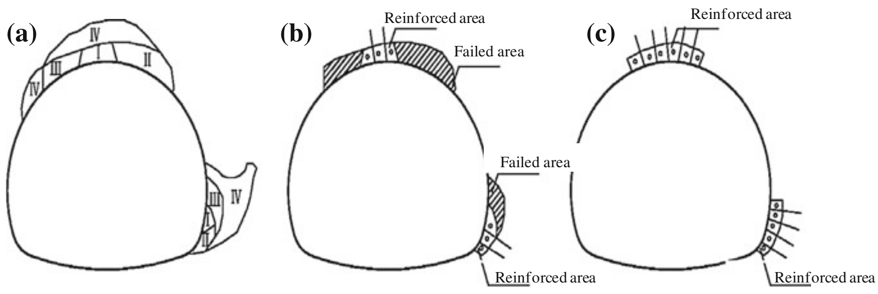


Fig. 2.31 Evolution of space stability of mountain tunnels during excavation and supporting under general geological conditions

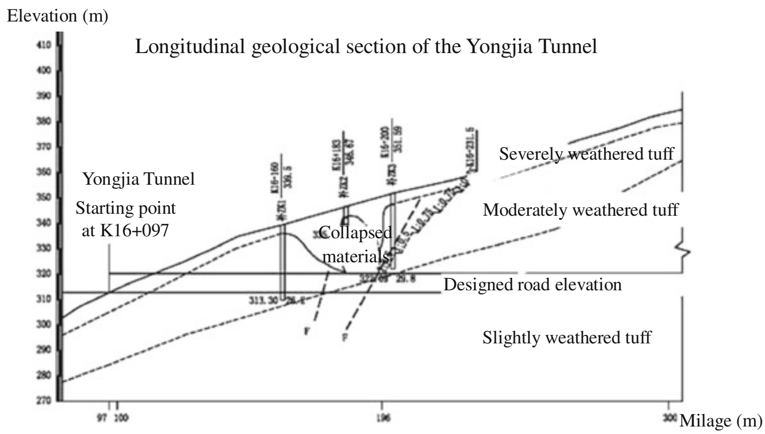


Fig. 2.32 Sudden collapse of about 2,420 m^3 in Yongjia Tunnel during excavation

of collapse was about 11–10 m respectively; the height and area were about 22 m and 2420 m^3 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

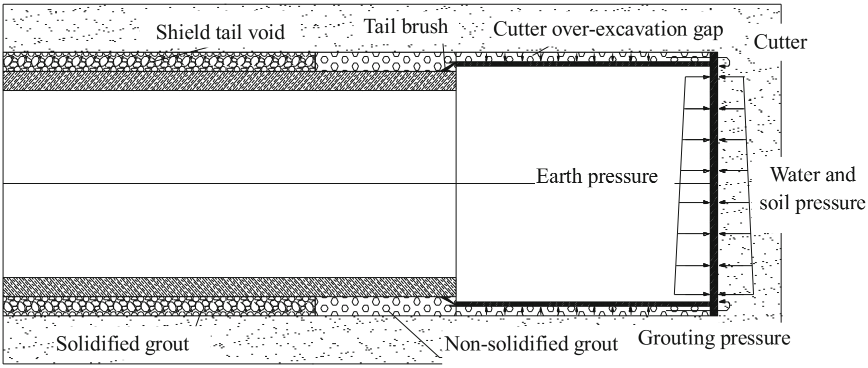


Fig. 2.33 Schematic diagram of shield tail void and grouting

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

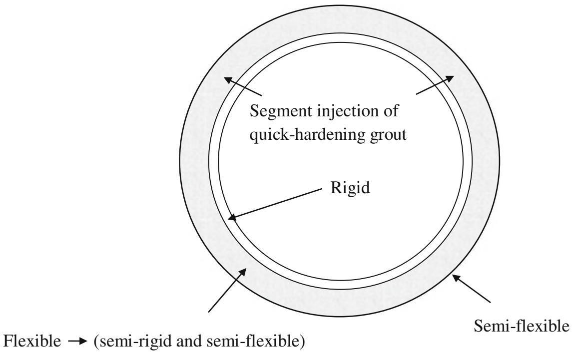


Fig. 2.35 Even elastic tires can better control deformation compatibility and even stress of wheel hubs than the rubber pneumatic tires

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 viscidity 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

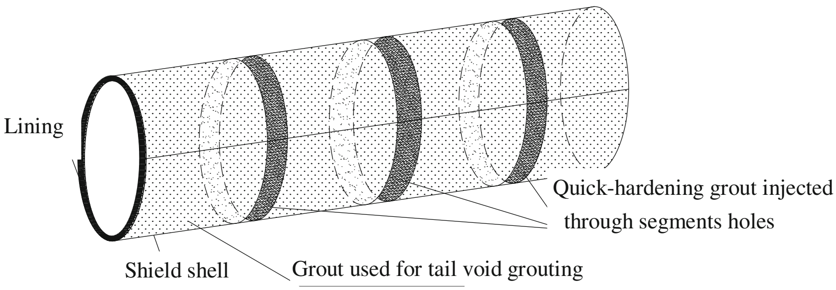
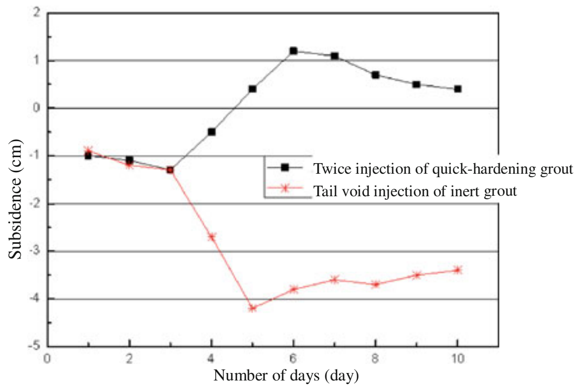


Fig. 2.36 Stability of hinged segments in weak strata

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.

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Part II
Application of Stable Equilibrium Theory to Construction of Mountain Tunnels

Chapter 3
Deformation Compatibility Control Technique for Tunnels in Soft Strata with Mixed Soil and Rock

Abstract Tunneling in soft strata with mixed soil and rock is usually complicated and requires great efforts to maintain the stability of tunnel and excavation face, especially for the tunnel entrance sections. The upper part of a tunnel is generally covered with loose deposits while the lower part passes through weathered rock. The surrounding rock has very low strength and barely any stabilizing capacity. This chapter introduced how to use the underground engineering stable equilibrium theory and corresponding techniques to solve such a case.

3.1 Basic Information

The geological investigation was conducted according to the local code [1, 2]. The main layer above the roof of the buried sections contains loose silty clay with gravel, which is 20 m to 25 m thick embedded with rock blocks. The layers below lying beneath is composed of weathered bedrock. The severely weathered layer with extremely fractured rock mass is 2–5 m thick. The moderately weathered layer with poor stability rock is 10–20 m thick. The underground water is predominated by quaternary pore water and bedrock fissure water. The impact of underground water is noticeable. Upon start of excavation, there was serious water seepage and even water burst in the tunnel. *Local residents said that there is no runoff on the ground surface during rainstorm. This means that there is serious water leakage as the strata is loose, which is in accordance with exposure at the tunnel entrance and the geological map as shown in Fig.* 3.1.

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Fig. 3.1 Geological condition at entrance of the Tunnel

At the entrance of the tunnel, there are many boulders. On March 20, 2014, the design company, contractor, supervisor and construction company visited the site and conducted a survey, and optimized the design for the entrance. The entrance of buried section was then pushed forward by 5 m (adjusted from K1+701–K1+706). However, the following problems were encountered after construction, which resulted in difficulty on the construction site.

(i) After excavation and supporting of the first stair of the slope above the open cut section, collapse occurred below the first stair during excavation of the second stair;
(ii) When excavating the foundation of the large forepole umbrella, grey brown silty clay was found below central point of the tunnel. The clay got softer as the excavation went deeper. The wet clay was very much like the mud;
(iii) There was serious water seepage on the upper and lower part of the roof crown, and on the right side slope;
(iv) The bearing capacity of the foundation of the open cut sections was insufficient.

3.2 Construction Scheme for the Initial Stage

The slope gradient of the cut slopes was reduced to 1:1. Excavation would be carried out from upside down. The height of each excavation stair was controlled at about 2 m. Supporting was provided in the meantime. The supporting parameters were adjusted to *C20* shotcrete (15 cm thick) with double layers of steel nets (Φ6.5-20 cm × 20 cm). Small grouted pipes Φ42 was adopted in a plum-shape layout (4 m long, with an interval of 1.0 m × 1.0 m). Excavation, shotcrete, and rockbolt application and steel nets hanging were carried out simultaneously [3].

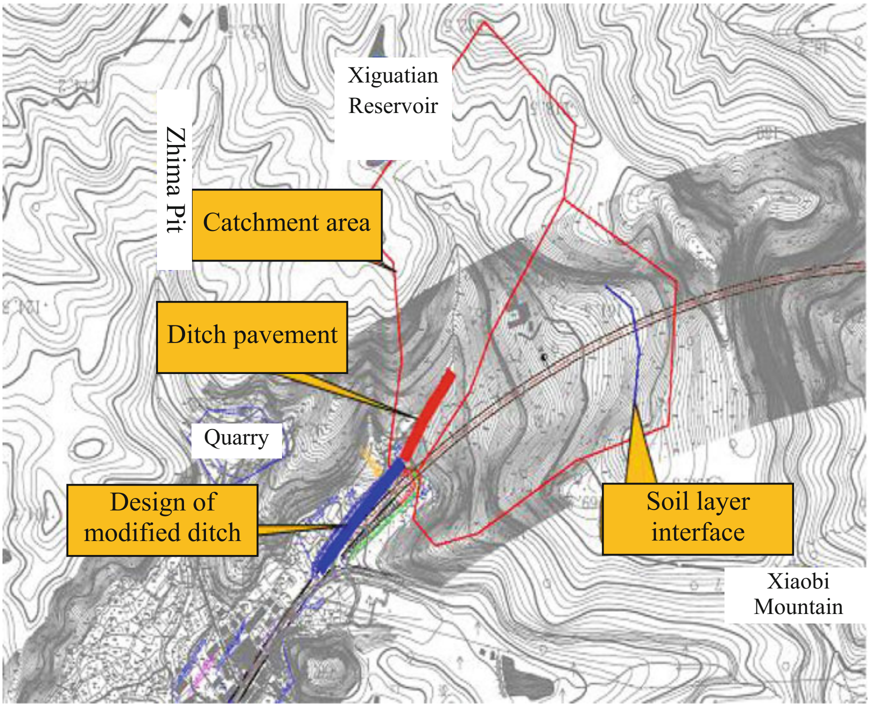


Fig. 3.2 The topographical map of the construction site

(i) Surface drainage: there was a large gully on the left of the tunnel. This gully stretched from the entrance of the tunnel all the way into the mountain. The gully was dry within 220 m, beyond which there was water flow. To prevent seepage of gully water into underground, which might impact tunnel construction, the drainage measures were taken for this section [4]. 30 cm thick *C20* concrete was paved on the bottom of the existing gully to prevent seepage of runoff water. The gully was connected to the modified ditch downstream. Meanwhile, 30 cm thick *C20* concrete was also paved on the bottom of the modified ditch to prevent seepage of water into the open cut and road sections. The pavement length was about 426 m.

(ii) There was a small gully 30 m away from the entrance of the buried sections on the right side. Another small ditch was plowed to drain the water into the catchwater outside the open cut sections as shown in Fig. 3.2.

(iii) (iii) Water drainage of the tunnel

(a) The small amount of water seepage could be treated after primary support. If l points were found on the sidewall, PVC pipes (or Φ5 cm flexible pervious pipe) should be inserted into the pre-drilled holes on the wall for diversion.

(b) Serious water seepage on sidewalls: pre-treatment should be provided before primary support. Holes should be drilled at the seepage points and PVC pipes (or ϕ5 cm flexible pervious pipes) would be inserted for diversion.

According to the original design, the base of open cut sections should be replaced with 50 cm thick *C20* concrete. According to the geological report, the bearing capacity of silty clay with gravel is between 200 and 260 kPa. The bearing capacity was low because the drainage was blocked, and the water softens the foundation soil. Therefore, after the drainage is working, the bearing capacity of the foundation soil should be tested. If the capacity is lower than 300 kPa, then the actual test data should be used to check whether the 50 cm thick *C20* concrete can meet the bearing capacity requirements. If not, 1 m *C20* concrete should be used instead. If the requirements are still not met, reinforcement with steel pipe piles or other schemes should be used.

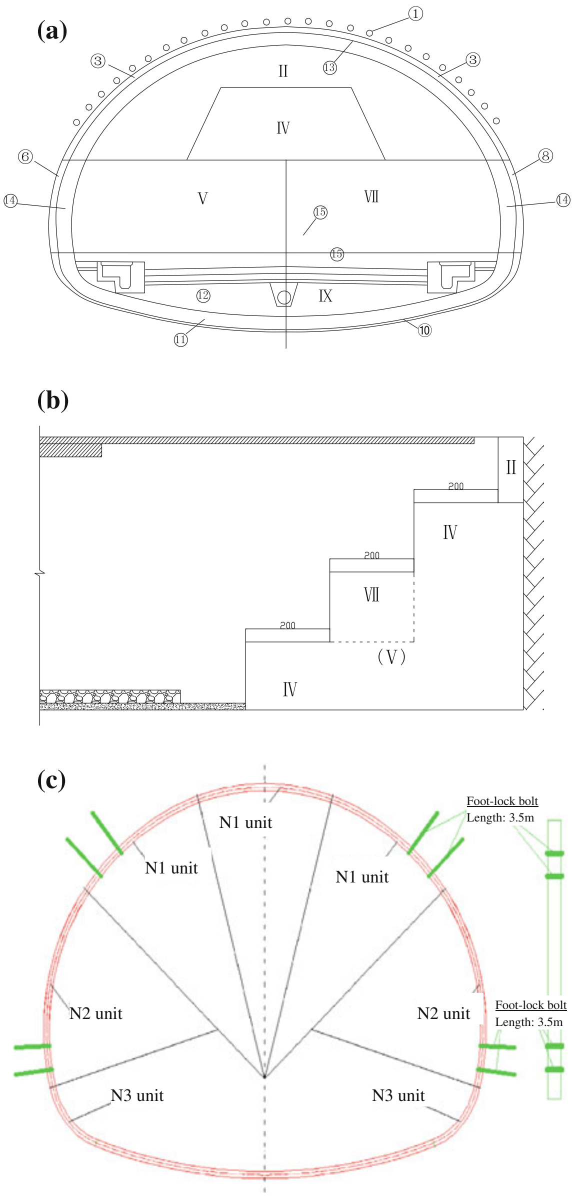
1. **Installation of Forepole Umbrella**

The cover layer above the roof of buried sections is composed of very loose rock, embedded with many rock blocks and boulders. During construction, engineers may encounter hole collapse, deviation of hole location and difficulty in drilling. To ensure construction quality of forepole umbrella, the hole drilling with casing method should be adopted. The original design of large forepole umbrella: arrangement of centric angle of 100°, with circular interval of 42 cm, totally 35 forepoles for each cycle. Afterwards, in order to guarantee construction safety and prevent collapse of sidewalls and improve self-stabilizing capacity of sidewalls, the number of forepoles was increased. The large forepole umbrella should be arranged with a centric angle of 120°, with circular interval of 42 cm and 43 pipes for each cycle. The longitudinal length that use forepole umbrella is 52 m (K1+704–K1+756).

2. **Tunnel Excavation and Supporting**

For 45 m long entrance (K1+706–K1+751) section, the following construction sequences were adopted. The cover layer above the tunnel roof is predominated by silty clay in a loose status, with poor self-stabilizing capacity. To improve the stability of surrounding rock and ensure construction safety of the tunnel face and reduce potential of collapse, the face buttress method was used, with 2 m long buttress. The tunnel face advanced 50 cm each excavation-support cycle. Small grouted pipes was also adopted on the horizontal and longitudinal directions of the sidewalls. Φ70 small steel pipe piles were used at the tunnel bases. The supporting parameters were improved by replacing 18# I-shaped steel sets with 20# I-shaped ones, with a longitudinal interval of 50 cm. The number of foot-lock bolts was increased, with shotcrete thickness of 30 cm instead of previous 26 cm. Furthermore, the length of open cut sections was reduced by 15 m (K1+680) as shown in Fig. 3.3.

Fig. 3.3 Schematic diagram for excavation and supporting of buried sections



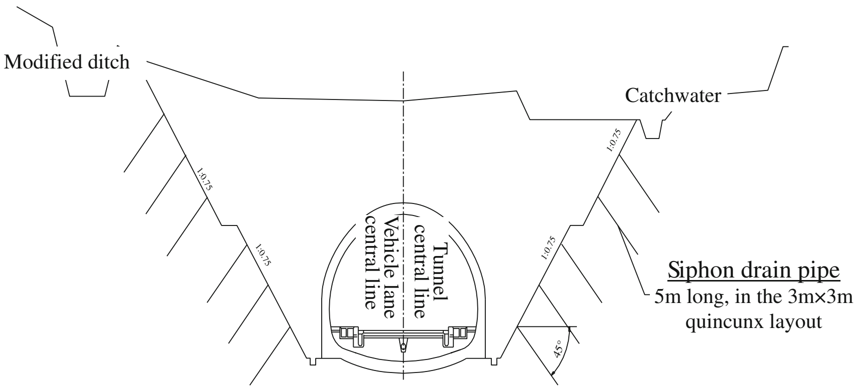


Fig. 3.4 Drainage plan for the side slope of open cut sections

Actually, the initial construction scheme of the tunnel has the following problems: (1) the slope drainage method of the open cut sections is not good for stability of side slope during construction; (2) the distance of un-closed sections along longitudinal direction is too long, which is not good for maintaining stress equilibrium during construction; (3) the top heading steel sets of the primary support cannot form a loop independently, which is not good for stress equilibrium and deformation control during construction; (4) the replaced gravel layer of tunnel base could not form close structure in time, which may result in squeezed deformation or even failure of the primary support of the bench. The above four problems may introduce potential construction risks, which is not good for protecting tombs and plants on the hillsides at the entrance of the tunnel.

3.2.1 Improvement of Construction Scheme

Considering the four problems discovered in the initial construction scheme, the following improvements have been made by the design and construction companies. Please refer to Figs. 3.4, 3.5, 3.6 and 3.7.

Therefore, the improved construction scheme of the tunnel can maintain stability of the tunnel face and surrounding areas during construction. It may also help avoid risks and protect of tombs and plants on the hillsides at the entrance of the tunnel.

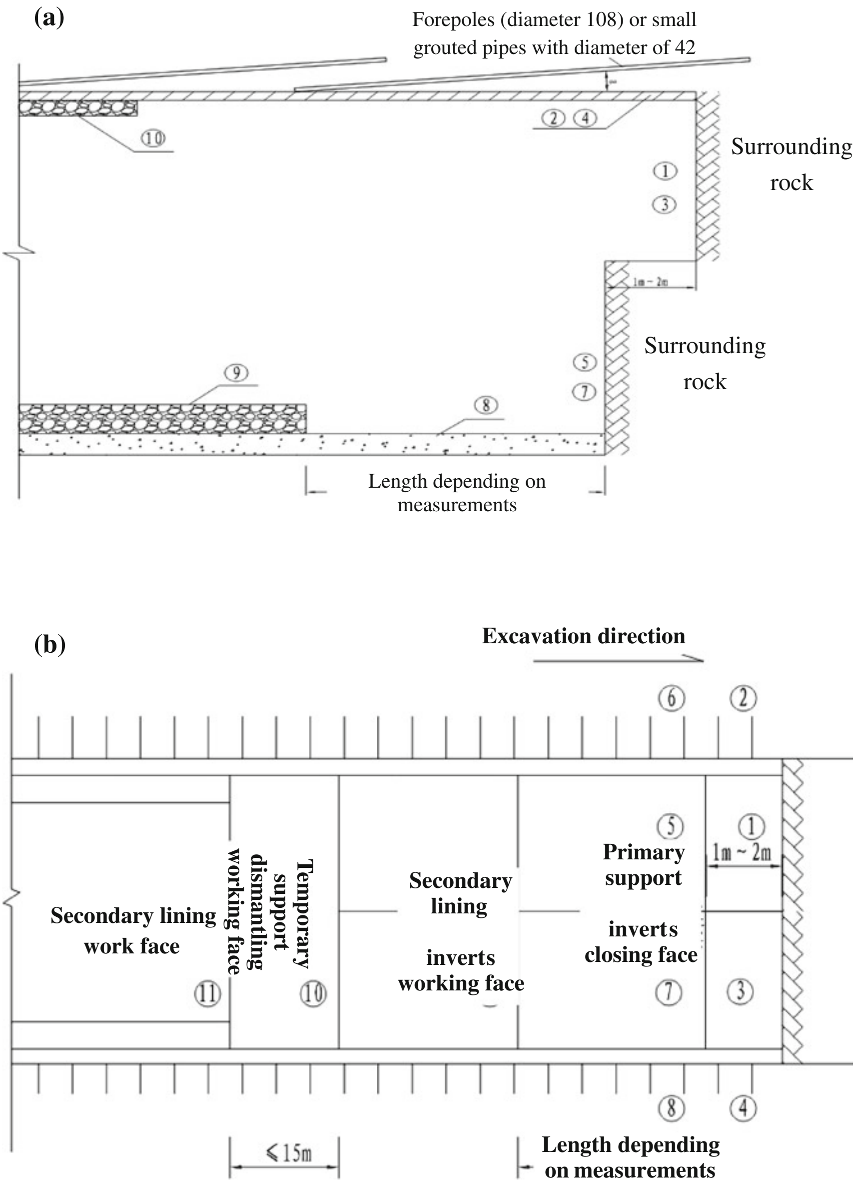


Fig. 3.5 Sequences of longitudinal excavation

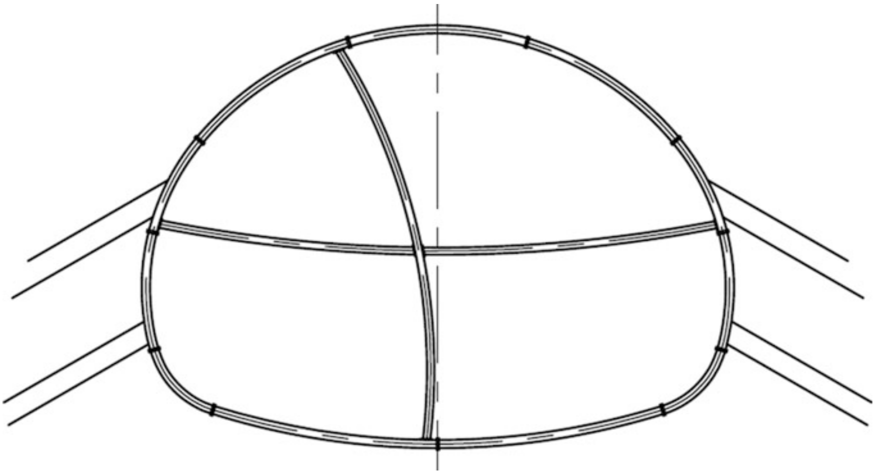


Fig. 3.6 Structural diagram of steel sets

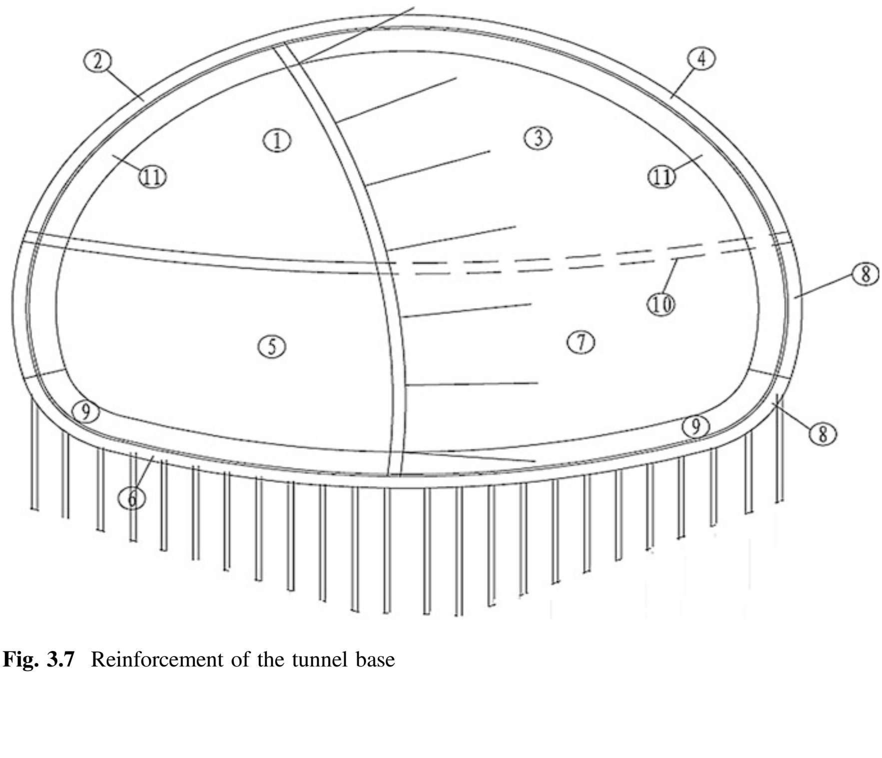


Fig. 3.7 Reinforcement of the tunnel base

Fig. 3.8 Sampling location and prepared specimen

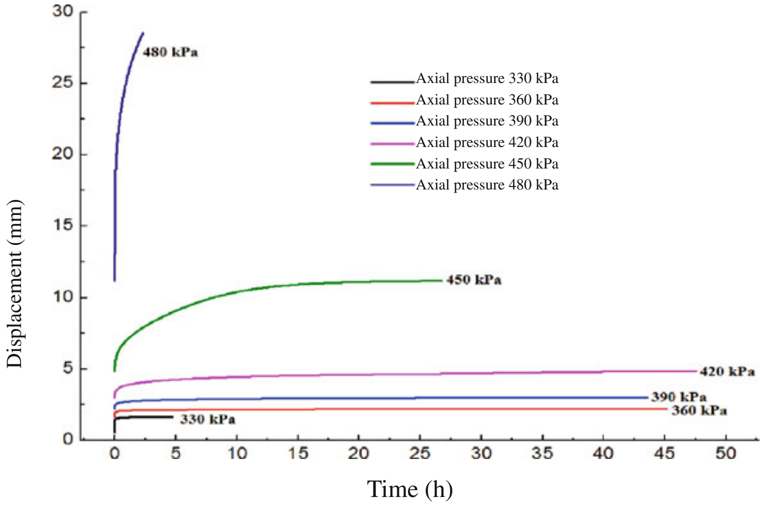


Fig. 3.9 Deformation response of soft soil to different axial stress

3.3 Testing and Verification

To verify the improved scheme, cylindrical specimens with diameter of 900 mm and height of 2000 mm were taken from the bottom and middle part of the tunnel. The composition of specimens was complicated, including the mixture of intensely weathered gravels, gravels and soil as shown in Fig. 3.8. The relation between the surrounding rock deformation and resistance at the bottom and middle part of the tunnel was discovered through laboratory tests, so as to verify the important role of the excavation and supporting sequence in maintaining spatial stability when tunneling under poor geological conditions.

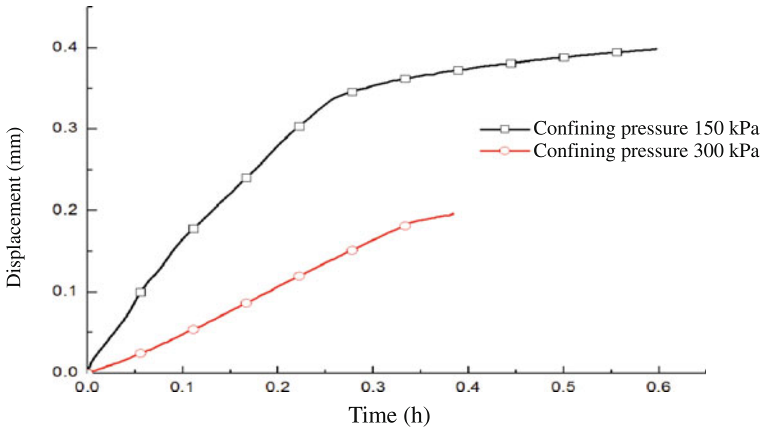


Fig. 3.10 Deformation response of soft soil to different confining pressures

As shown in Fig. 3.10, the displacement of soft soil is 4 mm and 2 mm when confining pressure is 150 and 300 kPa respectively. This means that with higher supporting rigidity and higher confining pressure, there will be smaller surrounding rock deformation. In such a case, the surrounding rock deformation or deformation saltation can be better controlled. The general trend of displacement variation against different axial stress as shown in Fig. 3.9 experienced salutatory deformation. This indicates that if the surrounding rock is composed of severely weathered gravel, gravel and soil, then deformation will be more salutatory as confining pressure increases. It is of great significance to effectively control the deformation compatibility of surrounding rock. Therefore, pre-reinforcement and simultaneous supporting and excavation in the improved scheme is very important for each step to maintain the spatial stability during construction of the tunnel. Otherwise, the tunnel surrounding rock deformation cannot be effectively controlled, which may result in salutatory deformation and safety problems of tunnels.

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Chapter 4
Reasonable Construction Methods for Tunneling Beneath an Operating Highway

Abstract The main controlling target for tunneling beneath expressway is that the subsidence of the road should be within a certain limit to ensure regular operation of the road. It is very important to choose a reasonable construction method for such projects. Therefore, selection of reasonable tunneling schemes and control of strata subsidence are greatly important to control the impact on existing roads within an allowed limit. In this chapter, the underground engineering stable equilibrium theory and corresponding techniques were applied to the case of an undercrossing tunnel.

4.1 Engineering Background

One tunnel passes beneath an expressway. The plan of the tunnel is shown in Fig. 4.1. The clearance between the roof crown and the road surface of the expressway is about 3.5–5 m, which falls into the category of shallow-buried tunnel as shown in Fig. 4.2. The tunnel passes through mainly completely and moderately weathered granite. The completely weathered layer is very thick, and in the acid color. The core of the borehole looks like loose sandy soil. The underground water mainly includes bedrock fissure water, which is stored in the fractured rock mass. The water is comparatively sparse, and is supplied by rainfall infiltration. Therefore, there will be more underground water in rainy seasons.

As shown in Fig. 4.2, in order to guarantee the safety of engineering structure during tunnel construction, the STM theory and construction method proposed by Academician Mengshu [1, 2] may be adopted, and the related regulations on corresponding codes may be followed for design and construction. Furthermore, the pressure induced by the surrounding rock on the top of tunnel roof may be determined according to M. Лромобъяконоб theory and K. Terzaghi theory. Surrounding rock deformation can be controlled using the strong pre-reinforcement

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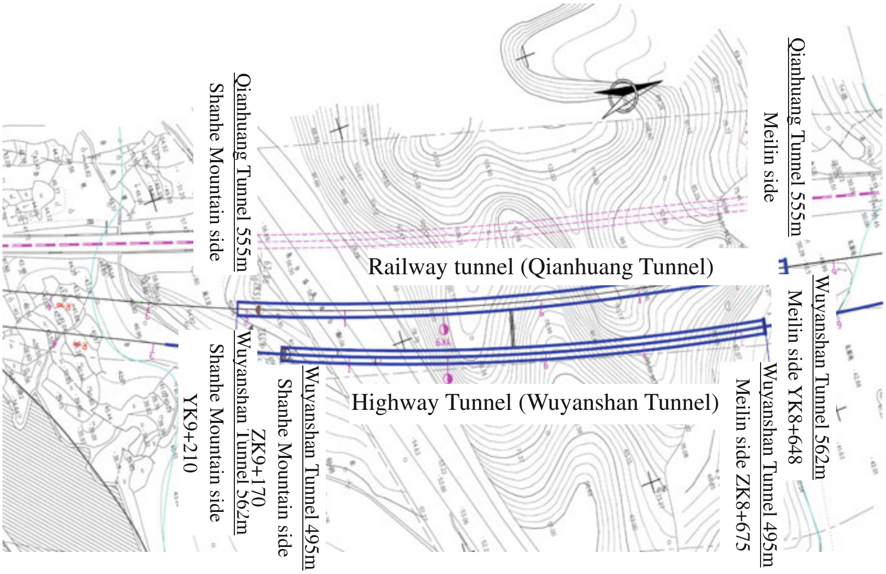
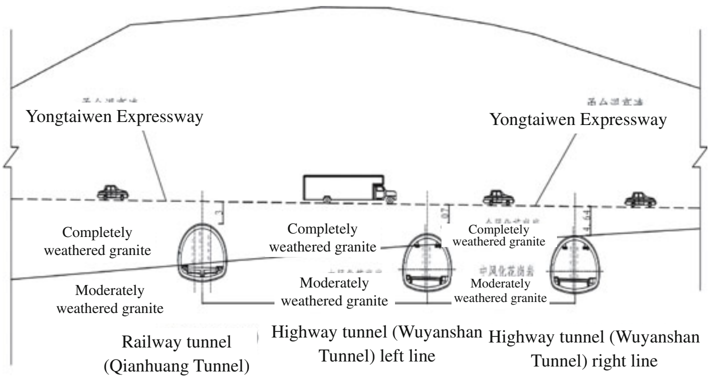


Fig. 4.1 Plan of the tunnel

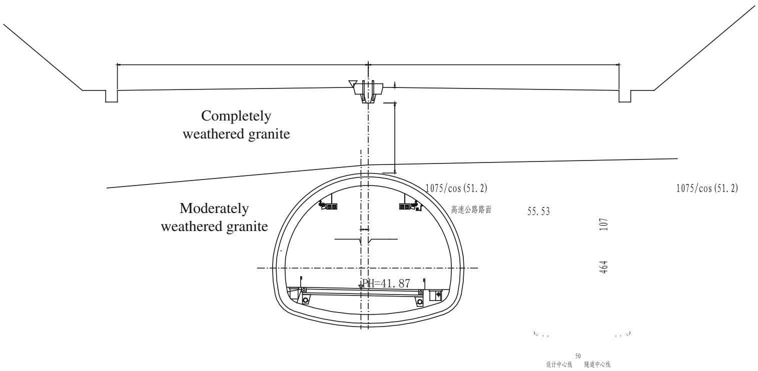
technique. In such a case, multiple-heading excavation, timely strong pre-reinforcement, short advance step and timely closing become extremely important. The core is *the effectiveness of the underground engineering bearing layer, and the timeliness of formation of the structural bearing layer* (*i.e., the time-space effect*), i.e., basically maintaining the original status of surrounding rock, and prevention and mitigation of localized failure or instability of tunnel surrounding rock, which may result in overall instability of the tunnel. During each step of construction, the rock- support interaction should be kept in a stable equilibrium and deformation compatibility control status.

4.2 Lessons Learned from Similar Tunnel Design Schemes

Please see Fig. 4.3a for the preliminary design scheme of the tunnel. In the scheme, the excavation area of the top heading in the surrounding rock is too big, and the installation time of primary support was too long. Without timely closing of the base to form a closed loop, the equilibrium stability and deformation compatibility control requirements cannot be met, which was not good for controlling tunnel deformation and caused loosen of cover rock and excessive subsidence of road surface. Accidents once occurred in tunnels that use such a scheme as shown in Fig. 4.3b.



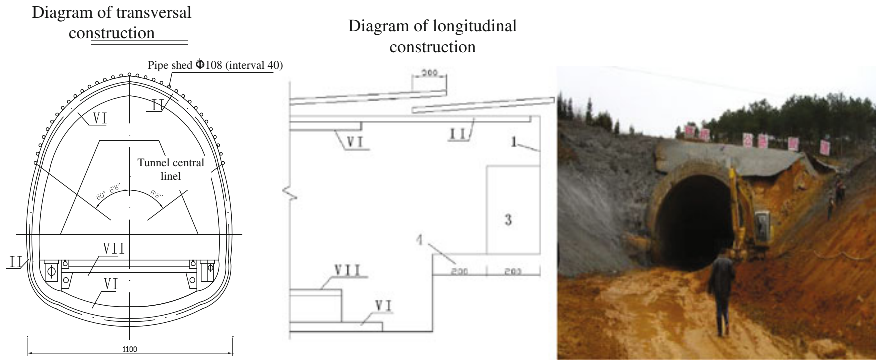
(a) cross section of the railway/highway tunnels



(b) cross section of the intersection between the tunnels and expressway

(c) photo of the railway tunnel passing beneath an expressway

Fig. 4.2 A railway tunnel passing beneath an operating expressway



(a) construction scheme (b) failure scene

Fig. 4.3 Failure of shallow-buried tunnel construction scheme

For construction of a railway tunnel beneath a highway, the following supporting and construction scheme should be considered.

(1) **Pre-reinforcement**

The Φ150 mm forepole umbrella was used. The forepoles are made of hot rolled seamless steel pipe with wall thickness of 6 mm. To ensure the construction quality and effectiveness of the forepole umbrella, 45 m long forepoles are installed to create a 30 m overlap between successive umbrellas on both ends of the tunnel. The circular interval of forepoles was 40 cm as shown in Fig. 4.4.

The mileage of exit forepole umbrella is DK70 + 274–DK70 + 229. The sections close to the side ditch on the exit side was excavated using open cut method to make space for forepole installation. The umbrella in the buried section was located at DK70 + 215–DK70 + 260. To ensure smooth installation of forepole umbrella,

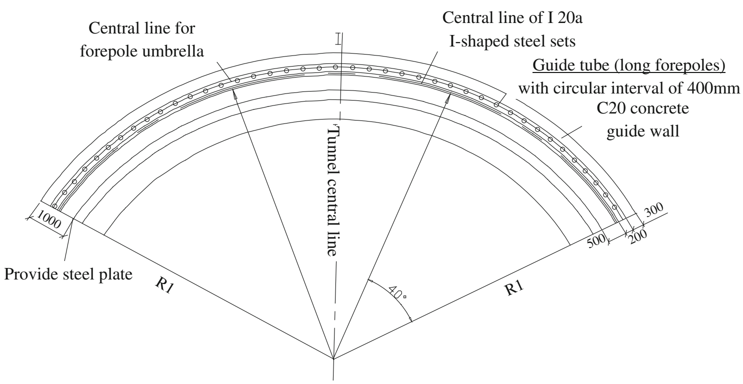


Fig. 4.4 Forepole umbrella support for railway tunnel

1 m radial over-excavation was carried out within 120° of the arch at sections DK70 + 200–DK70 + 215 to make space for forepole installation.

The borehole diameter for forepoles was 180 mm. In the umbrella, reinforcement cage was installed. The cage was welded using four Φ22 main reinforcement bars to improve the bending resistance of the umbrella.

(2) **Supporting Measures**

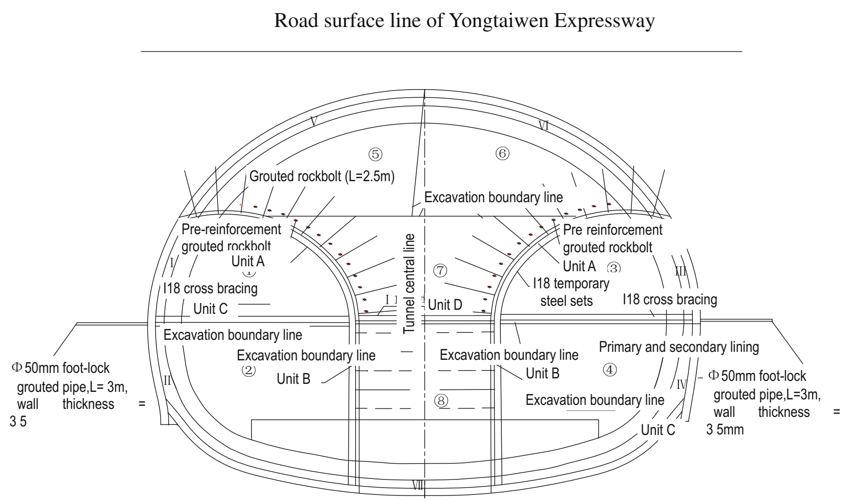
To guarantee strength of primary support and minimize the deformation of the road surface, one layer of initial stage secondary lining should be provided. Excavation and supporting was carried out using the central pillar method. In addition, the initial stage reinforcement concrete secondary lining was provided soon after the excavation for the respective part, with 20–25 cm lining.

The I20a I-shaped steel sets was used to reinforce the primary support. The steel sets were provided at a longitudinal interval of 0.5 m.

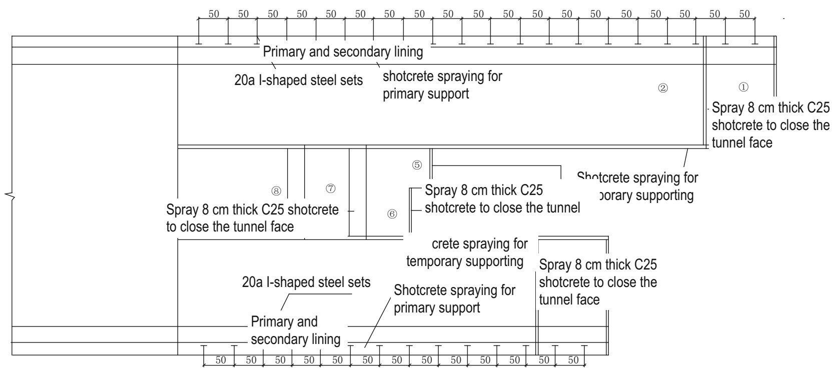
(3) Excavation Method

The central pillar method was adopted as shown in Fig. 4.5. The construction steps are shown as follows:

(1) The steel sets installed in the previous cycle was used as the pre-reinforcement for the tunnel; excavate part ① using the weak blasting; spray 8 cm thick shotcrete to reinforce the tunnel face; apply the primary support and temporary supporting for the pilot tunnel perimeter of part ①, i.e., preliminary spraying of 4 cm thick shotcrete and installation of I20a steel sets and I18 temporary steel sets; foot-lock anchors was provided, and I18 cross brace was also provided. After provision of system rockbolts, re-spraying of the shotcrete was required to reach the designed thickness; then the permanent supporting shotcrete for initial secondary lining of part ① was provided.
(2) After lagging behind part ① for a certain distance, part ② was carried out by weak blasting; spray 8 cm thick shotcrete to close the tunnel face; initially spray 4 cm b for the pilot tunnel perimeter; install I20a steel sets and I18 temporary steel sets for part ②; re-spraying shotcrete was required to reach the designed thickness; then the permanent secondary lining for part ① was provided.
(3) The steel sets installed in the previous cycle was used as the pre-reinforcement; excavate part ③ by weak blasting and provide primary support and temporary support for perimeter of the pilot tunnel in the same sequence as indicated in the first step.
(4) Excavate part ④ by weak blasting; provide primary support and temporary support for the perimeter of pilot tunnel; the sequences are shown in the second step.
(5) The steel sets installed in the previous cycle was used as the pre-reinforcement; excavate part ⑤ by weak blasting; spray 8 cm thick shotcrete to close the tunnel face; initially spray 4 cm thick shotcrete on the perimeter of the pilot tunnel; install the I20a steel sets for part ⑤; Re-spraying



(a) Cross section drawing for construction



(b) Horizontal section drawing for construction

Fig. 4.5 Construction sequences of railway tunnel using central pillar method

shotcrete was required to reach the designed thickness after provision of system rockbolts; apply initial secondary lining and permanent support for part ⑤.

(6) The steel sets installed in the previous cycle was used as the pre-reinforcement; excavate part ⑥ by weak blasting; apply primary support and temporary support for the perimeter of the pilot tunnel in the same sequence as the fifth step.

(7) Excavate part ⑦ by weak blasting; spray 8 cm thick concrete to close the tunnel face; provide I18 cross bracing for part ⑦.
(8) Excavate part ⑧ by weak blasting; spray 8 cm thick shotcrete to close the tunnel face; initially spray 4 cm thick shotcrete on the pilot tunnel base; install I20a steel sets to form a closed loop; re-spray the shotcrete to the designed thickness; apply initial secondary lining and permanent supporting for part ⑦.
(9) Remove the steel sets on the bottom of the two side walls where lining is completed.
(10) Inject grout into inverts for part ⑨ and fill the tunnel base;
(11) Remove I18 temporary steel sets and temporary cross bracing according to the measurement and monitoring results.

(4) **Blasting Method**

The blasting vibration velocity was controlled within 3 cm/s during construction blasting to reduce impact of blasting on the road surface. The specific measures are as follows:

The explosive payload of a single shot for the largest section was controlled within 1.004 kg. Short advance step was strictly followed for in-hole blasting. The maximum step of each cycle was 0.7 m. Excavation was completed in 8 sequences according to the design requirements. The millisecond detonator was used to control the blasting vibration velocity within 3 cm/s. Ensure the road would not be damaged due to blasting.

(5) **Traffic Organization for Yongtaiwen Expressway During Construction**

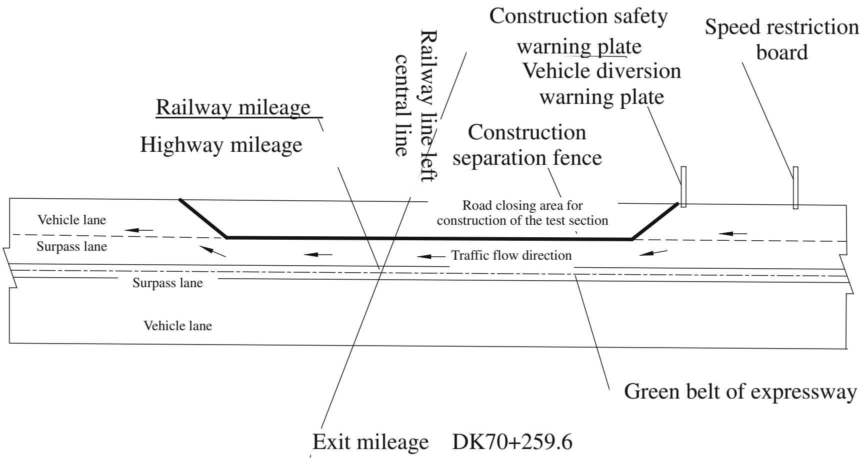


Fig. 4.6 Traffic organization chart for the expressway during railway tunnel construction

During construction of the tunnel, in order to reduce the load directly acting on the tunnel excavation face, one lane of Yontaiwen Expressway was temporarily closed to alleviate traffic load on the excavation face of the tunnel below, as shown in Fig. 4.6.

4.3 Highway Tunnel Construction Scheme

To avoid impacting adjacent operating railway tunnels and minimize impact on the traffic flow on the expressway, traditional blasting was not used for highway tunnels. Only mechanized excavation was used as shown in Fig. 4.7. As the excavation machinery cannot get into contact with any iron objects, no temporary supporting was allowed for the construction method. Therefore, the full-face excavation method with face buttress was used for Wuyanshan Tunnel to pass beneath Yongtaiwen Expressway. Mechanized excavation can minimize disturbance to surrounding rock caused by the blasting excavation method, and at the same time minimize impact on safe operation of Yongtaiwen Expressway.

To accommodate the machinery excavation scheme, the supporting parameters have been re-proposed for the section that passes beneath Yongtaiwen Expressway. MIDAS GTS software was used for checking calculation of the construction process to obtain the supporting structure stress and deformation condition during each excavation stage, and impact of the excavation method on the subsidence of Yongtaiwen Expressway. In this way, the feasibility of the excavation scheme was further verified, and provide a reference for optimization of supporting parameter, excavation procedures and measurement control values during construction. Consequently, safe operation of Yongtaiwen Expressway during tunnel construction period and safe construction of underpass section of the Wuyanshan Tunnel were guaranteed.

Fig. 4.7 Photo of the excavation machinery

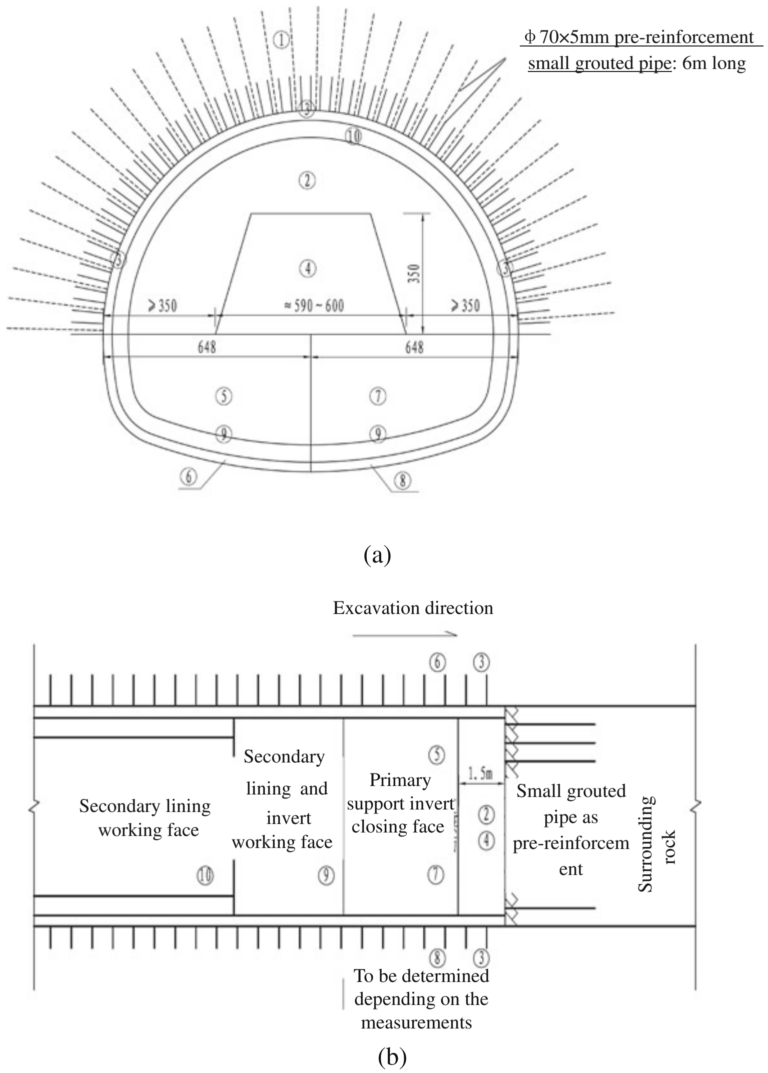
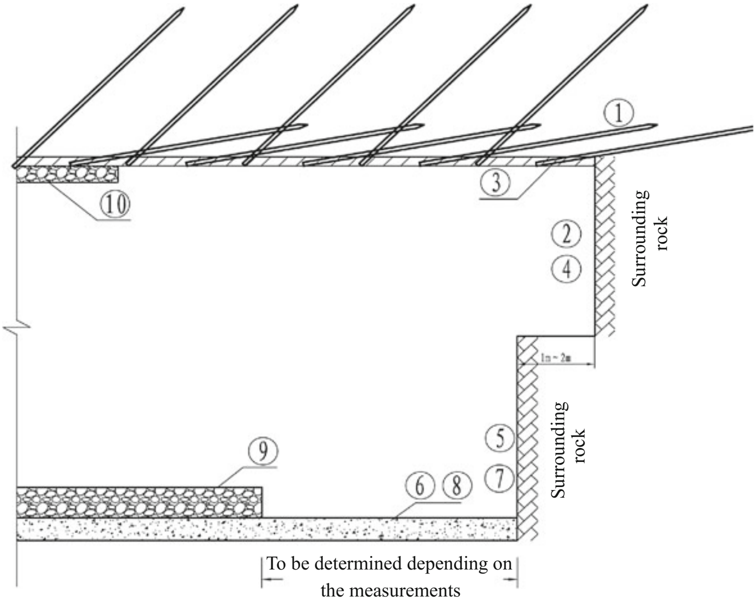


Fig. 4.8 Excavation and supporting sequences of highway tunnels

Please refer to Fig. 4.8 for the construction procedures:

① Small grouted pipe pre-reinforcement for the roof;
② Top heading excavation;
③ Top heading primary support;
④ Excavation of face buttress;
⑤ Left side bench and invert excavation;
⑥ Primary support for left sidewall and left invert lining;



(c)

Fig. 4.8 (continued)

⑦ Right side bench and invert excavation;
⑧ Primary support for right sidewall and right invert lining;
⑨ Casting and backfill of secondary lining for inverts;
⑩ Casting of secondary lining concrete.

To increase the rigidity of primary support, the primary support was strengthened as follows:

Pre-reinforcement of the roof: Φ70.5 mm double-row small grouted pipe pre-reinforcement was used (length: 6.0 m); the longitudinal space between the two rows was 1.5 m. The first row of small grouted pipes were installed toward the face with elevation angle of 35°, circular space of 0.6 m and longitudinal space of 3.0 m. The second row of small grouted pipes were installed with elevation angle of 5°–10°, circular space of 0.6 m and longitudinal space of 3.0 m. To strengthen the rigidity of the second row, three Φ22 reinforcing bars were inserted in each pipe.

For primary support, H200*200*8*12 section steel sets with longitudinal space of 0.5 m was installed; *C25* steel-fiber-reinforced shotcrete with thickness of 28 cm was used; E6 double-layer steel nets (15 × 15 cm) was used; no system rockbolt

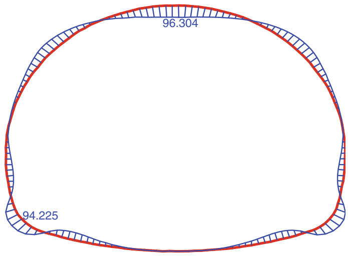


Fig. 4.9 Bending torque of primary support

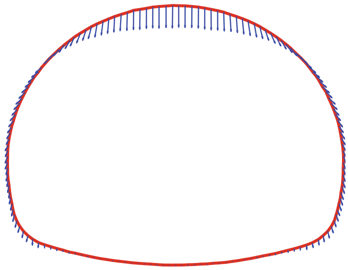


Fig. 4.10 Displacement of primary support

was provided within 100° of the roof; the grouted rockbolts of 4.0 m Φ25.5 were provided on sidewalls, at a circular space of 1.0 m.

For secondary lining, 50 cm thick formwork reinforced concrete was casted in place.

(1) Structural Strength of Primary Support

Before checking calculation of construction sequence, the commercial software Tongji GeoFBA was used to calculate the primary support strength of the tunnel in order to ensure permanent safety of the structure.

Given small cover depth and poor surrounding rock conditions, the surrounding rock basically has no self-bearing capacity and strong supporting is required. As secondary lining could not be provided in time, the primary support should be able to support all load after excavation.

The class VI surrounding rock parameters were used. The load mode of shallow-buried tunnel was used to calculate the impact of vehicle loading. The calculation results are given in Figs. 4.9 and 4.10.

The calculation results indicate that the maximum bending torque value is 96 kNm, observed at the roof crown. The maximum vertical displacement of the roof crown is 1.6 cm. The steel sets used was the H200*200*8*12 section steel, with longitudinal space of 0.5 m. According to the calculation, the stress requirements can be met.

Table 4.1 Strata parameters

Strata No.	Name	Elasticity modulus (MPa)	Unit weight (kN/m^3)	Cohesion force (kPa)	Internal friction angle (°)
1	Completely weathered granite	23.0	18.5	30	20
2	Severely weathered granite	150.0	21.0	50	25
3	Moderately weathered granite	2000.0	23.0	500	35
4	Grouted severely weathered granite	300.0	20.0	100	30

(2) **3D Numerical Simulation for the Construction Sequences Involving the Top Heading and Bench Method with Face Buttress**

(i) Introduction to finite element model, parameters and simulation conditions

The parameters (Table 4.1) used for the calculation were obtained from the drilling materials and regional experience. Due to unevenness of the strata soil, there were certain variation of parameters. However, the calculation results could roughly reflect the physical and mechanics properties of the section.

Since the distance between the two tunnels is big, excavation-induced mutual impact between the two tunnels was ignorable. It was suggested that the construction of one tunnel should not start until the other tunnel pass through the exiting expressway. Avoiding synchronous construction can minimize mutual impact between the two. In order to simplify the numerical model, the impact of only one tunnel on the expressway was simulated. Please see Figs. 4.11 and 4.12 for the model.

The top heading and bench method with face buttress was used. A passageway of at least 3.5 m wide was reserved on the each side of the buttress. The excavation sequence was, excavation of the top heading → reservation of face buttress (1.5 m

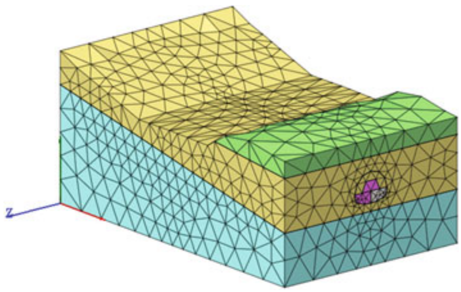


Fig. 4.11 3D finite element model

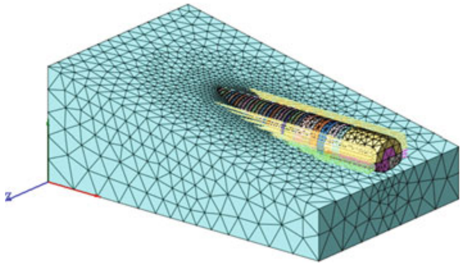


Fig. 4.12 Tunnel model

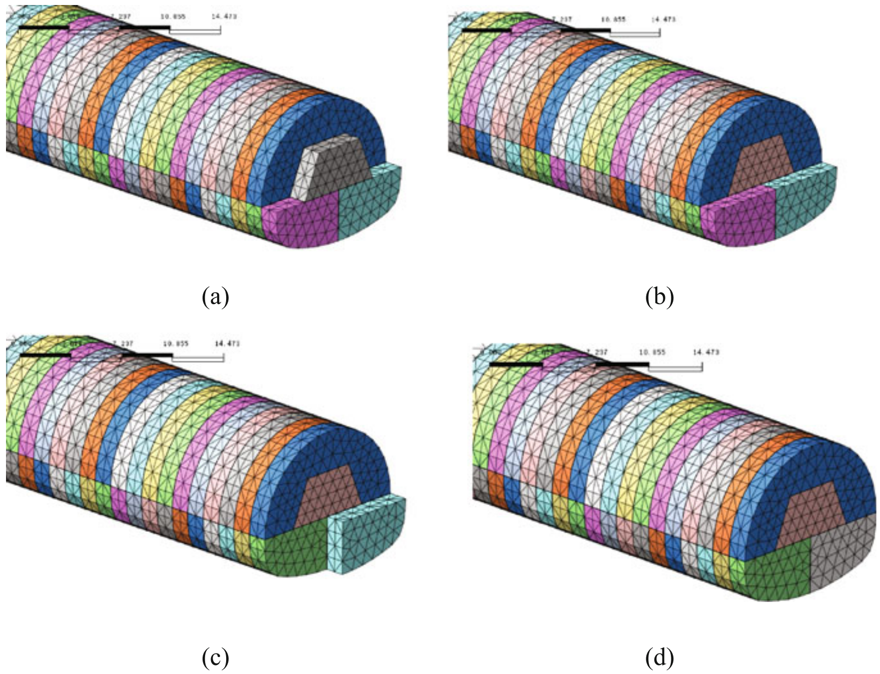


Fig. 4.13 Simulation of excavation sequences

long) → excavation of face buttress (1.5 m reserved between the buttress and sidewalls) → excavation of sidewalls. Finally, timely supporting was provided after excavation of each stage as shown in Fig. 4.13.

During excavation of top heading, only pre-reinforcement was provided for a section of 1.5 m ahead the tunnel face. Since primary support cannot be provided in time, the section was in an extremely dangerous status.

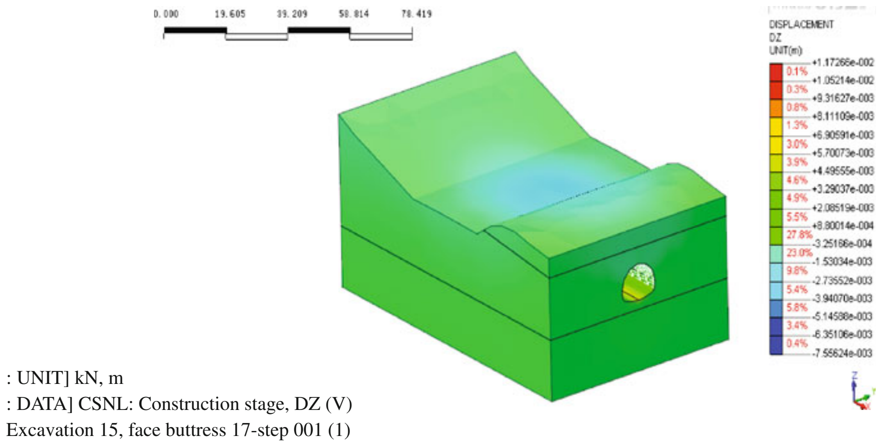


Fig. 4.14 Contour plot of vertical deformation

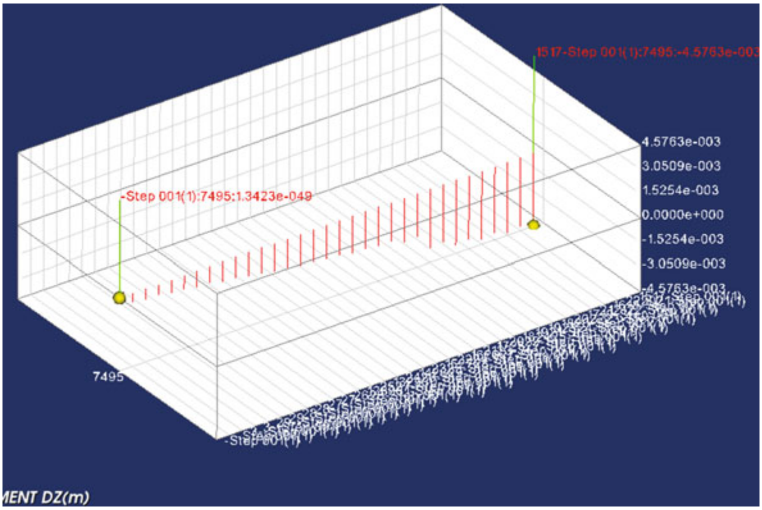


Fig. 4.15 Ground subsidence after excavation

(ii) Working condition simulation and results analysis
  a. Expressway subsidence

The double-row small grouted pipes were used as a means of reinforcement. The maximum vertical displacement would take place at the tunnel roof crown according to calculation, which would reach 7.6 mm. The subsidence of the expressway would be slightly smaller, of 4.5 mm as shown in Figs. 4.14 and 4.15. It indicates that, the vertical deformation of the surrounding rock can be effectively controlled with the provision of strong pre-reinforcement. Presence of the face

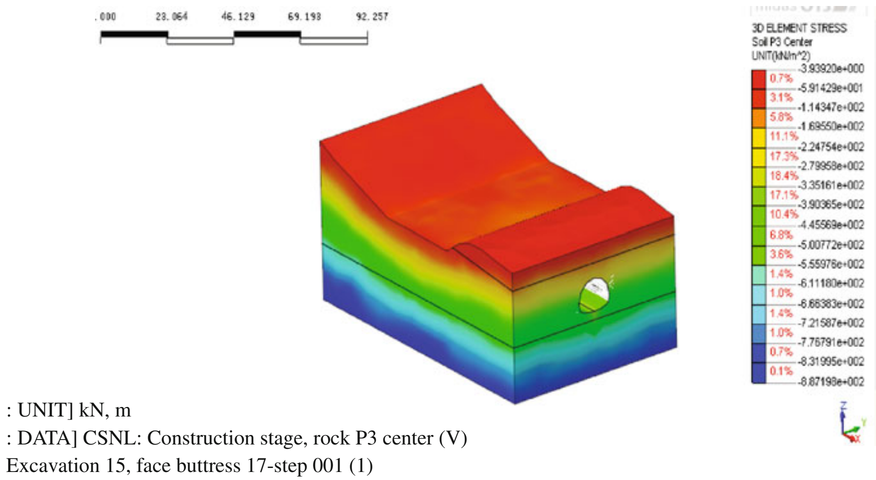


Fig. 4.16 Contour plot of major principal stress after excavation

buttress maintains the stability of the left and right sidewalls and the unsupported tunnel face. Therefore, pre-reinforcement with using double-row small grouted pipes is feasible.

b. Surrounding rock stress

According to Mohr-Coulomb yield criterion, no yield status is observed on the rock mass, which means the surrounding rock meets the design requirement as shown in Fig. 4.16.

(iii) Calculation summary

The following conclusions can be drawn based on the calculation results:

a. According to the calculation results using the load structural method, the initial structure of the tunnel meets the stress requirements;
b. According to the calculation results using the strata structural method, the maximum ground subsidence of Yongtaiwen Expressway is 4.5 mm. The influenced range of subsidence along the driving direction is 32 m. The impact on road surface structure and driving safety is ignorable;
c. During construction of a 1.5 m excavation step, the tunnel face below Yongtaiwen Expressway was stable due to provision of pre-reinforcement, pre-grouting and reservation of face buttress.

Due to limitation of the model itself and the selection of parameters for numerical simulation, the results are only very crude. However, the calculation results indicate that the mechanized excavation scheme and supporting measures are enough to guarantee safe operation of the expressway.

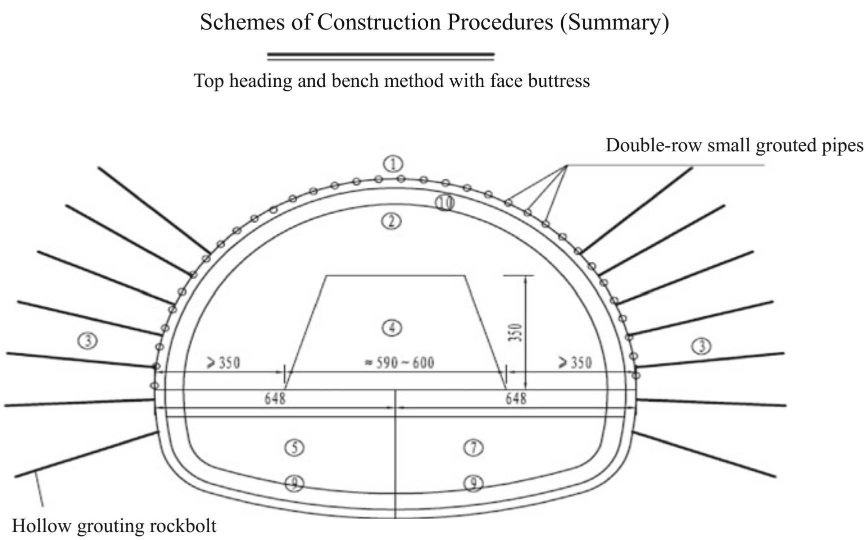


Fig. 4.17 Alternative scheme for excavation and supporting of the highway tunnel

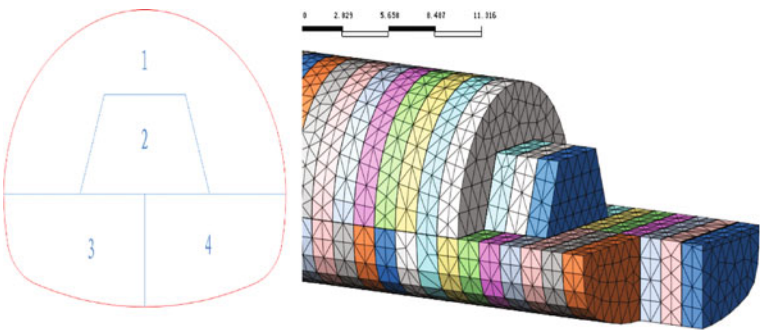


Fig. 4.18 Numerical simulation for alternative scheme of highway tunnel excavation method

4.4 Review of Alternative Scheme for Highway Tunnel Construction

In Figs. 4.17 and 4.18, the actual unlined section is 4.5 m, but the small grouted pipe for pre-reinforcement is only 6 m long which is not sufficient to guarantee the spatial stability of tunnel excavation and supporting. The calculation method shown in Fig. 4.18 implies the spatial stability of tunnel construction excavation and supporting, i.e., deformation compatibility control. In addition to measures shown in Fig. 4.18, other measures must be provide to guarantee the spatial stability, i.e., deformation compatibility control. Otherwise, the alternative scheme cannot meet the deformation compatibility control requirements. This must be kept in mind for actual design and calculation.

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Part III
Application of Stable Equilibrium Theory to Urban Underground Engineering Construction

Chapter 5
Application of Stable Equilibrium Theory to Metro Tunnels and River-Crossing Tunnels Using Shield Tunneling Method

Abstract The expansion of cities and increase of the population leads to heavy traffic which, in town, causes air pollution and traffic jam. These belong to the most important problems in nowadays urbanization. The metro tunnels and is considered as an efficient medium of public transportation and is widely used to alleviate traffic problems and environmental pollution. Since many developed cities have rivers crossed through, tunneling shields sometimes have to cross rivers. The tunneling inevitably caused ground subsidence and consequently influence on the buildings and river banks [1]. It is necessary to utilize the stable equilibrium theory and related techniques to control the ground subsidence and its negative impact within an allowed limit. This chapter illustrated the application of the theory with two such cases.

5.1 Deformation Compatibility Control Technique for Metro Shield Tunnel

1. **Introduction to the Project**

No. 7 and No. 8 shields were used for the Cheng-Hu double-line section of Line 1 metro in Hangzhou. The starting and ending mileages of the tunnel design is: right line K11 + 202.932 ~ K12 + 316.763, with an overall length of 1113.831 m; left line K11 + 202.932 ~ K12 + 316.763 with an overall length of 1113.831 m. The minimum radius on the plane is R = 450 m, and the maximum gradient on the longitudinal profile is 25 ‰. The buried depth of the tunnel roof is 11.2–17.5 m. Two Φ6340 mm earth pressure balance (EPB) shields (i.e., 7# and 8# shields) are used for construction.

The section is located in the central city. The section advances from east to west along the city avenues, passing below Anle Bridge, Zhonghe Overpass and Yongjin Interchange, as well as Chaiduo Bridge. The buildings and structures on the two

H. Zhu et al., *Stability Assessment for Underground Excavations and Key Construction Techniques*, Springer Tracts in Civil Engineering,
DOI 10.1007/978-981-10-3011-6_5

Fig. 5.1 Tunnel passing below Yongjin interchange viaduct

sides of the road are densely distributed. The traffic condition on the road is very busy. There are a lot of pile foundations below the road surface, with complicated engineering environments as shown in Fig. 5.1.

(1) Geological Condition

The geological structure and strata along the section are predominated by the silty soil and sandy soil area of estuary alluvial marine and colluvium sedimentation. The characteristics of soil differ due to difference in accumulation times and solidification conditions, ranging from looseness to medium denseness. The thickness is generally about 20 m, below which there are silty soft soil and clayey soil of interactive marine and terrestrial deposit. The round gravel layer of the ancient Qiantang River bed deposit is about 40–45 m below the ground in a medium dense—dense status. The underlying bed rock buried depth is about 55–63 m underground.

The shields in the section mainly pass through the following strata: ③$_6$ layer of silt embedded with sandy silt; ④$_2$ layer of muddy silty clay; ⑤ layer of silty clay; ⑦$_2$ silty clay layer; ⑧$_1$ silty clay layer; ⑧$_2$ silty clay with sand. In particular, the exit section of the Chengzhan Station falls within ③$_3$ and ③$_6$ silt layer embedded with sandy silt; the entrance section of the Hubin Station falls within ⑦$_2$ silty clay layer.

(2) Hydrogeological Conditions

Superficial layer ground water: the superficial part underground water along the metro line belongs to the phreatic water, and is retained in the upper ① layer of backfill soil and ③ layer of silt and silty sand; the supply sources mainly include atmospheric precipitation and runoff water. There is a mutual supply relation between the underground water and the pond water. The static water level is generally 0.85–2.4 m deep, which varies with seasons. The underground water is not corrosive to concrete structure, or to the rebars of the reinforced concrete soaked in water for a long time; it is moderately corrosive to steel structure.

Confined groundwater: the confined groundwater layer is mainly distributed in the ⑫ layer of fine sand and gravelly sand in the deeper portions. The aquitard layer includes the mud soil and clay layer on the upper part (④, ⑦, ⑧, ⑨, ⑩ and ⑪). The roof elevation is between 24.63 and 28.51 m.

2. **Control Measures for Ground Deformation during Construction**

To effectively control the ground deformation and minimize impact on surrounding environment and protect structures and buildings along the metro line, the following construction measures were taken according to the characteristics of shield construction:

(1) **Information-based Construction Principle was Adopted**

The soil chamber pressure was adjusted and the excavated earth volume was controlled according to the monitoring results of ground deformation.

According to the construction principle of EPB shield and strata property, after the soil cut using the cutter were brought into the soil chamber, its plastic flow property was improved so that the soil evenly fill the chamber and the spiral conveyor. The shield jack applied pressure on the soil and acted on the excavation surface, to maintain the water and soil pressure equilibrium and stability of the surface.

(2) **Strengthen the Segments Grouting Process**

The tail void grouting and secondary segment re-grouting were implemented strictly, with the quality of grout maintained.

Weak strata construction: the shield tail void grouting was adopted; the grout with satisfactory filling ability was used to avoid collapse of surrounding rock. If the best time is missed, there will be caved spaces. In this way, the original stability and stress condition of the strata in the original status will be changed. By injecting grout with certain strength and similar specific gravity as soil, the soil mass around the tunnel segments could be effectively supported, so that the complete filling and solidification requirements can be satisfied. The grout after solidified, acted as a hoop of the tunnel segments, which can improve the bearing capacity of the tunnel segments and control their discontinuous deformation. In this way, the stable equilibrium of soil-structure interaction was maintained and the soil deformation is controlled.

(3) **Keep Satisfactory Sealing Performance of the Shield Tail to Avoid Leakage**

The shield tail sealing measures that include three sealing brushes and the filled-in grease were used to resist the water and soil pressure and tail void grouting pressure outside the shield tail. Generally, the number of sealing brushes and grease chambers are determined depending on the pressure resistance of each grease chamber and the maximum expected soil and water pressure. The shield tail sealing should withstand the soil and water pressure and prevent loss of grout.

In addition, proper selection of segments, proper control of the shield posture, maintaining even shield tail gap and reasonable adjustment of the dosage of sealing grease should be guaranteed to prevent the grout from flowing into the sealing brush from the side with bigger void and solidifying there; because this will destroy the sealing performance of the tail.

(4) **The Shield Posture was Adjusted in Time to Maintain its Satisfactory Status and Avoid Excessive Disturbance to the Surrounding Soil Mass**

During advancing of the shield, the data measured by the shield automatic guidance system were used as the reference. The difference between the front and rear central points of the shield body and that between the shield central line and designed tunnel central line was kept to the minimum. The path line of the shield was kept coincided with the shield excavation boundary line as far as possible.

3. **Comparison of Control Effect of Ground Deformation**

For 7# shield, tail void grouting of inert grout was adopted. However, for 8# shield, the semi-rigid grout with certain strength was filled in. To compare the ground deformation of the two line, 5 sections were selected from the entire lines. Ground deformation of 7# and 8# shields is shown in Figs. 5.2 and 5.3 and Table 5.1.

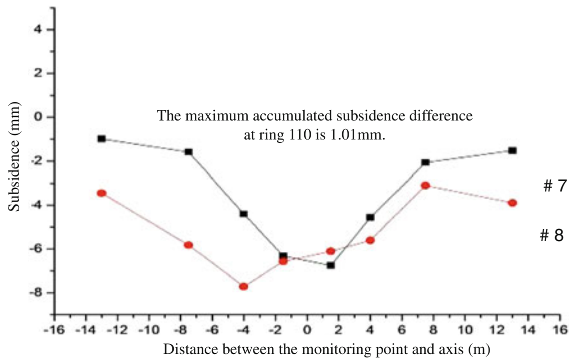


Fig. 5.2 Comparison of ground deformation at ring 110 for #7 and #8 shields

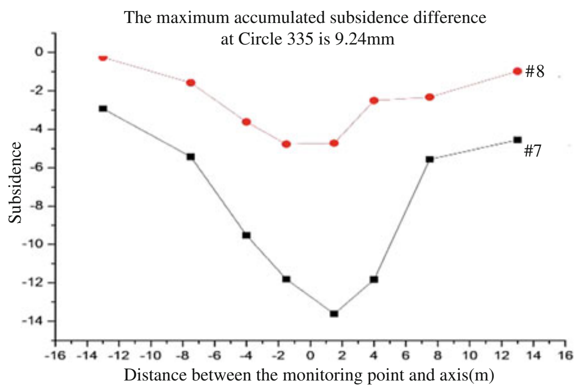


Fig. 5.3 Comparison of ground deformation at ring 335 for #7 and #8 shields

Fig. 5.4 Shield entry and re-filling of quick-hardening grout in the tunnel

Table 5.1 Comparison of maximum ground subsidence for 7# and 8# shields

Ring No.	Line	Maximum subsidence (mm)	Ring No.	Section line	Maximum subsidence (mm)
Ring 110	7# shield	−7.71	Ring 335	7# shield	−13.62
	8# shield	−6.75		8# shield	−4.87
Ring 485	7# shield	−25.83	Ring 610	7# shield	−41.41
	8# shield	−23.47		8# shield	−15.2
Ring 810	7# shield	−61.27			
	8# shield	−17.24			

According to the deformation at the 2 sections of the tunnel and another 3 sections, the deformation of 7# was larger than that of 8# shield.

4. **Comparison of Construction Parameters of 7# and 8# Shields**

Please see Table 5.2 for construction parameters of 7# and 8# shields.

5. **Impact of Abnormal Posture of 7# Shield on Tail Void Grout Amount and Shield Entry**

As shown in Fig. 5.5, the advance profile of the shield covered an area larger than the excavation area of the shield. When difference between the front and rear central lines of the shield reached 150 mm, the grout amount exceeded the theoretical amount by 0.876 m^3. Due to the large angle between the shield central line and excavation face, there was great disturbance to the soil mass during shield advancing, which was not good for controlling ground deformation. Due to the angle between the shield and excavation face, the soil mass imposed a remarkable friction force on the shield, and thus increased the shield thrust force. In particular, in the soil whose liquidity index is less than 0.25, the increase of shield thrust force was more obvious.

Table 5.2 Construction parameters for 7# and 8# shields

Item		Description
Construction parameters	7# shield	Soil pressure: 0.30–0.35 MPa; cutter torque: 60–80 %; thrust force: 2600–2900 t; the excavated earth volume exceeded the theoretical volume by far
	8# shield	Soil pressure: 0.22–0.28 MPa; there was sometimes virtual pressure during advancing after addition of foaming agent; total thrust force: 1500–2000 kN; cutter torque: 30–50 %, i.e., 1500–2500 kNm. The excavated earth volume was close to the theoretical volume
Segments assembly	7# shield	According to the designed line characteristics, the layout segment assembling method was adopted. The pasting rectification was required. There was frequent segment breakage and water leakage during advancing
	8# shield	According to the actual posture of the shield, the selected segments were assembled, which can properly control the tail void, and guarantee satisfactory assembly quality
Grout filling	7# shield	The grout amount was 6–7 m^3. Frequent grout leakage was observed during grouting. The ground subsidence was large as shown in Fig. 5.4
	8# shield	The shield grout amount was 2–3 m^3. The sealing performance of the shield tail was satisfactory without any grout leakage. Secondary injection of quick hardening grout (as shown in Fig. 5.5) was carried out for the segments at an interval of 5 rings, to effectively filing induced void and control ground subsidence

During the entry of shield, the difference between the verticality values of the front and rear points was 150 mm. The front point was closer to the center of the entry portal. After the shield reached the entry reinforced area, the shield thrust force reached about 3600 t while the thrust force of 8# shield was only 500 t.

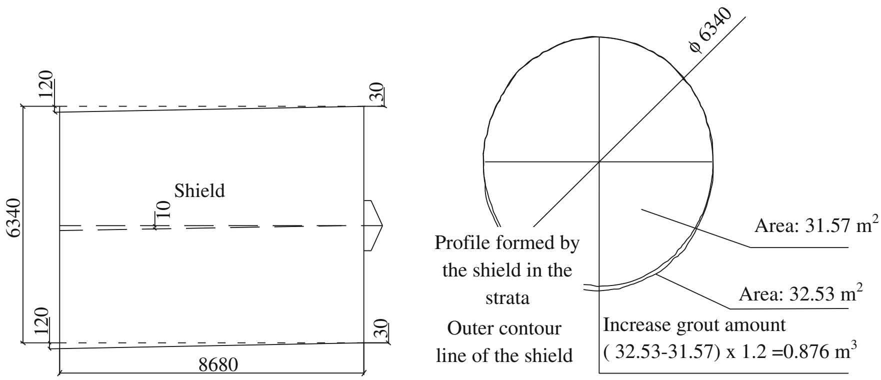
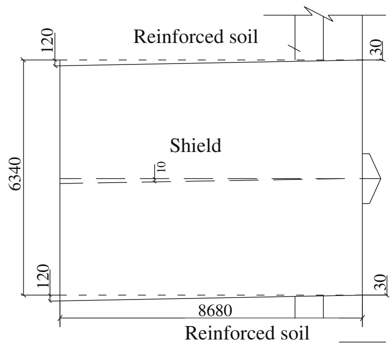


Fig. 5.5 Illustration of relation between real posture and design construction parameters for 7# shield (all dimensions in mm)



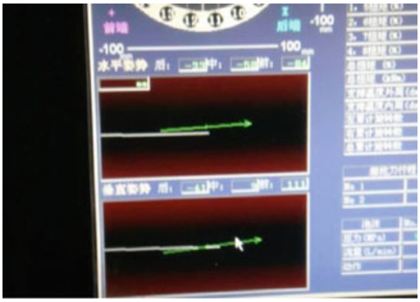


Fig. 5.6 Postures of 7# shield in the reinforced soil (all dimensions in mm)

The shield had no advance rate. After manual removal of pit supporting structure at the end, the shield still could not advance even with the maximum thrust force.

Due to the non-ideal shield posture, the periphery of the shield was subjected to constraint of reinforced soil and remarkable friction resistance, which stopped the shield from advancing as shown in Figs. 5.6 and 5.7.

5.2 Deformation Compatibility Control Technique for Qianjiang Tunnel Crossing Embankments of the Two Banks

5.2.1 Introduction to the Project

(1) Introduction to the Tunnel

Qianjiang Tunnel is 4.45 km long (mileage K11 + 400 ~ K15 + 850) shield tunnel, located 2.5 km (Fig. 5.8) upstream of Ninghai Yanguan Town. The circular tunnel section of Qianjiang Tunnel is 3242.783 m long for the west line and 3245 m long for the east line. As the river-crossing tunnel has a large diameter, long mileage and high technical difficulty and complicated geological conditions, Qianjiang Tunnel is regarded as the key part for Qianjiang Channel and its connecting lines.

The Qianjiang Tunnel includes 6 lanes (two lines), including west line and east line. For construction of the circular tunnel section, an ultra-large slurry shield with a diameter of 15.43 m was used. The construction sequences were as follows: exit from work pit on the south bank of the river (west line) → entry into work pit on the north bank of the river (west line) → turn round the shield in the work pit on the north bank of the river → exit from the work pit on the north bank (east line) → entry into work pit on the south bank (east line) → shield disassembling and

Fig. 5.7 Construction scene of the entry of 7# shield

removal. Therefore, the shield tunnel penetrates the embankments of both the south and north banks twice.

The single-layer segments were used for tunnel lining. The outer diameter, inner diameter, ring width, and duct piece thickness are 15,000, 13,700, 2000 and 650 mm respectively. The general wedge shaped segments were used. One ring is composed of ten segments, among which there were seven standard segments, two adjoining ones and 1 capping one. The ordinary ring segments were made of the reinforced concrete, with concrete strength of *C60*, and impermeability of 1.2 MPa. The stagger-jointed assembling process was used for the segment rings. A total of 38 *M30* longitudinal inclined bolts were used between two adjacent rings; two *M39*

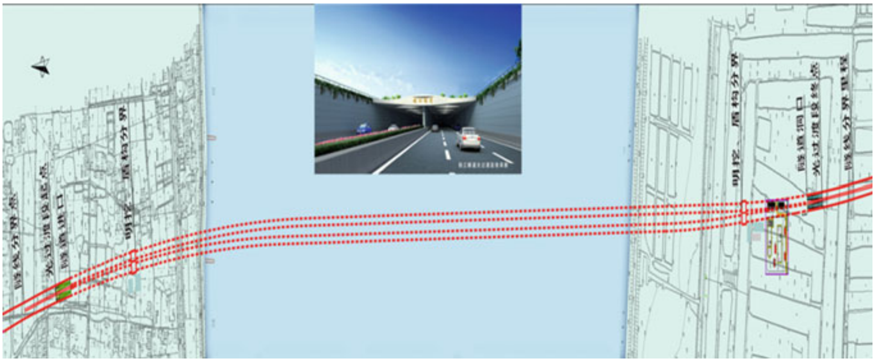


Fig. 5.8 Plan sketch of Qianjiang tunnel

circular inclined bolts were used radically between two adjacent segments. There were totally 20 bolts in each ring.

(2) **Geological Conditions**

The Qianjiang Tunnel passes through thick quaternary covering layer in the marine depositional plain of the Qiantang River [2]. The open cut section is located on the north side of the river, where tunnel penetrates mainly silty clay layer. The strata of the river bed are predominated by silty clay and clayey silt. The strata of open cut section on the south side of the river is predominated by muddy silty clay, silt and silty sand layer as shown in Fig. 5.9.

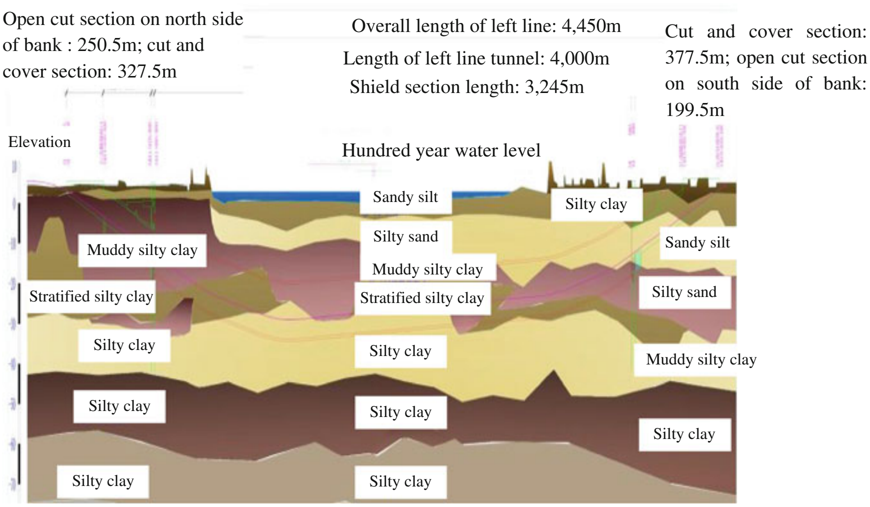


Fig. 5.9 Schematic geological profile for Qianjiang tunnel

Fig. 5.10 Current situation of Haining Embankment on the north bank

(3) **Structure of North Embankment**

The north embankment was made of stone in the period of Ming and Qing Dynasties. During 1997–2003, it was reconstructed into standard embankment. The embankment is 5 m high, with timber piles under it. On the outer side of the embankment, there is two-stage stone slope protection and wood piles rows for foot protection. Because the stone embankment is basically intact, the standard embankment is basically constructed on the original one. After completion of the embankment, the soil ridge top elevation is 8.87 m and the width is 4 m. On the outer side, there is a masonry wave protection wall, with wall top elevation of 9.67 m. This soil ridge inner slope is a 1:2.5 earth slope with plantation protection. The inner embankment top is about 11 m wide, with an elevation between 6.88 and 7.37 m. A flood protection road of 4 m wide as shown in Fig. 5.10 has been built close to the inner side. The shield tunnel passed below the wood piles of the stone embankment.

5.2.2 *Impact of Shield Construction on the Ancient Embankment*

According to the related analysis results presented in evaluation reports on flood protection for Qianjiang Tunnel, the subsidence of earth and stone embankment top on the north bank of the Qiantang River using Peck formula are given in Figs. 5.11 and 5.12.

The FEM (finite element method) software Plaxis 2D is used to build up a numerical model to analyze impact of shield tunnel construction on the above places. The finite element calculation model for the ancient seawall embankment on

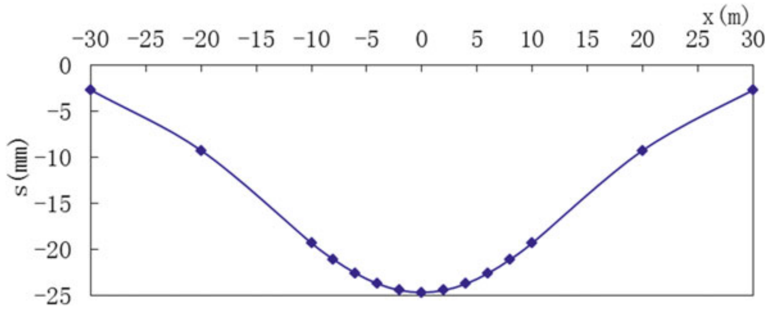


Fig. 5.11 Horizontal subsidence of soil ridge embankment top on the north bank of the Qiantang river

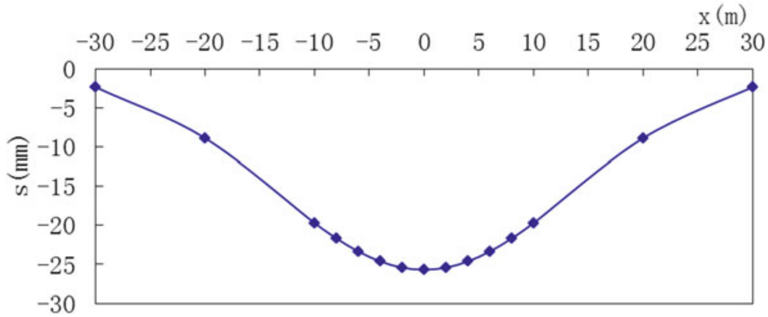


Fig. 5.12 Horizontal subsidence of the stone embankment constructed in Ming and Qing dynasties

the north side is shown in Fig. 5.13. The 15-node unit is taken as the basic unit. There are 760 units and 6365 nodes.

Considering the tunnel excavation sequence, the contour of the subsidence caused by excavation of west line and east line tunnel is shown in Figs. 5.14

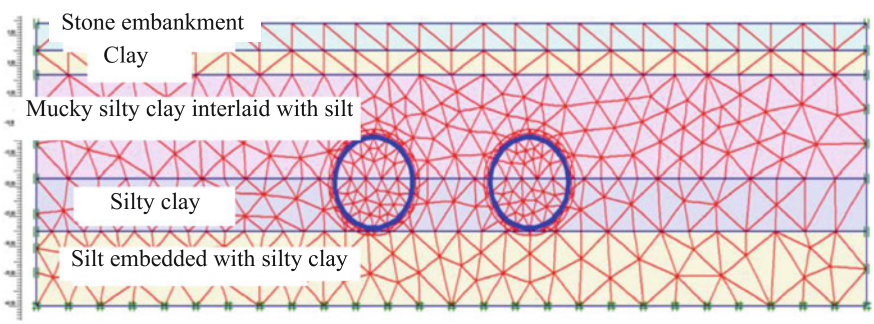


Fig. 5.13 Elements of shield tunnel simulation model (stone embankment built in Ming and Qing dynasties)

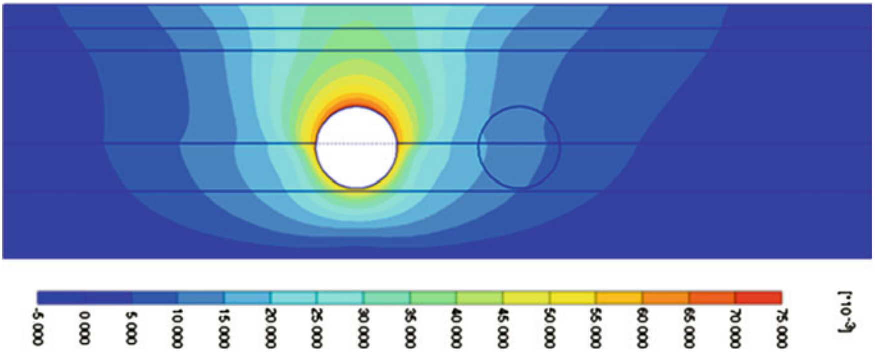


Fig. 5.14 Displacement contour of after excavation of west line

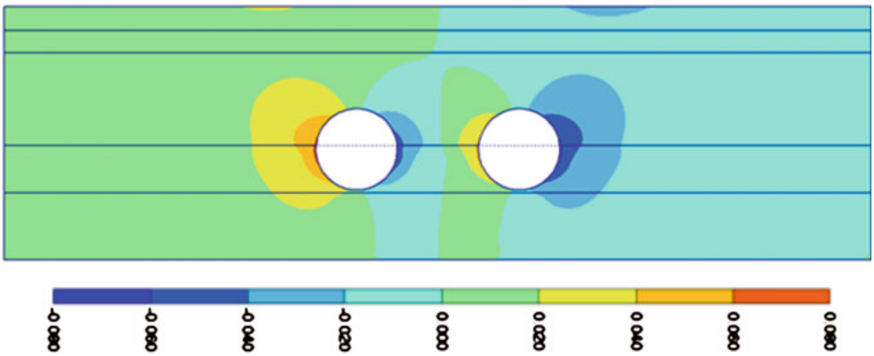


Fig. 5.15 Total displacement of the model after excavation of both lines

and 5.15 respectively. Excavation of the west line tunnel results in subsidence trough on the ground above the excavation. The maximum subsidence was observed right above the tunnel axis, while it tended to reduce towards the left and right sides. The maximum subsidence was 33.29 mm, which was 7.61 mm higher than the results obtained using the empirical formula. After excavation of the two tunnels, there was also a subsidence trough formed on the ground above the excavation. The maximum subsidence was observed right above the second tunnel axis. The subsidence tended to reduce towards the left and right sides. Due to the mutual impact between excavations of two lines, the maximum ground subsidence reached 49.50 mm.

According to the analysis in the evaluation reports on flood protection, after excavation of the two tunnels, there would be mutual impact between the west and east line tunnels. The maximum subsidence of the soil ridge embankment top of the embankment on the north bank of the Qiantang River would be 47.14 mm; the maximum subsidence on the surface of the ancient stone embankment on the north bank would be 49.50 mm; the maximum subsidence of the ground surface at the spur dike outside the embankment on the north bank would be 53.39 mm. Based on the above information, the monitoring indexes for dike subsidence were proposed as follows. The uneven subsidence slope control value of the north bank is 0.1 % and the maximum subsidence of the dike is 20 mm.

According to the above analysis, and based on comprehensive consideration of shield tunnel stability and equilibrium and deformation compatibility control, the environmental control of shield tunnel construction is classified as follows.

The first-class control section was the section where the tunnel passes the ancient stone embankment, with the maximum subsidence controlled within 10 mm; the second-class control section included the soil embankment sections on the south and north banks, with the maximum ground subsidence controlled within 50 mm; the third-class control section included the river bed section of the shield, focusing on guaranteeing the stable equilibrium of the shield tunnel. Therefore, the parameters for shied tunnel stable equilibrium construction and construction parameters for auxiliary measures such as shield grouting were determined according to the above classification. For example, the tail void grout amount for the section that pass through river bed should be 22 m^3 (filling ratio of 1.07 %) for each ring. However, the tail void grout amount for the section that pass below the stone embankment on the north bank should be 24–29 m^3 (filling ratio of 120–140 %) for each ring under first-class control. The pre-grouting function of the shield head was activated, to fill and reinforce the upper strata of the shield in advance.

5.2.3 *Construction Controlling Measures for Shield Tunneling with Stable Equilibrium Impact Considered*

As the major difficulty of the construction was the stability of the stone embankment, related measures to be taken for the shield construction are described in details.

(1) **Technical Preparation before Penetration**

① Before advancing of the shield, the shield was inspected and maintained according to the shield maintenance manual provided by the shield manufacturer, in order to ensure the shield remained in a good status;
② Before penetrating the embankment, the impact of various construction parameters during the trial advancing period on ground subsidence were summarized; technical preparation was made for the shield to pass through the embankment based on the experience on the west line tunnel;
③ In order to provide reference for the shield to pass beneath the embankment, the embankment should be monitored to get hold of its natural subsidence before the shield exits the pit;
④ The ship thrown stone block mixture was used as the suppression platform for the north embankment one to two months in advance.

(2) Advancing Measures of the Shield during Penetration

According to the cover soil condition of the tunnel at the embankment and the impact of shield construction on ground structures, the part 30 m (15 rings) before the shield reached the embankment and 20 m after the shield tail passed (10 rings) were regarded as the major control sections for penetration of the shield into the embankment. Meanwhile, various construction parameters for the construction section were specified strictly. Cut water pressure, shield advancing rate, mud and water control, tail void grouting and sealing grease pressure injection were carried out strictly in accordance with the requirements.

(i) Cut water pressure

If the cut water pressure fluctuates dramatically, the disturbance to the soil mass in front of the cutting face will be increased, resulting in loss of soil mass; therefore, the pressure fluctuation should be minimized. During construction, fluctuation were controlled between −0.02 and +0.02 kg/cm^2 by adjusting the bubble chamber pressure and mud and water level the cut water pressure, to guarantee the stability of the tunnel face. In addition, the pressure of the cut face was properly increased to allow for certain upheaval of the stone embankment on the north bank in advance, which can make up later subsidence.

(ii) Mud and water quality index

High-quality mud and water was delivered to the cut face during penetrating the embankment, so as to properly support the soil mass in front of the cut face. Generally, the mud and water density was controlled between 1.2 and 1.3 g/cm^3 and the viscosity was controlled above 20 s.

(iii) Advancing speed and deviation reflection control

The advancing speed in this stage should not be too high, and was generally controlled below 20 mm/min, which was slower than the regular speed 50 mm/min. Even speed advancing was used to fully release the stress generated by advancing of the shield in the soil mass, and avoid generation of excessive advancing force or concentration of advancing force which may cause internal system failure of the shield. It can also facilitate deviation rectification of the shield. Furthermore, considering there may be unknown obstructions below the embankment, close attention was paid to the variation of the cutter torque. The shield advancing axis was controlled to avoid excessive or frequent shield deviation rectification. In this way, disturbance to the soil layer was reduced and ground subsidence was controlled.

(iv) Enhance management of tail void grouting

Tail void grouting aims at preventing ground subsidence. Tail void grouting control includes the grout amount and grout pressure control. During shield advancing, the grout amount was taken as the control index. The grout amount of the section was 120–140 % of the tail voids, i.e., the total grout amount was 24.0–29 m^3. During penetration, if excess embankment subsidence was observed, the emergent grouting treatment on the ground was initiated.

(v) Segment re-grouting

Secondary grouting was carried out via the segments grouting holes, to properly control the grouting pressure and grout amount and thus reduce later stage subsidence of the embankment.

(vi) Pay close attention to grout leakage at the shield tail and fill in sufficient tail grease;
(vii) Ensure that the grouting quality and avoid leakage from the grouting hole.

(3) **Information-based Construction**

Monitoring and measurement were carried out for both the embankment and shield machine during construction to facilitate information-based construction.

(i) Embankment monitoring

The interval of longitudinal monitoring points along the tunnel axis was reduced to 3 m. A total of 7 cross-sections were provided on the embankment. One distribution point was located right above the advancing axis center, and 8 points each were distributed within 32 m range on the left and right of the point. The monitoring point interval was 3 m within 20 m from the central line and 6 m within 20–32 m from the central line.

In the section from where the tunnel face was 30 m from the protection scope of the embankment (the embankment protection scope includes the embankment and 30 m extension from the embankment slope toe) to 20 m beyond where the shield tail exited the protection scope, the monitoring frequency of the subsidence was adjusted to 1 time/day. The frequency was gradually reduced after stabilization of the subsidence.

In this section, the monitoring frequency for the embankment was 1 time per ring. The monitored results were fed back to the construction site. If the actually measured differential subsidence or subsidence rate was excessive, the measuring points and frequency needed to be increased.

The warning and early warning criteria of monitoring subsidence variation were set as ± 10 and ± 8 mm respectively. If the accumulated variation exceeded 60 % of the warning value, the daily variation warning value was reset to ± 2 mm.

The measurements were reported and fed back in time so that the new technical parameters such as construction parameters and grout amount can be determined, and the subsidence controlling performance were eventually reflected through monitoring. Then repeating, verifying and improving the results can ensure safety of the embankment and tunnel construction quality.

5.2.4 Construction Practice Results

In April 2011, the west line tunnel passed through the north bank stone embankment successfully for the first time. The maximum subsidence observed during construction was +20 mm as shown in Fig. 5.16. The east line tunnel penetrated the north bank stone embankment for the second time in May 2012. The maximum total subsidence was −9.27 mm. The subsidence variation rate at the point was 0.5 mm/15d = 0.033 mm/d, indicating stabilization.

The Qianjiang Tunnel have features like strong bore tidal, shallow cover and large diameter. Its design and construction fully reflect the concept of shield stable equilibrium and deformation compatibility control. The specific technical advantages are shown as below.

(i) The design stable equilibrium status of the current structures including foundation pits and segments was high, which meets the stress requirements of the structural elements and effectively prevents localized failure. It was recommended that an expansion joint be provided on the elephant foot of carriageway plate and bottom filling concrete at an interval of 10 m, so as to accommodate the uneven longitudinal subsidence of the duct piece.

(ii) Injection of hard grout was used during construction to ensure the overall stable equilibrium and deformation compatibility control of each stress elements of the shield.

(iii) The combination of manual and automatic measurement and deviation rectification system was adopted during construction, so as to effectively control the strata deformation and regulate structure system along the design path.

During shield construction, the low self-stabilizing capacity of soft soil and consolidation and creep deformation of the soft soil causes equilibrium status transformation and deformation compatibility control problem under the strata-structure interaction, due to the temporary space excavated by the shield. In such a

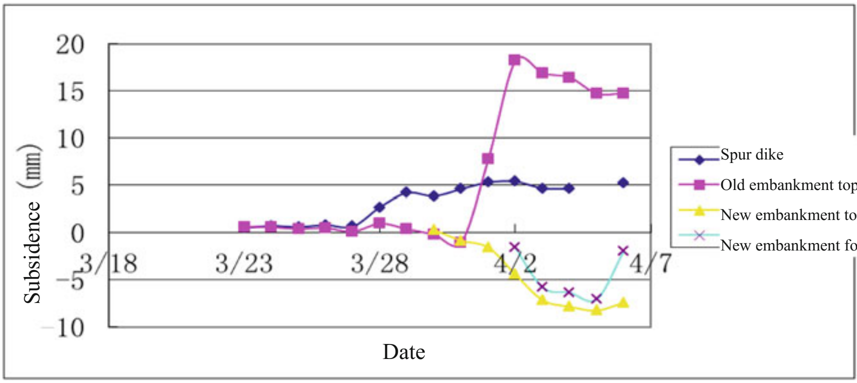


Fig. 5.16 Subsidence during shield advancing in north embankment

case, there will be certain uneven subsidence. The technical advantage (ii) and (iii) can basically maintain the original status of the strata, and thus ensure that the strata-structure interaction remained in the stable equilibrium status.

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Chapter 6
Deformation Compatibility Control Technique for Foundation Pits

Abstract Foundation pits also involve large amount of underground excavations, which are somehow similar to the tunneling sequences. Foundation pits instability not only poses dangers to the projects themselves but also puts the surrounding buildings or facilities at risk. This chapter introduced a few cases of instable pits, discussed possible causes of their failures and proposes feasible measures to achieve stable equilibrium and the deformation compatibility control status.

6.1 Typical Cases of Foundation Pit Instability

Basic phenomena discovered through statistical investigation of foundation pit are as follows. If the excavation sequences and supporting structures of underground foundation pit are reasonable, the interaction between the surrounding rock and supporting structure can reach the stable equilibrium and deformation compatibility control status, and engineering construction will go on smoothly with satisfactory safety quality. Otherwise, there will be potential engineering hazard or even failure. As shown in Fig. 6.1a, the building collapsed due to instability of the retaining structure of the underground garage under construction, the lateral force of the disposed soil, and low bending resistance of PHC piles. The excavation sequences and supporting structure of the foundation pit under construction as shown in Fig. 6.1b were reasonable, and thus stability and safety of nearby buildings was guaranteed.

1. **Insufficient Supporting Capacity**

The foundation pit for a certain jacking project was located on the edge of an existing railway line. The depth of the pit was 5–6 m. Φ800 mm bored piles were used as the retaining structure. The cement-soil-mixing piles were used between bored piles to prevent water seepage. As space was required for prefabrication of

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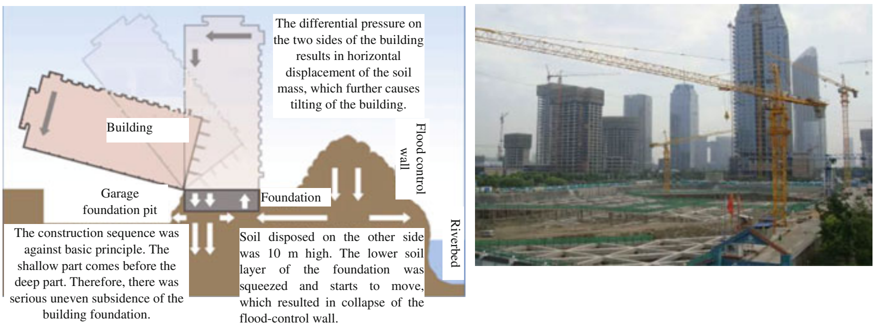


(a) collapse scheme of a building (b) nearby buildings near reasonable excavation and supporting

Fig. 6.1 Building collapse caused by bad excavation of foundation pit and stability of nearby building guaranteed by reasonable excavation and supporting

the jacking box culvert, horizontal inner support was not provided for the retaining structure which belongs to a cantilever retaining structure. Only periphery beam was provided on the pile top as shown in Fig. 6.2. The surrounding soil of the foundation from top to bottom were backfilled soil, mud, silty clay and silty clay embedded with sand and gravel.

During prefabrication of the box culvert in the foundation pit, the bored piles deformed too much towards the foundation pit, which resulted in failure of the periphery beam. The bored piles tilted and collapsed towards the foundation pit as shown in Fig. 6.3.

Through analysis of the accident cause, only the part of the bored pile in the mud layer was used to balance the water and soil pressure. No supporting or anchoring

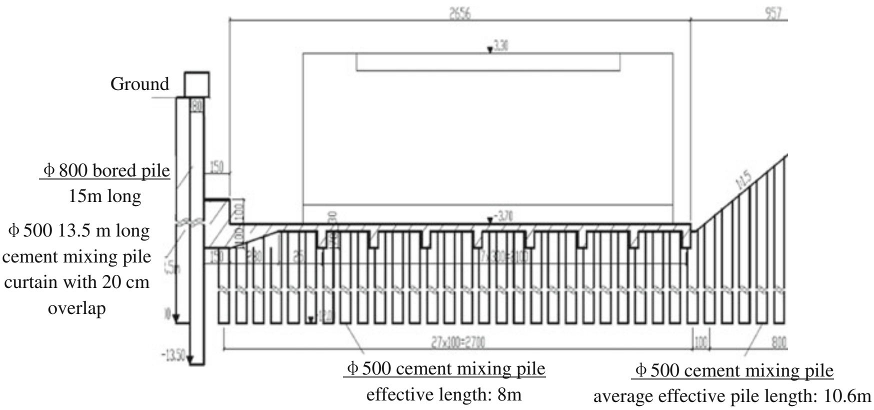


Fig. 6.2 Design diagram of foundation retaining structure (all dimensions in cm except elevation)

Fig. 6.3 Failure of foundation pit periphery beam and bored piles

structure was provided. The deformation compatibility capacity provided by the periphery beam was not sufficient, which resulted in failure of the foundation pit. For such foundation pits, anchoring structure should be provided outside the periphery beam.

2. **Retaining Structure Instability**

The instability of a certain foundation pit retaining structure resulted in large-area collapse: The collapsed pit was about 75 m long, 20 m wide and 16 m deep as shown in Fig. 6.4. According to initial identification of the accident causes, the main causes include: (i) pre-reinforcement was not provided or not provided properly for the pit base, which resulted in sudden piping or extruding deformation, as well as instability of the equilibrium status at the foundation pit bottom as shown in Figs. 6.4 and 6.7; (ii) the plane support system with horizontal steel pipe supporting would develop abrupt change of stress status due to pit excavation, which

Fig. 6.4 Failure scene of deep foundation pit and support system

Fig. 6.5 Support system before and after the Failure

belongs to bifurcation instability. This further caused overall instability failure (for plane supporting, flexible connection should be provided to prevent the steel pipe from falling, which may hurt people) as shown in Fig. 6.4. Figure 6.5 was not the spatial support system (non-equilibrium torque and collapsed supporting), so it cannot enable the foundation pit to “basically maintain the original status of surrounding rock” and enable the interaction between surrounding rock and support system to reach the stable equilibrium and deformation compatibility control status. The two problems jointly caused collapse of foundation pit. (iii) The overall excavation of the foundation pit was not good for maintaining the stable equilibrium and deformation compatibility control as shown in Fig. 6.5; if the construction period allows, the bench excavation method might be used with the bottom reinforced stage by stage as shown in Fig. 6.6, which can basically maintain stability of the pit and prevent collapse. (iv) The large-area collapse of the foundation pit and

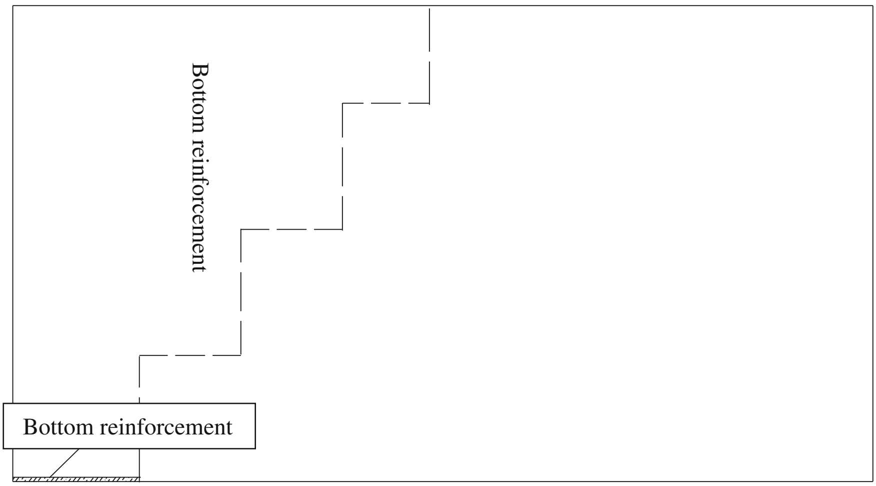


Fig. 6.6 Excavation using bench excavation method with bottoms reinforced stage by stage

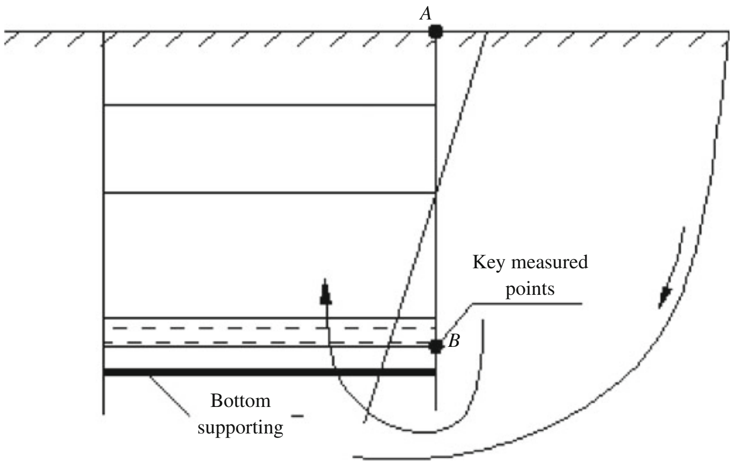


Fig. 6.7 Schematic mechanism of deep foundation pit instability

the support system failure are shown in Fig. 6.4. The foundation pit sidewall instability was observed as shown in Fig. 6.5. The main cause was the loss of stability of the interaction between the rock/soil and support system. The construction section is predominated by soft soil, in which the support system may easily rotate and lose its stability, with the rotating point located at the bottom. Therefore, point B instead of point A, was the key point for measurement as shown in Fig. 6.7. However, the monitoring results obtained at point A were used to guide construction, so the effect of monitoring feedback was not good. After deformation and cracking of the surrounding ground occurred (maximum displacement was 30 cm, lasting for about 40 days), if enough bottom support or even one bottom support was provided in time as shown in Fig. 6.7, the stable equilibrium of the support system would be improved. Even if there was sudden small-scale surging, sliding of soil mass on the two sidewalls which caused large-area collapse would not occurred. The collapse of the deep foundation pit basically resulted from the failure to control bottom sudden surging and supporting structure stability.

The experts that analyzed the deep foundation pit accident proposed 3 principles to be followed for engineering construction of the deep foundation pit. (i) Excavation of the foundation pit should be carried out layer by layer and block by block. The up-supported exposure time should be controlled within a certain limit. The thickness of one excavation layer should be controlled within 3 m, while size of each block should be controlled between 15 and 20 m. (ii) Supporting should come before excavation; attention should be paid to details; deformation of the pit should be controlled. (iii) Timely drainage is very important on rainy days; after completion, the concrete should be reinforced in time to ensure the foundation pit does not deform. The three principles basically meet the stable equilibrium and deformation compatibility control requirements for the interaction between the foundation pit and support system.

6.2 Deformation Compatibility Control Technique for Ultra-Deep Foundation Pit of Qianjiang Tunnel

The dimensions of the test shaft of Qianjiang Tunnel were 45.80 × 23.40 m (length × width), with the maximum excavation depth of 28.25 m. The test shaft would be used as the starting pit of the shield.

From top to bottom, the main strata discovered by prospecting and drilling includes plain fill, sandy silt, silty clay, sandy silt, silty sand, and muddy (silty) clay layer; phreatic pore water and confined pore water are closely related to the tunnel engineering construction. Please see Figs. 6.8 and 6.9 for photos of the construction process.

Fig. 6.8 Photos taken during construction of working shaft of Qianjiang Tunnel

Fig. 6.9 Internal view of working shaft of Qianjiang Tunnel

1. **Technical Requirements for the Foundation Pit**

(1) The open cut excavation and bottom-up construction method was used for the foundation pit; before construction of the retaining structure, the underground pipelines within the influenced area of the pit construction should be checked and handled properly.
(2) The vertical construction error of underground diaphragm wall should be no more than 1/200, and should be no more than 1/300 within the depth scope of the foundation pit.
(3) After removing molds of the cast-in-place guide wall of the underground diaphragm wall, the upper and lower supporting should be provided on the longitudinal direction to support the two guide walls. Before the concrete of the walls reached the design strength, no passage of heavy-duty machinery and transportation equipment near the wall was allowed, so as to avoid deformation of the wall. A row of Φ700 double-shaft cement-mixing piles should be used as a means of reinforcement behind the guide wall.
(4) The section-skipping construction method should be used for the underground diaphragm wall. Construction of the next slot section can be started after the strength of the previous section concrete reached 70 % or more of the designed strength.
(5) Upon completion of each excavation section, other matters such as dregs at the bottom should be removed. During bottom clearing and fabrication of rebars, the main reinforcement should be connected by welding with joints staggered. The number of joints within 500 mm and 35 d (d is the diameter of rebar) should be no more than 50 % of the rebars. The main rebar should be connected with other reinforcement by spot welding. The stirrup end should be made into a hook of no less than 135°, and the length of the straight section should be no less than 10 d. During hoisting of the reinforcement cage, hoisting section by Sect. 2.2 (sections at the most) was allowed based on the hoisting capacity. The section division should be selected from the parts with low internal force and few reinforcement bars, so as to reduce times of the reinforcement cage abutting and eliminate any potential quality and safety hazard. During hoisting of the reinforcement cage of the diaphragm wall, lateral oscillation of the reinforcement cage was not allowed, as it may cause wall to collapse. After installation of the reinforcement cage into the section, whether the height of the top met the design requirements was checked. Then the cage was fixed onto the guide wall using the U-steel. The purlin embedded rebars should be adjusted for the guide pipe location to avoid mutual impact.
(6) The quality of underwater concrete should be taken into full consideration to guarantee the grading and strength during casting. Commercial concrete should be used in this case. Continuous casting was required without interruption, so that the evenness and integrity of the concrete can be guaranteed. The concrete strength at the design elevation of underground diaphragm wall top must meet the design requirements. No scum was allowed at the design

elevation. Before casting periphery beam, the scum on the top and concrete exceeding the elevation should be removed by chiseling.

(7) Support structures should be installed as soon as excavation is carried out to avoid any excessive deformation of the retaining structure caused by non-timely supporting. Excavation of the foundation pit must be stopped after reaching the top of each support structure, and then slot cutting and support installation must be carried out in time. Pre-tension should be applied on the steel support elements as specified, to ensure that the deformation of the retaining structure was kept within the designed allowable scope. After installation of the steel supporting, stability of the support elements should be checked; construction can be restored after safety confirmation.

(8) Steel support elements must be fabricated and installed carefully in accordance with the design requirements. During installation of the supporting ends, the bearing plate should be perpendicular to the axis of support elements so that loads can act onto the support elements axially and supporting instability can be avoided. As the most important part of the foundation pit construction, the support system must strictly meet the design requirements. Effective measures should be taken during construction to ensure that pre-applied axial force may be re-applied when the axial force decreased.

(9) During foundation pit excavation and construction period, it was not allowed to damage the support system with the construction machines. The support system supports no other load that the axial force to avoid instability caused by overload of the support system.

(10) After completion of temporary stand column, the cavity drilling part above the stand column pile top (base) should be filled using sand and gravel.

(11) After excavation of the foundation pit, field engineers checked the exposed surface of underground diaphragm wall if it met the related regulations and codes [1]. Any leakage of the retaining structure was blocked before construction of the foundation pit main body structure. Construction of the main structure of the pit can only be started when there was no water leakage or permeation of the retaining structure.

(12) To control the subsidence of the underground diaphragm wall, two grout injection pipes were installed 0.5 m below each underground diaphragm wall. The grout injection scope should be 1.2 m wide and 1.0 m deep below the underground diaphragm wall. The grouting pressure is controlled as 1.0–1.5 MPa, with grout amount of 40–50 % and water cement ratio of 1:06–1:1. The grout material should be cement slurry or cement-based grout, with replacement ratio no less than 40 %. The grouting parameters should be determined through tests.

(13) During construction, the overload around the pit should be no more than 20 kPa; fences were provided around the pit to guarantee personal safety.

(14) During excavation of the pit, water leakage and permeation between underground diaphragm walls should be blocked to prevent large-amount loss of underground water and damage to safety of surrounding buildings/structures.

(15) During construction, the construction site should be checked based on the geological investigation report. In case of any non-conformance, field engineers should inform the project supervisors and design companies immediately so that they can carry out onsite adjustment and treatment to meet the design requirements.
(16) One time excavation to the bottom was allowed for the foundation pit. After the excavation reached 300 mm above the pit bottom elevation, acceptance inspection was required. Manual excavation was required to reach the base. Bottom reinforcement should be carried out in time to minimize disturbance to base foundation soil. Blind drains should be provided on the base to improve diversion and drainage of underground water during construction, so as to prevent the foundation soil soaked underwater.
(17) Two grouting pipes must be provided for grouting under the bottom of each stand column bored pile. It was required that the differential subsidence between the underground diaphragm wall and the bored pile was no more than 10 mm.
(18) Water-free operation design was required for the foundation pit. Before excavation of the foundation pit, underground water level should be checked to ensure it met the design requirements. If there is any water leakage or permeation, excavation should be stopped and the supervision and design organizations should be informed.
(19) The following instructions are important for foundation pit drainage. Catchwater should be provided on the top of the pit; the ground at the slope protection should be slightly higher than the outer ground; the cracks on the ground surface should be blocked timely; Water accumulated at low area should be drained to prevent the surface water from seeping into the pit and scour the side slope; drainage ditches should be provided for the slope toe to drain the seeped water in time. Open ditch drainage was adopted in the pit; standby dewater well should also be provided in the pit. Drainage ditch and water-collecting well should be provided in the pit to drain the ponding and rain water in the pit. Setup of the water-collecting well should be determined according to the section and the water yield. Sufficient water drawing equipment should be provided for construction in rainy season, so as to drain the rain water in time.
(20) Excavation of the foundation pit can only be started after strata (including working shaft and follow-up section) were reinforced according to the design requirements. It was recommended that the hole opening reinforcement outside the starting work shaft should be finished and the design strength should be reached before excavation. Otherwise, temporary # shaped frame that was made of reinforcement concrete should be provided within the hole opening scope as a means of temporary supporting; the concrete # shaped frame should be anchored onto the W1 side wall.
(21) The foundation pit is located beside the Qiantang River, with fishponds and rescue rivers nearby. During construction, power generators with sufficient capacity should be provided during construction, so as to guarantee power

supply to the drainage system and protect the site against flooding in case of power failure and rainfall.

(22) The 28 d unconfined compression strength standard value of the high pressure jet grouting pile should be no less than 1.0 MPa, with the replacement ratio no less than 20 %. The verticality deviation of pile body should be no more than 1/200. The pile location deviation should be no more than 50 mm. The allowable deviation of the pile diameter is ±10 mm. The allowable deviation of pile bottom deviation is +100 and −50 mm.

(23) Excavation of foundation pit, construction of the bored pile and retaining structure should be carried out in strict accordance with *Acceptance Code for Construction Quality of Building Ground and Foundation* (GB 50202-2002) [2].

(24) Construction of the bore pile should meet the following requirements. The pile location deviation axis and vertical axis direction should be no more than 50 mm, and the verticality direction should be no more than 0.5 %; the dregs at the bored pile bottom should be no more than 100 mm and the hole depth deviation should be between 0 and +300 mm. The concrete strength should meet the design requirements. The binding, hoisting and embedding of the reinforcement cage should also meet the design requirements.

(25) Before constructing the side wall of the working shaft, the underground diaphragm wall should be washed and chiseled. It was required that the surface should be rough surface with unevenness no less than 20 mm.

(26) For any matters regarding construction requirements and quality acceptance standard that were not detailed in the instruction, please refer to the prevailing national regulations and codes.

2. **Requirements for Deformation Compatibility Control of the Foundation Pit**

The foundation pit was 28.25–16.89 m deep, and is the class-1 pit with important coefficient of 1.1. The pit deformation limit was as follows. The retaining wall top horizontal displacement was ≤2 ‰ h_0; the maximum horizontal displacement of retaining wall was ≤3 ‰ h_0; the maximum subsidence of ground surface outside the pit was ≤2 ‰ h_0 (h_0 was the foundation pit depth).

Please refer to Table 6.1 for the pit retaining structure type. The support system of the working shaft foundation pit was made up of five reinforcement concrete supports and one Φ609 mm steel pipe support. The first three reinforcement concrete supports were used in combination with the periphery, top beams and the whist beam of the first underground floor. The support system for the subsequent cut and cover section had four to six supports depending on the depth of the foundation pit. Two rounds were supported using reinforcement concrete and the remaining rounds were supported using the Φ609 mm steel pipes.

The foundation pit was reinforced using the high pressure jet grouting pile. The working shaft was reinforced using the grid stripping. The reinforcement width was 4 m along the wall perimeter. The stripping width was 3.25 m, and the depth was 4 m.

Table 6.1 Retaining structure of test shaft foundation pit for Qianjiang Tunnel

Engineering section	Mileage	Foundation pit depth (m)	Foundation pit width (m)	Support type
Working shaft	LK15 +250.000 to +273.005	28.25	46.5	1200 mm diaphragm wall
Cut and cover section	LK15 +273.005 to +276.500	25.0	38.7	1200 mm diaphragm wall
	LK15 +276.5 to +318.500	24.8–23.5	37.0–38.7	1000 mm diaphragm wall
	LK15 +318.500 to +392.000	18.9–16.89	33.1–37.0	800 mm diaphragm wall

The deepening section of open excavation was reinforced using the skirt edge and stripping. The LK15 +319.7 to +362 section and blocking wall were reinforced using the skirt edge. For the open-cut deepening section, skirt reinforcement (width: 4 m, depth: 4 m) are used. The stripping width was 3.25 m and the depth was 3 m. The longitudinal interval was 6 m. The reinforcement width and depth of the non-deepening section were 3.25 and 3 m respectively.

According to the surrounding environmental of the pit and related codes, the foundation pit was class-1 foundation pit, with importance coefficient of 1.1. Table 6.2 presents the design safety coefficient of the foundation pit.

3. **Stability Assessment and Treatment Measures for Foundation Pit**

(1) **Description of Calculation Method**

The open cut method should be used for the cut and cover section and the working shaft structure. Design of the retaining structure depends on the geological condition, hydrological condition, surrounding environment and foundation pit safety class, as well as the engineering practice and structural calculation analysis results.

The calculation for the retaining wall structure was carried out for the construction stage and operation stage. For the construction stage, the "deformation first and supporting second" principle should be followed to simulate the different working conditions in the entire process of excavation and supporting. During construction stage, for the retaining structure, the stress analysis was carried out according to the construction process. The retaining structure during the excavation period should be used as the support structure to bear all water and soil pressure and

Table 6.2 Design safety coefficient of the foundation pit

Foundation pit class	Pit bottom anti-upheaval	Wall bottom anti-upheaval	Anti-piping	Pit bottom anti-sudden-surging	With supporting anti-topple stability
Class-1	1.8	2.0	1.5	1.2	1.3

lateral pressure caused by ground surface overload. The FEM was used for calculation of the structural displacement and internal force. The displacement variation under various working conditions of excavation was analyzed stepwise using elastic analyses. During the operation stage, it would bear all loads together with the main body structure.

The retaining form was the multi-point bar system structure. The FEM analysis for elastic bar system was used. The spring was used to simulate action of soil below the foundation pit. The spring rigidity was calculated using the "K" method. The passive soil pressure was considered according to the theory of beam on elastic foundation. The m method was used to calculate its horizontal resistance coefficient.

The FEM for isotropic spring elastic bar system was adopted to analyze concrete supporting and purlin system structure as a plane stress problem. The supporting load obtained through retaining wall analysis was used as the input of the initial load of the interaction between the purfin and retaining wall. Furthermore, the horizontal homogeneous spring was used to simulate the deformation compatibility control between the purfin and retaining wall, so as to analyze the stressed deformation of the concrete support and purfin system.

The composite wall form was adopted for the working shaft retaining wall and main body structure sidewall. During structural design of composite walls, the worst combination of load for the entire structure or single member was adopted, and calculation was carried out in stages with the load variation considered according to related codes. According to the geological conditions of the project and related codes, during pit excavation and back-construction stage, the methods that estimate water and earth together and separately were used for the clayey soil and silt, and sandy soil respectively. In the operation stage, the adverse long-term effect of water on structure should be taken into consideration for enveloping using both estimating water and earth together and separately method. The load combination according to the regular use limit status and bearing capacity limit status, and the load-structure mode was adopted for calculation. The worst combination was adopted to check calculate the bending resistance, shear resistance, compression resistance and torsion resistance and crack width of the structure. For the diaphragm wall, the enveloping reinforcement design was carried out according to the different working conditions of the retaining wall and composite wall.

(2) **Calculation Model**

The retaining wall structure was analyzed for the construction stage and the operation stage. The "deformation first and supporting second" principle was followed to simulate the different working conditions in the entire process of excavation and supporting. During the construction stage, for the retaining structure, the stress analysis was carried out according to the construction process. The retaining structure during the excavation period was used as the support structure that would bear all water and soil pressure and lateral pressure caused by ground surface overload. The retaining form was the multi-point bar system structure. The FEM for elastic bar system was used to calculate the displacement and internal force of the structure stepwise, with the displacement variation in the various actual conditions of excavation considered.

The spring was used to simulate action of soil below the foundation pit. The spring rigidity was calculated using the "K" method. The passive soil pressure was considered according to the theory of beam on elastic foundation. The m method was used to calculate its horizontal resistance coefficient.

The safety coefficient of wall bottom upheaval resistance was calculated using the Prandtl and Terzaghi formula [3]. The supporting anti-topple stability coefficient equaled to the ratio of torque induced by the soil pressure on the two sides of the retaining structure below the lowest support to supporting point. Please refer to Figs. 6.10 and 6.11 for the typical calculation model and the load distribution diagram respectively.

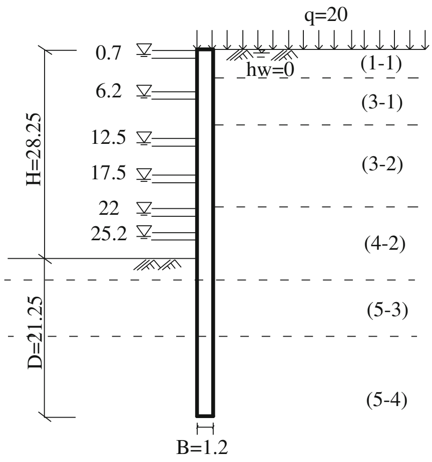
(3) **Emergency Rescue Measures**

The dewatering of the ground water and confined groundwater poses direct impact on safety of the foundation pit excavation. The construction organization should include the emergency plan in the construction organization design. Measures should be taken in emergencies according to the emergency plan to guarantee the safety of the foundation pit.

Construction quality of retaining structures is crucial to the safety of foundation pit. There may be quality defects of the diaphragm wall, which may harm foundation pit safety. The construction organization should prepare related quality defect prevention measures and emergency plan.

The internal support system can guarantee the foundation pit safety and stability, and improve the stress of retaining structure. Measures should be taken during construction to ensure that the location of the purfin and support is accurate and that the pre-applied axial force meets the requirements. No damage to the supporting, stand column, purfin and joints is allowed during construction.

Fig. 6.10 Calculation model



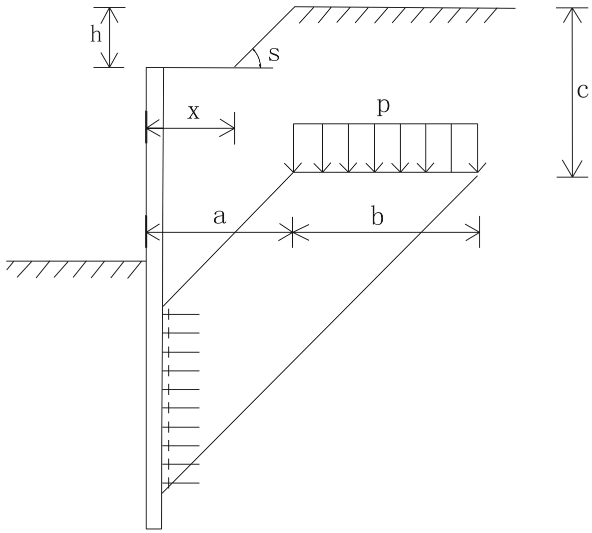


Fig. 6.11 Load distribution

The foundation pit excavation should be carried out layer by layers and blocks by blocks in a good order. Supports should be provided in time; monitoring was required. For appearance of support axial force, structural deformation, foundation pit upheaval and ground surface subsidence, as well as misplace of diaphragm wall joint, water leakage and irregular underground water level, emergency measures should be taken according to the emergency plan.

During geological investigation, scattered methane was discovered in shallow strata, which was reflected by intermittent bubbling. During construction, related measures should be taken according to the specific condition to prevent contamination of environment caused by methane. Emergency plan should be made to ensure the safety of workers and the foundation pit.

(4) **Monitoring of Foundation Pit Stability**

(1) Monitoring is the basis for dynamic design and information-based construction of the retaining structure. The design of retaining structure was adjusted according to the feedback of monitoring results during construction. The finally determined retaining scheme was both safe and economical.
(2) Subsidence of adjacent roads was monitored during construction. If any crack or subsidence was discovered, personnel of related organizations were informed. Analysis and treatment was required.
(3) During construction, the horizontal displacement and reinforcement stress of the retaining structure were measured. If the horizontal displacement exceeded the allowed limit, the support was strengthened or other effective measures were taken; construction only continued with the safety guaranteed.
(4) The support axial force and deflection were monitored during construction to avoid instability and overload.

(5) Cracks of diaphragm walls, nearby buildings and ground surface were monitored during construction to ensure stability and safety of the foundation pit.
(6) The monitoring and warning indexes were controlled by accumulated variation and variation rate. The warning indexes of accumulated variation are given in "construction monitoring diagram" in details.
(7) The observational data and analyzing results were included in the completion data to be handed over for examination.

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