JTG

Industry Standards of
the People's Republic of China
中华人民共和国行业标准

JTG 3370.1—2018(EN)

Specifications for Design of Highway Tunnels
Section 1 Civil Engineering

公路隧道设计规范 第一册 土建工程

(英文版)

Issue date: December 25, 2018 Effective date: May 1, 2019

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Editing organization in charge: China Merchants Chongqing Communications Technology Research & Design Institute Co., Ltd.

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中华人民共和国交通运输部

公　　告

第 50 号

交通运输部关于发布《公路隧道设计规范　第一册　土建工程》英、法文版等 7 项公路工程行业标准外文版的公告

为促进公路工程行业标准的国际合作与共享，现发布《公路隧道设计规范　第一册　土建工程》英文版［JTG 3370.1—2018（EN）］［代替标准号 JTG D70—2004（E）］及法文版［JTG 3370.1—2018（FR）］、《公路隧道设计规范　第二册　交通工程与附属设施》法文版［JTG/T D70/2—2014（FR）］、《公路隧道照明设计细则》英文版［JTG/T D70/2-01—2014（EN）］、《公路隧道通风设计细则》英文版［JTG/T D70/2-02—2014（EN）］、《公路隧道抗震设计规范》英文版［JTG 2232—2019（EN）］、《公路隧道养护技术规范》英文版［JTG H12—2015（EN）］。

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中华人民共和国交通运输部

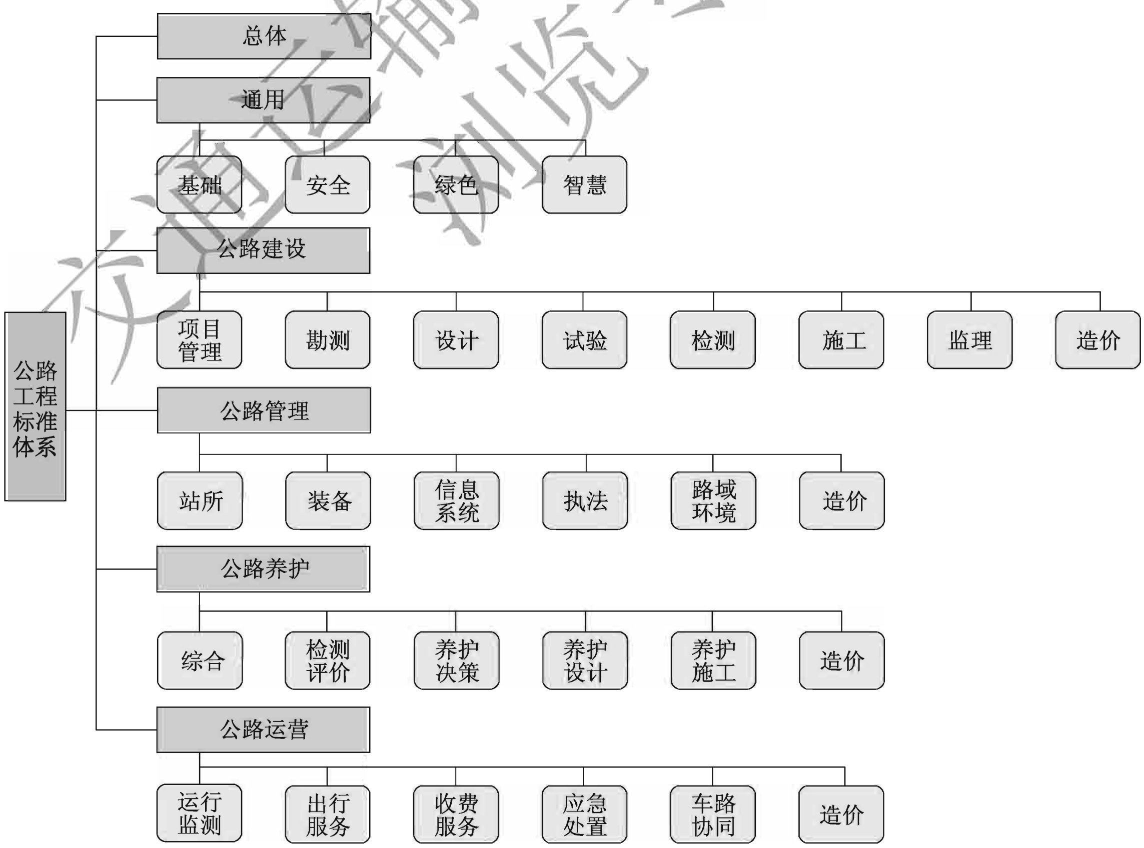
2023 年 9 月 20 日

交通运输部办公厅　　2023 年 9 月 21 日印发

英文版编译出版说明

标准是人类文明进步的成果,是世界通用的技术语言,促进世界的互联互通。近年来,中国政府大力开展标准化工作,通过标准驱动创新、协调、绿色、开放、共享的共同发展。在丝绸之路经济带与21世纪海上丝绸之路,即"一带一路"倡议的指引下,为适应日益增长的全球交通运输发展的需求,增进世界连接,促进知识传播与经验分享,中华人民共和国交通运输部组织编译并发布了一系列中国公路行业标准外文版。

中华人民共和国交通运输部发布的公路工程行业标准代号为JTG,体系范围涵盖公路工程从规划建设到养护和运营管理全过程所需要的设施、技术、管理与服务标准,也包括相关的安全、环保和经济方面的评价等标准。



《公路隧道设计规范 第一册 土建工程》(简称《规范》)是中国山岭公路隧道设计与建造的重要技术标准,主要用于各等级新建和改扩建山岭公路隧道的地质勘察、隧道结构、荷载计算、防水与排水、辅助工程、特殊地址段、路基路面、抗震设计和洞内预留预埋等设计,可供监督管理部门、地质队、设计院、施工企业、工程监理等使用。1990 年由交通部首次发布实施《公路隧道设计规范》(JTJ 026－90),其前瞻性引入了当时具有世界先进水平的公路隧道设计和施工方法,为此后山岭公路隧道建设提供了基础性保证,在发挥了重要作用。在充分总结中国相关科研成果和大量工程经验的基础上,吸收借鉴国际先进的山岭公路隧道设计与建造技术,不断进行改进与修订,2004 年曾发布修订版。随着 21 世纪以来中国公路隧道建设规模迅猛发展,2022 年中国公路隧道总量达 24850 处、2678.43 万延米,并以年均超 12% 的速度快速增长,公路隧道规模的扩大、公路隧道种类的增多,公路隧道设计与建造技术取得长足进步,2018 年再次发布新版《规范》,重点对以钻爆法为主要开挖手段的新建山岭公路隧道和改扩建山岭公路隧道设计方法的技术要求进行统一和规范。《规范》以安全、耐久、经济、节能、环保为基本制订原则,强调隧道主体结构具有规定的强度、稳定性和耐久性,方便养护和维修作业,并提倡动态设计与信息化施工的思想,可适用于不同类型山岭公路隧道土建工程设计的要求。

本英文版的编译发布便是希望将中国的工程经验和技术成果与各国同行进行交流分享,为其他国家山岭公路隧道设计与施工提供参考借鉴。

《公路隧道设计规范 第一册 土建工程》英文版的编译工作由中华人民共和国交通运输部委托招商局重庆交通科研设计院有限公司主持完成,并由中华人民共和国交通运输部公路局组织审定。本规范在编译过程中得到欧美多名专家的支持,特别感谢巴基斯坦专家 Asim Amin、澳大利亚专家 Bedi Anmol、新加坡专家温大志、中国专家严金秀,以及巴基斯坦专家 Babar Khan、Usman Jilani、Rasheed Ahmed 等在编译与审定期间给予的协助与支持。

《公路隧道设计规范 第一册 土建工程》英文版标准的内容与现行中文版一致,如出现异议时,以中文版为准。

感谢中文版主要编写者蒋树屏、程崇国先生在本英文版编译与审定期间给予的指导与支持。

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英文版参审：张慧彧（中国），严金秀（中国），温大志（新加坡），
Babar Khan（巴基斯坦），Abu Bakar（巴基斯坦），
Usman Jilani（巴基斯坦），Rasheed Ahmed（巴基斯坦）

The People's Republic of China

Ministry of Transport

Public Notice

No.50

Public Notice on Issuing the English and French Versions of Seven Highway Engineering Industrial Standards including *Specifications for Design of Highway Tunnels Section 1 Civil Engineering*

The English and French versions of *Specifications for Design of Highway Tunnels Section 1 Civil Engineering* [JTG 3370.1—2018 (EN), substituting JTG D70—2004 (E); and JTG 3370.1—2018 (FR)], the French version of *Specifications for Design of Highway Tunnels Section 2 Traffic Engineering and Affiliated Facilities* [JTG D70/2—2014 (FR)], the English version of *Guidelines for Design of Lighting of Highway Tunnels* [JTG/T D70/2-01—2014 (EN)], the English version of *Guidelines for Design of Ventilation of Highway Tunnels* [JTG/T D70/2-02—2014 (EN)], the English version of *Specifications for Seismic Design of Highway Tunnels* [JTG 2232—2019 (EN)], and the English version of *Technical Specifications of Maintenance for Highway Tunnel* [JTG H12—2015 (EN)] are issued hereby for promoting international cooperation and sharing of standards in highway engineering industry.

The general administration and final interpretation of theforeign language versions of the above mentioned standards belong to Ministry of Transport, while particular interpretation for application and routine administration shall be provided by China Merchants Chongqing Communications Technology Research & Design Institute Co., Ltd.

Inevent of any ambiguity or discrepancies between the foreign language versions and Chinese version, the Chinese version should be referred and accepted.

Comments, suggestions and inquiries are welcome and should be addressed to China Merchants Chongqing Communications Technology Research & Design Institute Co., Ltd. (Address: Institute of Tunnel and Underground Engineering, No. 33 Xuefu Avenue, Nan’an District, Chongqing, P. R. China; Postal Code: 400067; E-mail: chengliang@cmhk.com).

It is hereby announced.

Ministry of Transport of the People’s Republic of China

September20, 2023

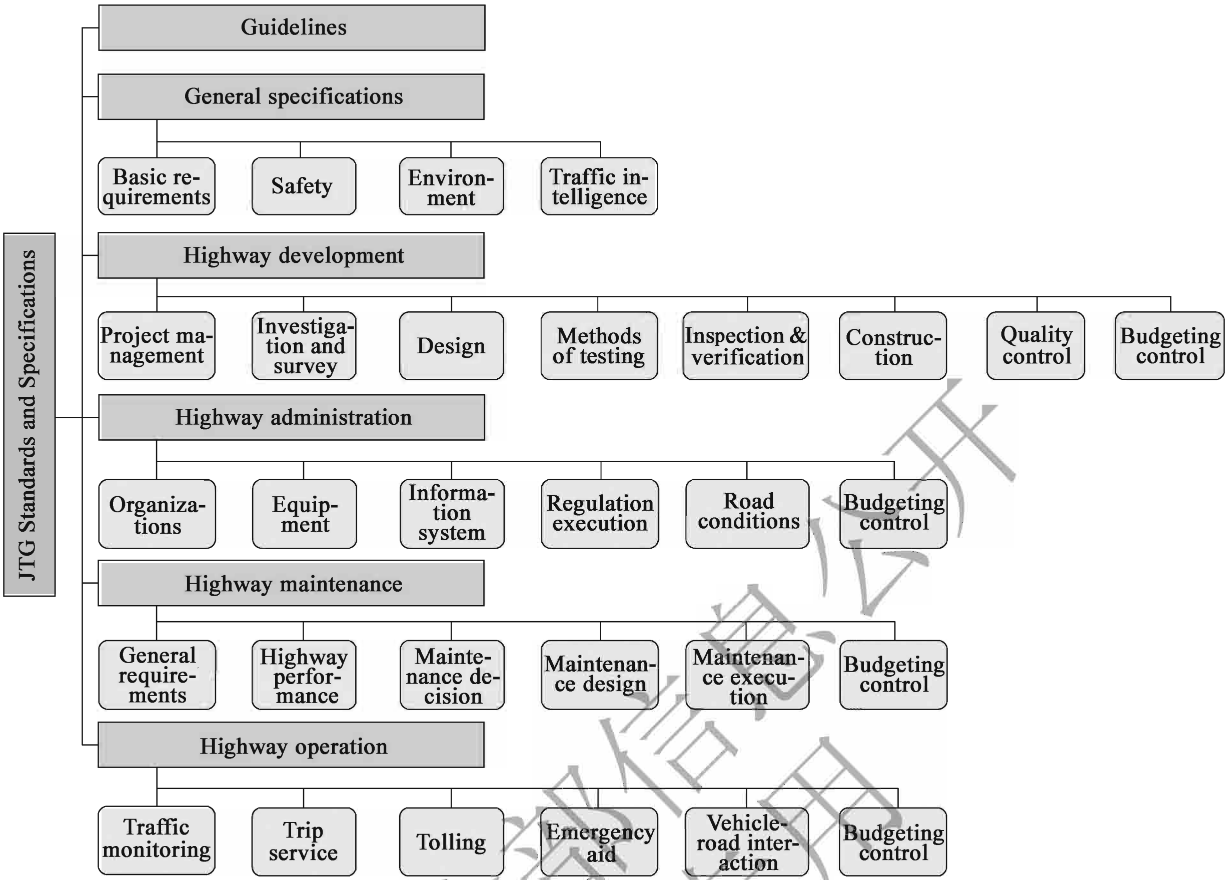
General Office of Ministry of Transport   Printed on September 21, 2023

Introduction to English Version

Standards reflect the achievement of civilization and progress, provide common languages for technical communications and improve global connectivity. In recent years, Chinese government has been proactively implementing the standardization to stimulate innovation, coordination, greening and opening up for shared development in China and worldwide. In light of mutual development along the Silk Road Economic Belt and the 21st-Century Maritime Silk Road (so called the "One Belt One Road" initiative), the Ministry of Transport of the People's Republic of China organized translation and published international version of Chinese highway industry standards and specifications to cope with the increasing demands for international cooperation in world transportation, achieve interconnected development and promote knowledge dispersion and experience sharing.

JTG is the designation referring to the standards and specifications of highway transportation industry, issued by the Ministry of Transport of the People's Republic of China. It covers the standards and specifications in terms of facilities, technology, administration and service for whole process from highway planning through to highway maintenance. The criteria for safety, environment and economy assessment are also included.

The Specifications for Design of Highway Tunnel: Section 1-Civil Engineering (hereinafter referred to as the Specifications) are important technical standards for the design and construction of mountain highway tunnels in China. It is mainly used by supervision and management departments, geological teams, design institutes, construction enterprises and engineering supervisors for geological surveys, tunnel structure, load calculation, water resistance and drainage, auxiliary engineering, special address field, subgrade and pavement, seismic design and reserved embedment of new, modified and expanded mountain tunnels of all levels. The Specifications for Design of Highway Tunnels (JTJ 026—90) were first issued for implementation by the Ministry of Transport of the People's



Republic of China in 1990, which foresightedly introduced the design and construction methods of highway tunnels with the world advanced level at that time, provide a basic guarantee for the construction of mountain highway tunnels and play an important role. On the basis of fully summarizing the relevant scientific research achievements and rich engineering experience of China, the Specifications were improved and revised continuously with reference to international advanced design and construction technology of mountain highway tunnels. In 2004, the revised version was released. With the rapid development of highway tunnel construction scale in China since the 21st century, the total number of highway tunnels in China reached 24,850 in 2022, amounting to 26,784,300 linear meters, and grew rapidly at an average annual rate of over 12%. With the expansion of highway tunnel scale and the increase of highway tunnel types, the design and construction technology of highway tunnels has made great progress. In 2018, the new version of the Specifications was released again, focusing on unifying and standardizing the technical requirements for the design methods of new mountain highway tunnels and modified & expanded mountain highway tunnels excavated by the drill – and – blast method. Based on the basic principles of safety, durability, economy, energy saving, and environmental protection, the Specifications emphasize the specified strength, stability, and durability for the main structure of tunnels, maintenance, and repair convenience, and advocate the idea of dynamic design and information - based construction. The Specifications are applicable to the design requirements for civil engineering of different types of mountain highway tunnels.

The purpose of compiling and publishing this English version is to exchange and share China's engineering experience and technical achievements with counterparts in other countries, and to provide reference for the design and construction of mountain highway tunnels in other countries.

The Ministry of Transport of the People's Republic of China entrusted China Merchants Chongqing Communications Technology Research & Design Institute Co., Ltd. to preside over the compilation of the English version of Specifications for Design of Highway Tunnel: Section 1-Civil Engineering, and the Highway Bureau of the Ministry of Transport of the People's Republic of China organized the review. These Specifications were supported by many experts in Europe and America during compilation. Special thanks are also given to Pakistani expert Asim Amin, Australian expert Bedi Anmol, Singaporean expert Wen Dazhi, Chinese expert Yan Jinxiu, Pakistani experts Babar Khan Usman Jilani and Rasheed Ahmed for their assistance and support during the editing and approval of these Specifications.

The English version of Specifications for Design of Highway Tunnel: Section 1-Civil Engineering is consistent with the current Chinese version. In the event of any ambiguity or discrepancies, the Chinese version shall be referred and accepted.

Gratitude is given here to Mr. Jiang Shuping and Mr. Cheng Chongguo, the editors in charge of Chinese version, for their guidance and support during the editing and approval of the English version.

Comments, suggestions and inquiries are welcome and should be addressed to the editing organization in charge of the English version (address: Tunnel and Underground Engineering Research Institute, Merchants Chongqing Communications Technology Research & Design Institute Co., Ltd., No. 33, Xuefu Avenue, Nan'an District, Chongqing, postal code: 400067, e-mail: chengliang@cmhk.com).

Editing organization in charge for English version: China Merchants Chongqing Communications Technology Research & Design Institute Co., Ltd.

Chief editor for English version: Wu Mengjun

Associate editors for English version: Ding Hao, Hu Xuebing, Hu Juyi, Cheng Liang, Li Ke, Guo Hongyan, Wang Daoliang, Wu Shengzhong, Wang Jianhua and Xu Xu

Review editor for English version: Asim Amin (Pakistan) and Bedi Anmol (Australia)

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Foreword to Chinese Version

According to The Ministry of Transport *Notice on Issuing the Order for Highway Engineering Standard Development/Revision Program* 2010 (TGLZ [2010] No. 132), China Merchants Chongqing Communications Technology Research & Design Institute Co., Ltd., as the principal drafter, has undertaken the task of revising the *Specifications for Design of Highway Tunnel* (JTG D70—2004) (hereinafter called the "previous edition").

The previous edition has been comprehensively revised on the guiding principle of "safety, durability, economy, energy efficiency and environmental protection", on the basis of highway tunnel design & research achievements and experiences across China in recent years and fully drawing on domestic and international standards and advanced technologies on highway tunnels. It has been approved and issued for implementation as the *Specifications for Design of Highway Tunnel: Section 1-Civil Engineering* (JTG 3370.1—2018) (hereinafter called "this edition").

This revised edition is organized into 18 chapters and 14 Appendices: General; Terminology and Symbols; Tunnel Survey and Surrounding Rock Classification; Overall Design; Building Materials; Load; Portal & Portal; Lining Structural Design; Structural Calculation; Waterproofing and Drainage; Special Types of Tunnels; Auxiliary Channel; Auxiliary Engineering Measures; Design of Tunnels in Special Geology; Tunnel Subgrade and Pavement; Seismic Design; Reconstruction and Expansion Design; In-tunnel Reservation, Embedment and Structures; Appendices A-P.

The changes with respect to the previous edition include:

1. Changed the applicable scope to "all classes of new, reconstructed or expanded highway tunnels excavated primarily by drill-and-blast method" from the

previous "all classes of highway two-lane tunnels drill-and-blast method";

2. Aligned highway tunnel construction gauge with other standards and specifications; adjusted and supplemented provisions on maintenance access/sidewalk, emergency stop zone, pedestrian and vehicle cross passages and parallel adit for single-tube bidirectional extra-long tunnel;

3. Added provisions on tunnel construction monitoring and measurement and advance geological forecast;

4. Adjusted and supplemented provisions on building materials;

5. Added calculation methods for rock load and seismic load on neighborhood and twin-arch tunnels;

6. Supplemented and adjusted structural design contents by simplifying structural design of two-lane tunnel lining and adding to structural design of large-span highway tunnel accommodating three or four lanes;

7. Added provisions on partial load combination and its safety factor; introduced the strength reduction finite element method (FEM) for safety factor of surrounding rock during construction;

8. Improved provisions on design of tunnel waterproofing and drainage and supplemented provisions on waterproofing and drainage design for tunnels in cold areas;

9. Improved provisions on design of neighborhood and twin-arch tunnels and supplemented provisions on design of special tunnels such as branching-out and shed tunnels;

10. Improved design specifications for auxiliary caverns such as vertical shaft, inclined shaft, cross passage and underground machine room;

11. Added and detailed design contents of auxiliary engineering measures;

12. Subdivided the "karst and goaf" from the previous edition into two sections

(“karst” and “goaf”); supplemented design specifications for loess tunnel; changed “rockburst“ from the previous edition to “High in-situ stress”; added design specifications for permafrost tunnels;

13. Supplemented and adjusted provisions on cement concrete pavement and composite pavement in tunnels; added design requirements for CRC (Continuously Reinforced Concrete) surface course; added provisions on installing drainage under the tunnel pavement structure;

14. Deleted design specifications for ventilation and lighting in the previous edition;

15. Added three new chapters: “Seismic Design”, “Reconstruction and Expansion Design”and “In-tunnel Reservation, Embedment and Structures”.

Comments and suggestions from users of this publication are welcome and should be addressed to the routine administration team (Attention: Cheng Chongguo, Tunneling and Underground Engineering Branch, No. 33, Xuefu Avenue, Nan'an District, Chongqing 400067; e-mail: chengchongguo@cmhk.com; Tel: 023-62653439; Fax: 023-62653128). The feedback will be considered in the next edition.

Editing organization in charge: China Merchants Chongqing Communications Technology Research & Design Institute Co., Ltd.

Associate editing organizations:

Zhejiang Provincial Institute of Communications Planning, Design & Research
Tongji University
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CCCC Second Highway Consultants Co., Ltd.
Chongqing Jiaotong University
Research Institute of Highway Ministry of Transport
Hunan Provincial Communications Planning, Survey & Design Institute

Chief editor: Jiang Shuping

Associate editors:

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Liu Hongzhou	Lin Zenghai		

Contributors:

Xiao Bo Xu Chongbang Wan Jianhua

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1 General Provisions

1.0.1 These Specifications are intended to provide standards and guidelines for design of highway tunnels.

1.0.2 These Specifications are applicable to all classes of new, reconstructed or expanded highway tunnels excavated primarily by drill-and-blast method.

1.0.3 Tunnel designs shall fulfill highway functions and follow the basic principles of "safety, durability, economy and environmental protection".

1.0.4 Highway tunnels can be grouped into four categories by length, as specified in Table 1.0.4.

Table 1.0.4 Classification of highway tunnels by length

Classification	Extra-long tunnel	Long tunnel	Medium tunnel	Short tunnel
Length (m)	$L > 3000$	$3000 \geq L > 1000$	$1000 \geq L > 500$	$L \leq 500$

Note: The length, L of a tunnel is the distance between the intersections of its axis at the top of the road surfaces and end planes of the portal lining at both ends.

1.0.5 The primary tunnel structure shall be designed as a permanent structure with specified strength, stability and durability. It shall meet all the service life requirements, and shall allow for easy maintenance and repair.

1.0.6 The design of civil engineering components of a tunnel such as the primary structure, pavement, waterproofing and drainage shall be carried out in comprehensive consideration of the design of operational facilities, such as the ventilation, lighting, traffic monitoring, power supply and distribution and fire protection.

1.0.7 Civil engineering design of a tunnel shall follow the principle of observational design approach with information technology based construction monitoring. A general plan for geological inspection, prediction, and construction monitoring and measurement shall be developed to provide basis for observational design approach to make timely adjustment to support parameters and construction methods.

1.0.8 Tunnel design shall be such that land use is minimal, existing vegetation preserved wherever possible and muck and sewage properly disposed.

1.0.9 Tunnel design shall proactively and safely adopt new technologies, materials, equipment and processes in line with relevant national technological and economic policies.

1.0.10 Highway tunnel design shall comply with relevant current national and industrial standards, in addition to these Specifications herein.

2 Terms and Symbols

2.1 Terms

2.1.1 Highway tunnel

An underground passageway for motor vehicles, non-motor vehicles and pedestrians to pass through. Highway tunnels are commonly classified into motor vehicle tunnels and mixed-use tunnels for motor vehicles, non-motor vehicles and pedestrians

2.1.2 Mountain tunnel

A tunnel featured by excavation through mountains

2.1.3 Drill-and-blast tunnel

A tunnel excavated by manually or mechanically drilling boreholes, loading explosives and controlled blasting

2.1.4 Cut-and-cover tunnel

A tunnel built by cut and cover method

2.1.5 Shed tunnel

A highway structure formed by open excavation followed by constructing a roof supported by columns on one side and walls on the other.

2.1.6 Construction gauge

Clear defined-space required in highway tunnels to ensure vehicular and pedestrian passage

2.1.7 Clear section

The cross-section area surrounded by the intrados of tunnel lining, pavement and side drain

2.1.8 Emergency stop zone

An area inside the tunnel for temporary parking of faulty or inspection vehicles

2.1.9 Cross passage

A transverse, nearly horizontal passage linking two tunnels or caverns, or linking the tunnel to ground surface

2.1.10 Vertical shaft

A vertical excavation provided to improve operational ventilation or construction conditions

2.1.11 Inclined shaft

An inclined excavation at a certain angle to improve operational ventilation or construction conditions

2.1.12 Twin tunnels with small clearance

Two parallel tunnels constructed in sufficiently close proximity to each other such that one will have a structural impact on the other.

2.1.13 Twin-arch tunnel

Two parallel arch tunnels with no rock pillar in between and their man-made support structures connected together

2.1.14 Branching-out tunnel

A bidirectional large-span or twin-arch tunnel which transitions into two separate tunnels

2.1.15 Surrounding rock classification

Classification of surrounding rock by stability property according to rock mass integrity and strength

2.1.16 Rock Quality Designation

Rock Quality Designation determined using rock hardness and integrity as basic parameters

2.1.17 Modified Rock Quality Designation

Rock Quality Designation corrected according to groundwater, main weak structure plane, initial stress, etc.

2.1.18 Load

A force acting on and creating stress within a structure

2.1.19 Loosening pressure

The pressure acting on the lining structure as a result of loosening of surrounding rock mass

2.1.20 Distortional pressure

The pressure acting on the lining structure as a result of deformation of surrounding rock mass

2.1.21 Rock load

The pressure acting on the lining structure as a result of deformation or loosening of surrounding rock mass. It is a collective term for distortional pressure and loosening pressure

2.1.22 Eccentric load

Unsymmetrical load acting on a tunnel's lining structure

2.1.23 Portal

A structure provided to support and retain rock and soil on the heading slope at the opening of a tunnel

2.1.24 End-wall tunnel portal

A retaining wall provided to retain soil on the heading slope at the opening of a tunnel

2.1.25 Cut-and-cover tunnel portal

A structure provided to protect the slope at the opening of a tunnel and to enable its lining structure to extend outward

2.1.26 Lining

Shell structure used to provide support to the surrounding rock of a tunnel

2.1.27 Tunnel invert

An arch inverted arch lining at the bottom of a tunnel

2.1.28 Shotcrete and rockbolt lining

A surrounding rock support structure composed of shotcrete, rockbolt, reinforcement mesh, steel rib support or a combination thereof

2.1.29 Monolithic lining

A tunnel lining structure constructed of concrete placed in formwork or masonry after tunneling

2.1.30 Composite lining

A composite lining structure composed of shotcrete and rockbolt lining, waterproofing layer and concrete placed in formwork

2.1.31 Pilot tunnel

A small adit driven through the excavated cross-section ahead of the tunnel face of the main tunnel

2.1.32 Advance geological forecast

Activities to detect, examine and evaluate geological conditions ahead of the tunnel face by means of geophysical exploration or drilling

2.1.33 Tunnel construction monitoring and measurement

Activities to observe, monitor, examine and evaluate deformation of surrounding rock and strata as well as the stress and deformation of support structures inside the tunnel or on the ground surface, using various tunnel construction monitoring and measurement instruments and tools

2.1.34 High in-situ stress

An in-situ stress field high enough to causes rock burst and rock stripping in hard rock strata or large deformation and significant decrease of clear section after the tunnel is driven

2.2 Symbols

BQ—Rock Quality Designation;

$[BQ]$—Modified Rock Quality Designation;

R_c—Saturated uniaxial compressive strength of rock;

R_a—Ultimate compressive strength of concrete or masonry;

R_w—Ultimate compressive strength of concrete in bending;

R_1——Ultimate tensile strength of concrete;

$I_{S(50)}$—Measured point load strength index of rock;

K_1—Groundwater influence correction coefficient;

K_2—Correction coefficient for influence of main weak structure plane occurrence;

K_3—Correction coefficient for influence of initial stress;

K_v—Rock mass integrity index;

J_v—Volumetric joint count of rock;

S_n—The number of joints per meter of survey line in the nthjoint set;

S_k—The number of joints not in sets per cubic meter of rock;

v_{pm}—Velocity of elastic longitudinal wave in rock mass;

v_{pr}—Velocity of elastic longitudinal wave in rock;

σ_{max}—Maximum initial stress perpendicular to tunnel axis;

γ—Unit weight of surrounding rock;

k—Coefficient of elastic resistance;

E—Modulus of deformation;

μ—Poisson's ratio;

φ—Internal friction angle;

φ_c—Calculated friction angle;

B—Width of excavated cross-section of a tunnel;

W—Width of carriageway;

L_L—Left lateral width;

L_R—Right lateral width;

L—Tunnel length;

C—Width allowance;

J—Width of maintenance access;

h—Height of maintenance access or sidewalk;

R—Sidewalk width;

H—Tunnel construction gauge height;

K—Coefficient of elastic resistance of surrounding rock;

δ—Lining displacement;

n—Ratio of side slopes excavated;

m—Ratio of backfilled slope surface.

3 Tunnel Survey and Surrounding Rock Classification

3.1 General

3.1.1 The content and scope of survey data to be collected shall be established according to the objectives at each phase of the tunnel design. Scoping surveys shall also consider the highway class, tunnel requirements and size before carrying out data collection, survey, mapping, exploration and test. The survey data shall be complete and accurate to provide sufficient information to fulfil the design requirements.

3.1.2 Surveys shall be conducted both before and during construction. The pre-construction survey shall be scoped to adequately inform the intent and accuracy requirements of the corresponding design phase. Surveys during construction shall be conducted in a timely manner to verify the design assumptions and predict problematic geological conditions that may be expected during construction; surveys performed during construction should provide a basis for design changes and adjustments that may be required to facilitate safe and efficient construction.

3.1.3 Appropriate survey plans shall be developed according to topographic and geologic conditions of the tunnel site area, taking into account design phase, method and scope. Such plans shall be promptly corrected if the actual conditions are found during the survey to be inconsistent with assumptions.

3.1.4 The rock mass through which the tunnel is excavated shall be characterised using a combination of qualitative analysis and quantitative calculation.

3.2 Collection of information

3.2.1 The following information for the tunnel site area shall be collected:

1 Topographical data and related remote sensing and telemetering data;

2 Engineering geological, hydrogeological and surface water information, especially data on the type, nature, scale and hazard level of natural geological disasters;

3 Geologic mapping and exploration data;

4 Meteorological data including air temperature, rainfall, wind speed and direction;

5 Seismic history and ground motion parameters;

6 Traffic and construction conditions along the tunnel alignment;

7 Mineral resources along the tunnel alignment, existing works nearby, etc.

3.2.2 Social, cultural, environmental information, applicable laws and regulations shall be determined.

3.3 Topographical and geological survey

3.3.1 The objectives, content and extent of survey area of tunnel survey in various phases may be determined in accordance with Table 3.3.1.

Table 3.3.1 Objectives, content and scope in various phases of survey

Phase		Objective	Content	Extent of Survey Area
Pre-construction	Site reconnaissance	Provide basic data for developing tunnel alignment options.	Collect and analyze existing topographic, regional geological and meteorological data along the alignments; verify environmental, geological disaster, existing building, road traffic and construction planning data along the alignments.	To cover an area larger than that of all possible options
	Preliminary site investigation	Provide basic data for comparison of preliminary design options, cost estimate and next-phase investigation.	Collect and analyze the data obtained from previous phase; carry out preliminary site investigation and topographic mapping for proposed alignment options; carry out necessary geophysical exploration, drilling and testing.	To cover an area larger than that of all possible options

continued

Phase		Objective	Content	Extent of Survey Area
Pre-construction	Detailed site investigation	Obtain required data for technical design, calculations analyses and construction drawings; construction programme and cost estimation.	Detailed topographic mapping and worksite topographic mapping; detailed geological and environmental survey; borehole drilling, geophysical exploration and testing according to the design requirements	Both sides and surrounding areas of tunnel alignment; to be expanded as appropriate for extra-long tunnels, long tunnels and tunnels in karst
During construction		Predict and confirm engineering geological and hydrogeological conditions encountered during construction.	Supplementary topographic, geologic and environmental surveys; in-tunnel observation, measurement, advance detection and prediction	Construction influence zone within the tunnel and on ground surface

3.3.2 Tunnel engineering mapping shall comply with the following provisions:

1 Topographic maps, profiles and cross-sections shall be collected or prepared according to the requirements of the design phases.

2 The content and accuracy of the mapping data shall comply with the current *Code for Survey of Highway Engineering Geology* (JTG C20) and *Specifications for Highway Reconnaissance* (JTG C10).

3 Horizontal and vertical control points shall be set up as specified adjacent to the opening of the tunnel and auxiliary channels.

3.3.3 Pre-construction topographic and geological surveys shall include natural geography, engineering geology and hydrogeology etc., and based on the requirements of the phases, focus on the survey and analysis of:

1 Formation lithology and nature, type and scale of geological structures.

2 Characteristics of faults, joints and weak structural planes and their combined relationship with the tunnel; basic physical and mechanical properties of surrounding rock.

3 Groundwater type, groundwater table, aquifer distribution and the corresponding permeability, water volume, recharge relationship, water quality and its corrosivity to concrete, presence or absence of abnormal water inflow and water gushing.

4 Unfavorable geology and special rock and soil (including collapse, fault, talus, landslide, karst, natural or artificial pit, goaf, debris flow, running sand, collapsible loess, saline soil, rock salt, terrestrial heat, permafrost and glacier); their origin, evolvement, type, scale and trend; the extent of their impact on the stability of tunnel portal and proper.

5 Identify hazardous gas or rock strata, their distribution and the hazardous components and content, and predict and assess their impact on construction and operation.

6 Determine the seismic peak ground acceleration coefficient for the tunnel area in accordance with the *Seismic Ground Motion Parameter Zonation Map of China* (GB 18306) or Earthquake Authority's evaluation.

3.3.4 Topographic and geological surveys shall be carried out in accordance with the following requirements:

1 If regional faults, especially Holocene active fractures and seismogenic faults, are present in the tunnel area, the signs and characteristics of neotectonic activity and its relationship with seismic activity shall be investigated. The extent of its impact on tunnel works shall also be ascertained.

2 If major unfavorable geology or special rock and soil conditions affecting tunnel schemes are present in the tunnel site area, more geological data shall be collected for comprehensive analysis, prediction of locations where collapse, landslide, high in-situ stress, karst, water gushing and mud burst, running sand and hazardous gas overflow are possible after tunnel excavation and proposal of appropriate engineering measures.

3 For tunnels in complex geo-hydrological conditions, real-time hydrogeological monitoring over a period of time or studies on specific issues shall be performed when necessary, in addition to survey, exploration and test required for normal tunnels.

4 For tunnels along a river or mountain, survey and analysis shall be carried out to understand geological characteristics and the stability of slopes and the impact of current scour on the mountain and tunnel stability.

5 For tunnels next to a reservoir, the bank slope stability, reservoir capacity and water level (including wave height and back-water height) shall be ascertained. If the tunnel portal is located in a karst depression or at the bottom of a gully, the maximum level of seasonal back-water in the depression or gully shall be ascertained.

3.3.5 Geological survey during construction shall include the following:

1 Verify formation lithology, tectonics and groundwater based on observation of surrounding rock exposed by excavation; and determine the actual class of surrounding rock.

2 Detect and predict possible condition of surrounding rock or unfavorable geology ahead of the face as well as their location, nature and scale.

3.4 Meteorological survey

3.4.1 Meteorological survey shall cover the air temperature, air pressure, wind speed, wind direction, rainfall, snow accumulation, snow line, glacier characteristics, frozen depth, fogginess and the number of foggy days as well as past meteorological disasters. Extreme values shall be investigated for air temperature, wind speed, rainfall and snow accumulation.

3.4.2 Meteorological observation points (stations) should be set up as necessary at the tunnel site to continuously collect local meteorological data.

3.5 Survey of engineering environment

3.5.1 Natural environment conditions in and around the tunnel site area, including surface drainage system, outcrop of groundwater, fountain, hot spring, swamp, lakes, vegetation, mineral resources and wildlife, etc. shall be investigated.

3.5.2 Land use, farmland, water conservancy facilities, buildings, underground pipelines, etc. in the site area shall be investigated. The current conditions of key features in the site area, such as parks, forest reserves, cultural heritage sites and memorial buildings shall be investigated. The possible impact on them from tunnel construction shall be evaluated.

3.5.3 The environmental impact due to water consumption for construction and domestic use, traffic conditions and noise, vibration, sewage and release of exhaust that are generated during construction and operation shall be investigated. The extent of environmental impact from possible ground settlement, collapse, damage to surface buildings and depletion of water sources for industrial or domestic use as a result of groundwater loss during construction and operation shall be investigated and predicted.

3.5.4 Investigation of construction conditions shall include:

1 Traffic conditions, construction access, construction site, demolition and relocation, muck bank, water supply, power supply and communication conditions;

2 Source, quality, quantity, etc. of building materials;

3 Other factors that may affect construction.

3.6 Surrounding rock classification

3.6.1 The comprehensive evaluation of the class of the rock surrounding the tunnel should adopt the two steps as follows:

1 To carry out preliminary classification based on qualitative characteristics of the two basic elements, rock hardness and integrity; and the quantitative Rock Quality Designation, BQ.

2 On the basis of the basic quality index to obtain the modified Rock Quality Designation [BQ] by modifying the Rock Quality Designation, taking into account the effect of correction elements . Then comprehensive assessment is performed in conjunction with the qualitative characteristics of the rock to determine the detailed classification of the surrounding rock.

3.6.2 The Rock Quality Designation BQ of the surrounding rock shall be calculated from Eq. (3.6.2), using quantitative values of the classification elements, R_c and K_v:

$$BQ = 100 + 3R_c + 250K_v \tag{3.6.2}$$

subject to the following restrictions:

1 If $R_c > 90K_v + 30$, then $R_c = 90K_v + 30$ and K_v shall be substituted into the equation to calculate the value of BQ.

2 If $K_v > 0.04R_c + 0.4$, then $K_v = 0.04R_c + 0.4$ and R_c shall be substituted into the equation to calculate the value of BQ.

The values of R_c and K_v may be determined respectively in accordance with sections A.0.1 and A.0.2 herein.

3.6.3 When determining the detailed classification of surrounding rock, the Rock Quality Designation, BQ shall be corrected based on the extent of influence from groundwater, main weak structure planes and initial stress by applying Eq. (3.6.3):

$$[BQ] = BQ - 100(K_1 + K_2 + K_3) \tag{3.6.3}$$

where

$[BQ]$—Modified Rock Quality Designation;

K_1—Groundwater influence correction coefficient;

K_2—Correction coefficient for influence of main weak structure plane occurrence;

K_3—Correction coefficient for influence of initial stress.

The values of K_1, K_2 and K_3 may be determined according to Tables A.0.3-1, A.0.3-2 and A.0.3-3 in Appendix A hereto.

3.6.4 Surrounding rock mass class may be determined according to Table 3.6.4 based on survey, exploration and test data, qualitative characteristics of surrounding rock, Rock Quality Designation BQ or modified Rock Quality Designation [BQ], soil type and compactness of surrounding soil, etc., subject to the following requirements:

1 For the purposes of surrounding rock classification, qualitative division of rock hardness and integrity may be determined in accordance with A.0.5 and A.0.6 herein.

2 In cases where qualitative classification of surrounding rock based on main characteristics is inconsistent with the classification based on the value of BQ or [BQ], the qualitative characteristics and reliability of quantitative index parameters shall be re-examined, re-observed and re-tested.

3 During stages of engineering feasibility study and Preliminary site investigation, surrounding rock may be classified by qualitative or engineering analogy methods.

Table 3.6.4 Classification of rock mass surrounding highway tunnels

Surrounding rock class	Main qualitative characteristics of surrounding rock or soil	Rock Quality Designation BQ or modified Rock Quality Designation [BQ]
Ⅰ	Hard, intact rock mass	>550
Ⅱ	Hard, relatively intact rock mass Medium hard, intact rock mass	550~451
Ⅲ	Hard, relatively crushed rock mass Medium hard, relatively intact rock mass Medium soft, intact rock mass in monolithic or thick-layer structure	450~351

continued

Surrounding rock class	Main qualitative characteristics of surrounding rock or soil	Rock Quality Designation BQ or modified Rock Quality Designation [BQ]
Ⅳ	Hard, crushed rock mass Medium hard, relatively crushed - crushed rock mass Medium soft, relatively intact - relatively crushed rock mass Soft, intact - relatively intact rock mass	350 ~ 251
	Soil mass: 1. Compacted or diagenesis clayed and sandy soils 2. Loess (Q_1, Q_2) 3. Typical calcareous or ferruginous cemented gravelly soil, cobble soil and block stone soil	
Ⅴ	Medium soft, crushed rock mass; Soft, relatively crushed - crushed rock mass; All extremely soft rock and all extremely crushed rock	≤250
	Typical Quaternary semi dry hard - hard plastic clayey material and slightly wet - wet gravelly soil, cobble soil, pebble and breccia soil and loess (Q_3, Q_4). Non-clayey material in loose structure; clayey material and loess in loose soft structure	
Ⅵ	Soft plastic clayey material, moist and saturated silty sand layer, soft soil, etc.	

Note: The above table is not applicable to classification of surrounding rock in special conditions, such as swelling surrounding rock and permafrost.

3.6.5 The physical and mechanical parameters of each class of surrounding rock should be obtained from laboratory or field tests. If there are no test data and preliminary classification, they can be selected from Table A.0.7-1 in Appendix A hereto. The shear peak strength of rock mass discontinuity may be selected from Table A.0.7-2 in Appendix A hereto. If measurement data are unavailable, physical and mechanical parameters of each class of surrounding soil may be obtained from Table A.0.7-3 in Appendix A hereto.

3.6.6 The self-stability of each class of surrounding rock may be assessed based on surrounding rock deformation measurement and theoretical calculation, or based on Table 3.6.6.

Table 3.6.6 Determination of self-stability of surrounding rock

Surrounding rock class	Self-stability
Ⅰ	For a span ≤20m, stable in long term, occasional falling off, no cave-in
Ⅱ	For a span of 10-20m, almost stable, possible localized falling off or small cave-in; For a span <10m, stable in long term, occasional falling off
Ⅲ	For a span of 10-20m, stable for several days to 1 month, possible small-medium cave-in; For a span of 5-10m, stable for several months, possible localized block displacement and small-medium cave-in; For a span < 5m, almost stable
Ⅳ	For a span >5m, generally no self-stability, possible loosening, deformation and small cave-in within several days to months, evolving to medium - large cave-in; primarily arch failure due to loosening if the tunnel is shallow; obvious plastic flow deformation and crushing failure if the tunnel is deep; For a span ≤5m, stable for several days to 1 month
Ⅴ	No self-stability; stable for several days if the span is 5m or less
Ⅵ	No self-stability

Notes:

1. Small cave-in: cave-in height < 3m or cave-in volume < $30m^3$.
2. Medium cave-in: cave-in height of 3 ~ 6m or cave-in volume of 30 ~ $100m^3$.
3. Large cave-in: cave-in height > 6m or cave-in volume > $100m^3$.

4 Overall Design

4.1 General

4.1.1 Tunnel design shall meet the requirements of highway planning, highway functions, land resources, eco-environment and sustainable development. The horizontal and vertical alignment, construction gauge, section of clear spacing, ventilation, lighting, traffic monitoring and other facilities shall be consistent with the class of the highway.

4.1.2 Tunnels shall be designed so that they are safe, practical to use, reasonably economical, technically advanced and of reliable quality.

4.1.3 The overall design of tunnels shall follow the principles below:

1 The tunnel location shall meet the needs of highway function and development and shall comply with requirement of overall route alignment.

2 On the basis of investigations of topography, landscape, geology, meteorology and social and cultural environment, a scheme shall be recommended after comprehensive comparisons of alternatives in tunnel orientation, horizontal and vertical alignment, portal location and connection conditions at both ends.

3 Construction gauge shall be determined based on highway class and design speed so as to establish an economically reasonable internal profile of the tunnel while meeting its functional and structural loading requirements.

4 Horizontal and vertical alignments inside and outside the tunnel shall be coordinated and be smooth so as to meet traffic safety and comfort requirements.

5 An appropriate ventilation method shall be selected and the size of ventilation, lighting, traffic monitoring, disaster prevention and rescue facilities determined according to tunnel length, layout plan, traffic volume and its composition, environmental protection and safe operation requirements.

6 A holistic approach shall be adopted to design the waterproofing and drainage both in and out of the tunnel, auxiliary channel, muck disposal, traffic engineering facilities, management facilities and environmental protection, taking into account the highway class, tunnel length, construction method, duration and operational requirements.

7 Consideration shall be given to interaction of the tunnel with adjacent existing and planned structures.

8 The overall design of the tunnel shall take account of energy saving, energy consumption reduction and easy maintenance and repair.

4.2 Selection of Tunnel Location

4.2.1 The proposed tunnel shall be located in a stable stratum, avoiding locations with extremely complex engineering geological and hydrogeological conditions or highly adverse geological zones. If this is unavoidable, practical and reliable engineering measures shall be provided.

4.2.2 For long and extra-long tunnels passing through mountains, different route schemes shall be developed with different elevations over the mountain based on geologic mapping and comprehensive geological exploration over a relatively large area. The tunnel alignment and location on plan shall be determined after a comprehensive technical and economic comparison taking account of route connection conditions at both ends, construction and operation conditions.

4.2.3 When the route passes alongside a river or mountain via tunnel, technical and economic comparisons shall be made between long tunnel option and the option of a group of short tunnels or the option of groups of bridges and tunnels as well as the option with a high slope on one side or shed tunnel option.

4.2.4 Portals should not be located in adverse geological zones such as zones with potential of landslide, collapse, talus, rockfall and debris flow, or low lying valleys with drainage problems or under an unstable cliff.

4.2.5 For a tunnel running along a reservoir, river or creek, the design elevation of the shoulder at tunnel portal shall at least 0.5m above the calculated flood level (including wave height and backwater height). Where bank slope collapse due to prolonged immersion has an adverse impact on tunnel stability, appropriate engineering measures shall be taken.

4.2.6 The frequency standard for tunnel design flood level can be obtained from Table 4.2.6. When observed flood level is higher than standard value, the tunnel shall be designed for the observed flood level.

Table 4.2.6 Flood frequency standard of tunnel design water level

Tunnel type	Highway class			
	Expressway and Class-1 highway	Class-2 highway	Class III highway	Class-4 highway
Extra-long tunnel	1/100	1/100	1/50	1/50
Long tunnel	1/100	1/50	1/50	1/25
Short and medium tunnels	1/100	1/50	1/25	1/25

4.3 Tunnel alignment design

4.3.1 The horizontal alignment of a tunnel shall be determined by geology, terrain, route orientation, ventilation and other factors. Where a curve is required, widened circular curves with superelevation should not be adopted. The minimum radius of a circular curve with no superelevation for a tunnel shall be in accordance with Table 4.3.1. Where the horizontal alignment of a tunnel requires a circular curve with superelevation, the value of this superelevation should not be greater than 4.0%. If the design speed is 20km/h, the radius of the circular curve should not be less than 250m. The sight distance for each lane in the tunnel shall meet the sight distance requirement specified in the current *Code for Design of Highway Route* (JTG D20).

Table 4.3.1 Minimum radius of circular curve without superelevation (m)

Camber	Design speed (km/h)					
	120	100	80	60	40	30
≤2.0%	5500	4000	2500	1500	600	350
>2.0%	7500	5250	3350	1900	800	450

4.3.2 Expressway and Class-1 highway tunnels shall be designed as twin tunnels with one tunnel accommodating up-bound traffic and the down-bound traffic, and the two tunnels should be separated. Other forms of arrangement may be adopted for the following scenarios:

1 Twin tunnels with narrow pillar are permitted for short and medium tunnels with narrow space at portals, at bridge-tunnel connection, for contiguous tunnel group, where there are constraints due to nearby buildings or if there is a need to reduce land occupation outside the tunnel.

2 Twin-arch tunnels are permitted for short tunnels for which route development is very difficult as a result of narrow space at portals or constraints due to nearby buildings.

3 Branching-out tunnels are permitted at portals of long and extra-long tunnels, for bridge-tunnel connections where the space is narrow at portals or where there are special requirements.

4.3.3 The clearance between two separated tunnels should be such that the two tunnel structures do not affect each other adversely, taking into account route connection conditions at portals, geological conditions of surrounding rock, cross-section shape and dimensions, structural design, construction method, requirements of construction duration and other factors. The clearance between the two tubes should be 0.8-2.0 times the excavated width. A smaller value can be adopted if overall conditions of the surrounding rock are good; and a greater value should be adopted if they are poor. For tunnels with different span widths, the clearance shall be controlled by the larger span width.

4.3.4 The unidirectional gradient should be adopted for longitudinal gradient in a tunnel while bidirectional gradient is permitted for long and extra-long tunnels with recharged groundwater. The minimum radius and length of vertical curves in a tunnel shall be in accordance with Table 4.3.4.

Table 4.3.4 Minimum radius and length of vertical curves (m)

Design speed (km/h)	120	100	80	60	40	30	20
Minimum radius of convex vertical curve	17000	10000	4500	2000	700	400	200
Minimum radius of concave vertical curve	6000	4500	3000	1500	700	400	200
Minimum length of vertical curve	100	85	70	50	35	25	20

4.3.5 The vertical alignment in a tunnel shall take into account traffic safety, the capacity of operational ventilation, construction operation and drainage requirements, with a minimum longitudinal gradient of 0.3% and a maximum longitudinal gradient of 3%. A tunnel shorter than 100m may be exempt from this restriction. The maximum longitudinal gradient in medium and short tunnels on an expressway or Class-1 highway with topographical or other limitations may be increased appropriately after technical and economic justification and traffic safety assessment, but should not be greater than 4%.

4.3.6 The alignment of the route linking to the tunnel and the tunnel alignment shall be coordinated so that horizontal and vertical alignments within 3s travelling length at design speed inside the portal are consistent with those within 3s travelling length at design speed outside the portal. In difficult sections, horizontal curves inside and outside the portal may be designed as transition curves after technical and economic justification, provided that alignment guidance facilities are enhanced.

4.3.7 Horizontal and vertical alignment technical specifications for tunnels at intervals of less than 100m should be designed as a whole.

4.4 Clear section design

4.4.1 Fig. 4.4.1 gives the construction gauge for highway tunnels, into which no other structural elements shall encroach. The construction gauge width of two-lane tunnels on highways shall not be less than the basic width shown in Table 4.4.1, and shall meet the following requirements:

1 Construction gauge height: 5.0m for expressway, Class I and Class-2 highways; 4.5m for Class III and Class-4 highways.

2 The maintenance access or sidewalk, if provided, should include width allowance. If it is not provided, a width allowance not less than 0.25m shall be provided.

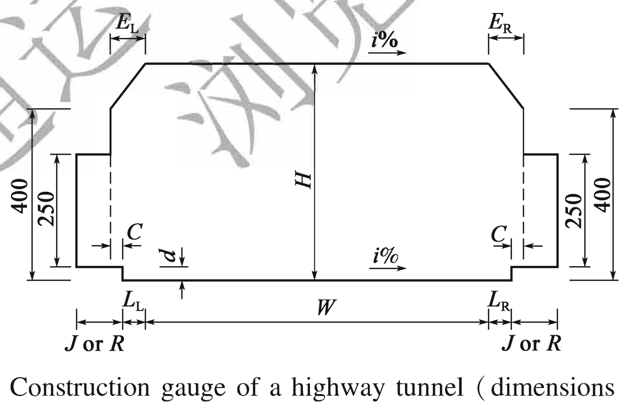


Fig. 4.4.1 Construction gauge of a highway tunnel (dimensions given in cm)

H—construction gauge height; *W*—carriageway width; L_L—left lateral width; L_R—right lateral width; *C*—width allowance; *J*—maintenance access width; *R*—sidewalk width; *d*—height of maintenance access or sidewalk; E_L—left top width of construction gauge, including width allowance C; E_R—right top width of construction gauge, including width allowance *C*.

Notes:

If $L_L \leqslant 1m$, then $E_L = L_L$; if $L_L > 1m$, then $E_L = 1m$;

If $L_R \leqslant 1m$, then $E_R = L_R$; if $L_R > 1m$, then $E_R = 1m$.

3 The cross fall of pavement in a tunnel shall be designed as one-sided slope if the tunnel is to carry unidirectional traffic, and may be designed as two-sided slope if it is to carry bidirectional traffic. The cross fall in the tunnel should be 1.5% -2.0% and should be consistent with the pavement cross fall outside the tunnel.

4 If one-sided slope is adopted, the construction gauge bottom line shall coincide with the pavement. If two-sided slope is adopted, it shall be placed horizontally on the highest point of the pavement.

5 A tunnel of a single-lane Class-4 highway shall be constructed to the standard for that of a two-lane Class-4 highway.

Table 4.4.1 Cross-section elements and basic width of construction gauge for a two-lane highway tunnel (m)

Highway class	Design speed (km/h)	Travel lane width W	Lateral width		Width allowance C	Maintenance access width J or sidewalk width R		Basic width of construction gauge
			Left side L_L	*Right side* L_R		Left side	Right side	
Expressway and Class-1 highway	120	3.75×2	0.75	1.25	0.50	1.00	1.00	11.50
	100	3.75×2	0.75	1.00	0.25	0.75	0.75	10.75
	80	3.75×2	0.50	0.75	0.25	0.75	0.75	10.25
	60	3.50×2	0.50	0.75	0.25	0.75	0.75	9.75
Class-2 highway	80	3.75×2	0.75	0.75	0.25	1.00	1.00	11.00
	60	3.50×2	0.50	0.50	0.25	1.00	1.00	10.00
Class III highway	40	3.50×2	0.25	0.25	0.25	0.75	0.75	9.00
	30	3.25×2	0.25	0.25	0.25	0.75	0.75	8.50
Class-4 highway	20	3.00×2	0.50	0.50	0.25			7.50

Note: For a three-lane or four-lane tunnel, the widths beyond those of the additional lane(s) are the same as presented in Table 4.4.1. The width of each additional lane shall not be less than 3.5m.

4.4.2 A maintenance access shall be provided on each side of the pavement in an expressway or Class-1 highway tunnel. A sidewalk, also serving as maintenance access, shall be provided on each side of the pavement in a Class II or III highway tunnel. The width of the maintenance access or sidewalk shall be in accordance with Table 4.4.1. A maintenance access or sidewalk is not required on the left side of travel direction in a twin-arch tunnel. Nor is it required in a Class-4 highway tunnel. However, a width allowance not less than 0.25m shall be reserved; the width allowance shall not be less than 0.5m where the design speed is higher than 100km/h. The height of the maintenance access or sidewalk may range from 250mm to 800mm, taking account of the following considerations:

1 The safety of maintenance personnel or pedestrians when they are walking;

2 Provide sufficient space to accommodate cables, water supply pipelines and drains.

3 The convenience of drivers and passengers getting fire-fighting equipment in case of emergency.

4.4.3 The internal clear section shall meet the following requirements:

1 Provide sufficient space for construction gauge and reserve an allowance not less than 50mm;

2 Provide sufficient space for in-tunnel finishes;

3 Provide sufficient space for traffic engineering and auxiliary facilities including ventilation, lighting, fire protection, monitoring and signs.

4 The cross-section shape should be beneficial to the stability and structural force of the surrounding rock mass.

5 Reference can be made to Appendix B for the cross-section shape and dimensions of a tunnel internal profile.

4.4.4 Side drain in the tunnel shall be arranged on both sides of the carriageway, taking account of maintenance access, lateral width and width allowance.

4.4.5 Where hard shoulder is not provided in an extra-long or long tunnel, or the hard shoulder is less than 2.5m wide, an emergency stop zone is required in a single two-lane tunnel; an emergency stop zone should be provided in a single three-lane tunnel; and an emergency stop zone is not required in a single four-lane tunnel.

4.4.6 The emergency stop zone shall be designed as follows:

1 The emergency stop zone shall be widened to at least 3.0m width toward the right of travel direction, and the sum of its width and the right lateral width (L_R) shall not be less than 3.5m.

2 The emergency stop zone should not be less than 50m long, of which an effective length shall not be less than 40m.

3 Cross slope of the emergency stop zone may be taken as 0 ~ 1.0%.

4 The spacing of emergency stop zones in unidirectional tunnel shall not be less than 750m and not more than 1,000m at distance.

5 Emergency stop zones in a bidirectional tunnel shall be arranged on both sides in a staggered manner. The spacing on the same side may be 800 ~ 1,200m and shall not be more than 1,500m in any case.

The components of the construction gauge for emergency stop zones are illustrated in Fig. 4.4.6. The specific dimensions shall be in accordance with 4.4.1 and 4.4.2 herein.

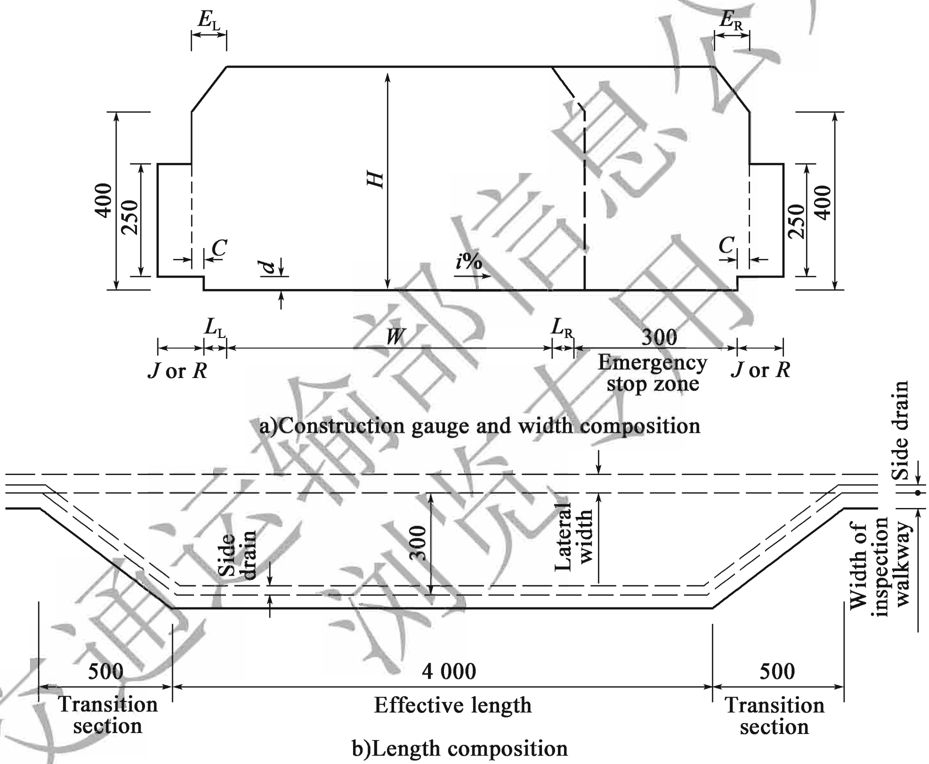


Fig. 4.4.6 Construction gauge, width and length of emergency stop zone (dimensions given in cm)

4.4.7 In a tunnel where maintenance access or sidewalk is not provided, refugees shall be arranged on both sides in a staggered manner. The spacing of refugees on the same side should not be greater than 500m. The refuge shall not be less than 1.5m in width, 2.2m in height or 0.75m in depth.

4.4.8 A short tunnel on a four-lane expressway, an independent cut-and-cover tunnel or shed tunnel, a medium or short tunnel at access to the city should be as wide as the subgrade.

4.4.9 A transition section not shorter than a 3s travelling length at design speed or 50m shall be provided at the portals in the approach section to smoothen the transition between cross-sections.

4.5 Cross passage and parallel adit

4.5.1 Between the twin tunnels carrying up and down traffic, cross passages shall be provided as follows:

1 For a pedestrian cross passage, the clearance width shall not be less than 2.0m and the clearance height not less than 2.5m. For a vehicle cross passage, the clearance width shall not be less than 4.5m and the clearance height shall be consistent with that of the main tunnel. Fig. 4.5.1 illustrates the cross-section construction gauge of a cross passage.

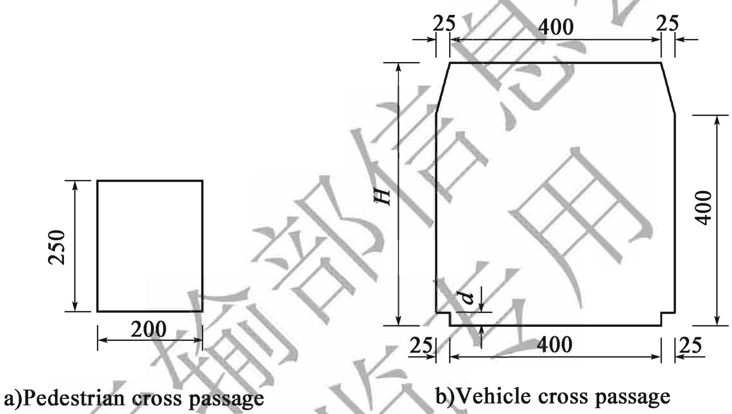


Fig. 4.5.1 Cross-section construction gauge of a cross passage (dimensions given in cm)

2 The spacing of pedestrian cross passages should be 250m and shall not be greater than 350m.

3 The spacing of vehicle cross passages should be 750m and shall not be greater than 1,000m; vehicle cross passages are not required for medium and short tunnels.

4 The curb height of a vehicle cross passage, d, should be consistent with the height of the maintenance access on the left side of travel direction in the tunnel.

4.5.2 For an extra-long tunnel with a single tunnel carrying bidirectional traffic, parallel passages should be provided as follows:

1 They should be provided next to the main tunnel along the entire length of its axis; and may be provided locally if there are constraints.

2 Their cross-section shall not be smaller than that of the pedestrian cross passage.

3　Pedestrian cross passages shall be provided between the parallel passages and main tunnel; the spacing should be 250 ~ 500m.

4　The drain invert level should be 0.2 ~ 0.6m below that of the main tunnel.

4.5.3　Where topographical conditions allow, a cross passage leading to the ground surface may be added.

4.5.4　For long and extra-long twin tunnels, a connecting passage shall be provided in an appropriate location outside the tunnel.

4.6　Tunnel construction monitoring and measurement and advance geological forecast

4.6.1　As part of tunnel design, schemes for tunnel construction monitoring and measurement and advance geological forecast during tunnel construction shall be developed considering geological conditions, construction methods, shoring methods, surrounding environment, etc.

4.6.2　The schemes for tunnel construction monitoring and measurement and advance geological forecast during tunnel construction shall define the purpose, content, requirements and information to be obtained.

4.6.3　During tunnel construction, observational approach and information-based construction process shall be implemented by making timely adjustments to excavation method and support parameters based on relevant information from tunnel construction monitoring and measurement and advance geological forecast.

4.7　Construction plan

4.7.1　Construction plan shall be prepared as part of tunnel design to address construction duration, construction method, division of work zones, temporary facilities, access road, muck bank, sewage treatment, tunnel construction monitoring and measurement scheme, advance geological forecast requirements, etc. Preparation of the construction plan shall follow the principles below:

1　Reasonable construction methods and schedule shall be determined considering tunnel length, layout, cross-section, duration, geologic conditions, natural conditions, etc.

2 Work zones shall be divided considering the tunnel's longitudinal gradient, engineering geological and hydrogeological conditions, muck bank, conditions for access road construction, cut-fill balance, etc.

3 The purpose, role and necessity of providing auxiliary channels shall be justified technically and economically, taking into account engineering geology, hydrogeology, construction method, construction operation and operational ventilation scheme.

4 Required technical specifications of key construction machinery and associated facilities shall be defined according to the size, geological conditions, etc. of the proposed tunnel.

5 Temporary facilities shall be arranged according to project size and portal conditions so that they satisfy construction needs, minimize disruption and damage to surroundings and facilitate reinstatement.

5 Building Materials

5.1 General

5.1.1 For building materials commonly used in tunnel works, the following strength grades may be selected:

1 Concrete: C50, C40, C30, C25, C20, C15;

2 Stone: MU100, MU80, MU60, MU50, MU40;

3 Cement mortar: M25, M20, M15, M10, M7.5;

4 Shotcrete: C40, C30, C25, C20;

5 Concrete block: MU30, MU20;

6 Rebar: HPB300, HRB400, HRB500.

5.1.2 The strength grade of building material at each position of tunnel works shall not be lower than that listed in Tables 5.1.2-1 and 5.1.2-2.

Table 5.1.2-1 Strength grades of building materials for lining and pipe trench

Position	Type of material			
	Concrete	Rubble concrete	Reinforced concrete	Shotcrete
Arch ring	C20	—	C25	C20
Side wall	C20	—	C25	C20
Arch invert	C20	—	C25	C20

continued

Position	Type of material			
	Concrete	Rubble concrete	Reinforced concrete	Shotcrete
Floor	C20	—	C25	—
Arch invert filling	C15	C15	—	—
Ditch and cable trough	C25	—	C25	—
Cover plates of ditch and cable trough	—	—	C25	—

Table 5.1.2-2 Strength grades of portal building materials

Position	Type of material			
	Concrete	Reinforced concrete	Rubble concrete	Masonry
End wall	C20	C25	C15	M10 cement stone pitching, block stone or concrete masonry facing
Top cap	C20	C25	—	M10 cement mortar coarse dressed stone
Wing wall and portal retaining wall	C20	C25	C15	M10 cement stone pitching
Side and interception drain	C15	—	—	M7.5 cement stone pitching
Revetment	C15	—	—	M7.5 cement stone pitching

Notes:

1. C20 shotcrete can be used for slope protection.
2. In areas where the average temperature in the coldest month is less than -15℃, the strength grade of cement mortar in the table shall be increased by one level.

5.1.3 Building materials shall be selected and used as follows:

1 They shall have the required structural strength and durability, as well as the required freezing resistance, resistance to seepage and erosion resistance.

2 Where there is aggressive water, the concrete and cement mortar used shall be made of erosion-resistant cement and aggregate, and the required erosion resistance depends on the erosional features of water.

3 In areas where the average temperature in the coldest month is below −15℃ and for tunnels susceptible to freezing damage, the strength grade of concrete shall be increased properly.

5.1.4 Materials used for concrete and masonry shall conform to the following provisions in addition to relevant national standards:

1 Alkali-reactive aggregate shall not be used for concrete batching.

2 The strength grade of concrete for reinforced concrete structures shall not be lower than C25. The strength grade of concrete for prestressed concrete structures shall not be lower than C30.

3 In reinforced concrete members, the technical conditions for rebar shall conform to the provisions of the *Steel for the Reinforcement of Concrete—Part 1: Hot Rolled Plain Bars* (GB 1499.1) and the *Steel for the Reinforcement of Concrete—Part 2: Hot Rolled Ribbed Bars* (GB 1499.2).

4 The strength grade of rubble shall not be below MU40. The strength grade of block stone shall not be below MU60. The strength grade of block stone or dressed stone shall not be lower than MU80. Stone that is cracked and susceptible to weathering shall not be used.

5 The amount of rubble added to rubble concrete shall not be more than 30% of the total volume.

6 The strength grades of concrete blocks shall not be lower than MU20.

5.1.5 Materials for shotcrete and rockbolt support shall conform to the following provisions in addition to relevant provisions in 5.1.1~5.1.4 herein:

1 Portland cement or ordinary Portland cement is preferred, and Portland slag cement may also be used, to make shotcrete.

2 Coarse aggregate shall be made of hard and durable crushed stones or pebbles, and not be made of alkali-reactive materials. The particle size of stone in shotcrete should not be greater than 16mm. The particle size of coarse aggregate in steel fiber reinforced shotcrete should not be greater than 10mm. Aggregate grading should be continuous grading. Fine aggregate shall be composed of hard and durable medium or coarse sand; and should have a fineness modulus of more than 2.5. The moisture content in sand should be controlled within a range of 5% to 7%.

3 The mortar rockbolt should be made of HRB400 or HRB500 hot rolled ribbed bars.

4 Hollow rockbolts should be made of seamless steel tubes for Q345 structures. The elongation at break (A) of rockbolt shall not be smaller than 16% and shall conform to the current *Seamless Steel Tubes for Structural Purposes* (GB/T 8162).

5 The composite materials of hollow rock bolts shall conform to 3 and 4 of this clause.

6 The rockbolt plates should be made of Q235 hot rolled steel plates.

7 Reinforcement mesh may be made of HPB300 hot rolled plain bars.

5.1.6 Additives may be added to concrete and shotcrete as needed and shall have performance required as follows:

1 Have little effects on the strength of concrete and the adhesion of concrete to surrounding rocks, and cause no corrosion to concrete and steel;

2 Have little influence on the setting time of concrete (except for accelerator and retarder);

3 Be not susceptible to absorb moisture and be easy for storage; and do not pollute the environment.

5.1.7 Steel fibers in steel fiber reinforced shotcrete should be made of plain carbon steel and meet the following requirements:

1 They should have square or circular sections with an equivalent diameter of 0.3 ~ 0.5mm.

2 The length should be 20 ~ 25mm and the length-to-diameter ratio should be 40 ~ 60.

3 The tensile strength shall not be less than 380MPa. They shall be free of oil stain and obvious rust.

5.1.8 Steel rib supports used for primary support should be steel lattice frame or structural steel rib support, and also can be made of steel tubes or steel rails. For characteristic parameters of structural steel, see Appendix C hereto.

5.1.9 Pavement materials in tunnels shall conform to relevant provisions of the current *Specifications for Design of Highway Asphalt Pavement* (JTG D50) and *Specifications for Design of Highway Cement Concrete Pavement* (JTG D40).

5.1.10 Waterproof materials in tunnels shall conform to the provisions of the current *Technical Code for Waterproofing of Underground Works* (GB 50108). Waterproof materials may be groutable waterstop materials, waterproofing membranes, embedded waterstops, back bonded waterstops, drainage pipes, waterproof concrete, etc.

5.1.11 Grouting materials shall meet the following requirements:

1 Grout shall be nontoxic, odourless and environment-friendly.

2 Grout shall have low viscosity, good flowability and strong groutability. Its setting time may be controlled as required.

3 Solidified grout shall have good stability and can meet the service life requirements of grouting works.

4 Grout shall be noncorrosive to grouting equipment, pipelines and concrete structures and easy to clean.

5.1.12 Waterproofing membranes should be made of ethylene-vinyl acetate copolymers (EVA), ethylene-vinyl acetate and bitumen copolymers (ECB), polyethylene (PE) or other materials with similar properties. Also, new waterproof materials can be selected and used, including pre-formed fully bonded waterproofing membranes or vertical waterproof and drainage boards. Membranes and their adhesives shall have good durability and good resistance to water, puncture, corrosion and bacteria.

5.1.13 Non-woven fabrics in tunnel should be polypropylene needled nonwoven geotextiles.

5.1.14 Circumferential and longitudinal drainage pipes shall have certain strength and good water permeability; and can be laid in a concave-convex manner along wall surface.

5.2 Material properties

5.2.1 The unit weight of common building materials shall conform to Table 5.2.1.

Table 5.2.1 Standard or calculated unit weight of building materials (kN/m^3)

Description	Concrete	Rubble concrete	Reinforced concrete (reinforcement ratio ≤ 3%)	Shotcrete	Steel	Stone pitching	Mortar block stone	Mortar coarse dressed stone
Unit weight	23	23	25	22	78.5	22	23	25

5.2.2 The standard strength of concrete shall conform to Table 5.2.2.

Table 5.2.2 Standard strength of concrete (MPa)

Type of strength	Strength grade of concrete							
	C15	C20	C25	C30	C35	C40	C45	C50
Axial compressive strength, f_{ck}	10	13.4	16.7	20.1	23.4	26.8	29.6	32.4
Flexural compressive strength, f_{cmk}	11	15	18.5	22	26	29.5	32.5	36
Axial tensile strength, f_{ctk}	1.27	1.54	1.78	2.01	2.2	2.39	2.51	2.64

Notes:

1. Concrete shall be poured vertically. When the height of concrete poured in one continuous pour is greater than 1.5m, the strength in the table shall be multiplied by 0.9.
2. For calculation of the cast-in-place reinforced concrete member subject to axial compression, if the circumference or diameter of the cross-section is smaller than 30cm, the strength in the table shall be multiplied by 0.8.

5.2.3 The design strength of concrete shall conform to Table 5.2.3.

Table 5.2.3 Design strength of concrete (MPa)

Type of strength	Strength grade of concrete							
	C15	C20	C25	C30	C35	C40	C45	C50
Axial compressive, f_{cd}	7.2	9.6	11.9	14.3	16.7	19.1	21.1	23.1
Flexural compressive, f_{cmd}	8.5	11	13.5	16.5	19	21.5	24	27.5
Axial tensile, f_{ctd}	0.91	1.10	1.27	1.43	1.57	1.71	1.80	1.89

5.2.4 The ultimate strength of concrete shall conform to Table 5.2.4.

Table 5.2.4 Ultimate strength of concrete (MPa)

Type of strength	Strength grade of concrete							
	C15	C20	C25	C30	C35	C40	C45	C50
Compressive strength R_a	12.0	15.5	19.0	22.5	26.3	29.5	33.6	36.5
Flexural compressive strength, R_w	15.0	19.4	23.6	28.1	32.9	36.9	42	45.6
Tensile strength, R_l	1.4	1.7	2.0	2.2	2.5	2.7	2.9	3.1

Note: The ultimate compressive strength of concrete with rubbles may be used as those given in the table.

5.2.5 The modulus of elasticity of concrete shall conform to Table 5.2.5. The shear modulus of concrete may be the products of multiplying the values listed in the table by 0.43. The Poisson's ratio may be 0.2.

Table 5.2.5 Modulus of elasticity of concrete (GPa)

Strength grade of concrete	C15	C20	C25	C30	C35	C40	C45	C50
Modulus of elasticity, E_c	22	25.5	28	30	31.5	32.5	33.5	34.5

5.2.6 The design strength of shotcrete shall conform to Table 5.2.6-1. The modulus of elasticity of shotcrete shall conform to Table 5.2.6-2.

Table 5.2.6-1 Design strength of shotcrete (MPa)

Type of strength	Strength grade of shotcrete				
	C20	C25	C30	C35	C40
Axial compressive	9.6	11.9	14.3	16.7	19.1
Flexural compressive	11.0	13.5	16.5	-	-
Tensile	1.1	1.27	1.43	1.57	1.71

Notes:

1. The strength of shotcrete is the product of multiplying its ultimate compressive strength by 0.95. Its ultimate compressive strength is obtained by standard test, in which test cubes with a side length of 10cm are fabricated by shotcrete slab cutting method and cured under standard conditions for 28d.
2. Cohesion may be determined by splitting method or by directly pulling the shotcreted layer.

Table 5.2.6-2 Modulus of elasticity of shotcrete (GPa)

Strength grade of shotcrete	C20	C25	C30	C35	C40
Modulus of elasticity	23	26	28	30	31.5

5.2.7 For C20 shotcrete, its ultimate axial compressive strength may be 15MPa. Its ultimate flexural compressive strength may be 18MPa. Its ultimate tensile strength may be 1.3MPa. The bonding strength of shotcrete to surrounding rocks shall not be lower than 0.8MPa for Classes I and II surrounding rocks and not be lower than 0.5MPa for Class III surrounding rocks.

5.2.8 The compressive strength of shotcrete at 1 day shall not be lower than 5MPa. The steel fiber reinforced shotcrete shall have a design strength grade of not lower than C25, a tensile strength of not lower than 2MPa, and a flexural strength of not lower than 6MPa.

5.2.9 The design tensile and compressive strength of rebars shall conform to Table 5.2.9.

Table 5.2.9 Design tensile and compressive strength of rebar (MPa)

Rebar grade	HPB300	HRB400	HRB500
Design tensile strength, f_y	270	360	435
Design compressive strength, f'_y	270	360	410

5.2.10 The total elongation of rebar under the modulus of elasticity and the maximum force shall conform to Table 5.2.10.

Table 5.2.10 Total elongation under the modulus of elasticity and the maximum force of rebar

Rebar grade	HPB300	HRB400	HRB500
Modulus of elasticity, E_s (MPa)	210	200	200
Total elongation under the maximum force (%)	10	7.5	7.5

5.2.11 The standard yield strength, standard ultimate strength and standard tensile or compressive strength of rebar may be adopted as per Table 5.2.11.

Table 5.2.11 Standard yield strength, standard ultimate strength and standard tensile or compressive strength of rebar (MPa)

Rebar grade	HPB300 (d = 6 ~ 22mm)	HRB400 (d = 6 ~ 50mm)	HRB500 (d = 6 ~ 50mm)
Standard yield strength, f_{yk}	300	400	500
Standard ultimate strength, f_{stk}	420	540	630
Standard tensile or compressive strength, R_g	300	400	500

Note: d in the table means the diameter of rebar.

5.2.12 The ultimate strength of stone shall conform to Table 5.2.12.

Table 5.2.12 Ultimate strength of stone (MPa)

Type of strength	Strength grade					
	MU100	MU80	MU60	MU50	MU40	MU30
Axial compressive	72.0	57.6	43.2	36.0	28.8	21.6
Flexural tensile	6.0	4.8	3.6	3.0	2.4	1.8

5.2.13 The permissible stresses on stone and concrete block masonries under axial and eccentrically compressed shall conform to Table 5.2.13.

Table 5.2.13 Permissible stresses on stone and concrete block masonries under axial and eccentrically compressed (MPa)

Type of masonry	Strength grades of stone and concrete blocks	Strength grade of cement mortar		
		M20	M10	M7.5
Rubble masonry	MU100	3.0	2.2	1.9
	MU80	2.7	2.0	1.7
	MU60	2.3	1.85	1.5

continued

Type of masonry	Strength grades of stone and concrete blocks	Strength grade of cement mortar		
		M20	M10	M7.5
	MU50	2.1	1.6	1.3
Block stone masonry	MU100	5.6	4.9	—
	MU80	4.7	4.1	—
	MU60	3.8	3.2	—
	MU50	3.3	2.8	—
Coarse dressed stone masonry	MU100	7.1	5.0	—
	MU80	6.0	4.8	—
	MU60	4.9	4.1	—
	MU40	3.7	3.4	—
Concrete block masonry	MU30	5.6	4.7	—
	MU20	4.4	3.6	—

Notes:

1. The permissible stresses on other masonries under compression between the strength grades of stone and cement mortar listed in the table may be determined by interpolation between the specified values.
2. If the height, h, of a concrete block exceeds 20cm, the permissible stress on concrete block masonry shall be the product of multiplying the value in the table by an increase coefficient, c. When $h \leqslant 40$cm, $c = 0.6 + 0.02h$. When $h > 40$cm, $c = 1.2 + 0.005h$. If c is actually greater than 1.7, it will be taken as 1.7.
3. If fine dressed stone and semi-fine dressed stone masonry *are* used for special needs, *then* the permissible stresses under compression shall be the product of permissible stress on coarse dressed stone masonry under compression by increase coefficients of 1.43 and 1.14 respectively. However, the increased permissible stress under compression shall not be greater than a half of the ultimate compressive strength.

5.2.14 The design compressive strength of mortar masonry shall conform to the following requirements:

1 The design compressive strength of mortar masonry of precast concrete block shall comply with Table 5.2.14-1.

Table 5.2.14-1 Design compressive strength of mortar masonry of precast concrete block (MPa)

Strength grade of concrete	Strength grade of mortar			
	M20	M15	M10	M7.5
C40	8.10	6.92	5.74	5.15
C30	7.01	5.99	4.96	4.46
C20	5.73	4.89	4.06	3.64
C15	4.96	4.24	3.51	3.15

2 The design compressive strength of mortar masonry of block stone shall comply with Table 5.2.14-2.

Table 5.2.14-2 Design compressive strength of mortar masonry of block stone (MPa)

Strength grade of stone	Strength grade of mortar			
	M20	M15	M10	M7.5
MU100	8.54	7.29	6.04	5.43
MU80	7.64	6.52	5.41	4.85
MU60	6.61	5.65	4.68	4.20
MU50	6.04	5.16	4.28	3.84
MU40	5.40	4.61	3.83	3.43

Note: For stone masonry, the strength shall be the product the values in the table by respective coefficients. The coefficient is 1.5 for fine dressed stone masonry, 1.3 for semi-fine dressed stone masonry, 1.2 for coarse dressed stone masonry and 0.8 for dry jointed stone masonry.

3 The design compressive strength of mortar masonry of rubble shall comply with Table 5.2.14-3.

Table 5.2.14-3 Design compressive strength of rubble masonry (MPa)

Strength grade of stone	Strength grade of mortar			
	M20	M15	M10	M7.5
MU100	2.0	1.71	1.41	1.27
MU80	1.79	1.53	1.26	1.14
MU60	1.55	1.32	1.09	0.98
MU50	1.41	1.21	1.00	0.90
MU40	1.26	1.08	0.89	0.80

5.2.15 The ultimate strength of masonry shall conform to Table 5.2.15.

Table 5.2.15 Ultimate strength of masonry (MPa)

Strength grade of mortar	Compressive strength R_a				Shear strength R_j
	Rubble	Block stone	Coarse dressed stone	Concrete masonry	
M7.5	3.0	—	—	—	0.35
M10	3.5	5.5	8.0	5.5	0.40
M15	4.0	6.0	9.0	6.0	0.50

Note: If the height, h, of a concrete block exceeds 20cm, the ultimate compressive strength of concrete block masonry given in the table shall be multiplied by an increase coefficient, c. When $h \leq 40$cm, $c = 0.6 + 0.02h$. When $h >$ 40cm, $c = 1.2 + 0.005h$. If c is actually greater than 1.7, it will be taken as 1.7.

5.2.16 The compressive modulus of elasticity of masonry shall be within a range of 10 ~ 15GPa. The shear modulus of elasticity of masonry should be 0.4 times the compressive modulus of elasticity.

5.3 Properties of waterproof and drainage materials

5.3.1 Technical specifications of waterproofing layers may be used as those specified in Tables 5.3.1-1 and 5.3.1-2.

Table 5.3.1-1 Technical specifications of common waterproofing layers

Item		Unit	Specifications	
			Polyethylene (PE)	Ethylene-vinyl acetate copolymer (EVA)
Tensile strength at break ≥		MPa	18	18
Elongation at break ≥		%	600	650
Tear strength ≥		kN/m	95	100
Watertightness (0.3MPa, 24h)			Without leakage	Without leakage
Low temperature bendability ≤		℃	−35℃, without crack	−35℃, without crack
Thermal Expansion	Expansion ≤	mm	2	2
	Contraction ≤	mm	6	6
Hot air aging(80℃, 168h)	Tensile strength at break ≥	MPa	15	16
	Elongation at break ≥	%	550	600
Alkali resistance ($Ca(OH)_2$ saturated solution, 168h)	Tensile strength at break ≥	MPa	16	17
	Elongation at break ≥	%	550	600
Artificial weathering	Retention rate of tensile strength at break ≥	%	80	80
	Retention rate of elongation at break ≥	%	70	70
Puncture strength ≥		N	300	300

Table 5.3.1-2 Technical specifications of pre-applied fully bonded waterproofing layers

Item	Unit	Specifications	
		P type	PY type
Soluble matter content ≥	g/m^2	—	2900
Tension ≥	N/50mm	500	800
Elongation at break of membrane ≥	%	400	—

continued

Item		Unit	Specifications	
			P type	PY type
Elongation at the maximum tension ⩾		%	—	40
Tear strength of screw rod ⩾		N	400	200
Impact property			Diameter: (10 ±0.1) mm, without leakage	
Static load			20Kg, without leakage	
Thermo tolerance			70℃, without displacement, flow or dripping for 2h	
Thermal aging (70℃, 168h)	Tension retention rate ⩾	%	90	
	Elongation retention rate ⩾	%	80	
Low temperature bend			−25℃, without crack	—
Flexibility at low temperature			—	−25℃, without crack
Permeability ⩽		Sheet	—	2
Anti-water channeling			0.6MPa, without water channeling	
Peeling strength with concrete placed later	Without treatment ⩾	N/mm	2.0	
	Surface contaminated by cement flour ⩾		1.5	
	Surface contaminated by silt ⩾		1.5	
	Ultraviolet radiation aging ⩾		1.5	
	Thermal aging ⩾		1.5	
Peeling strength with concrete placed later after immersion into water ⩾		N/mm	1.5	

Note: The thickness of high-molecular main material in a P type product shall not be less than 0.7mm, and the full thickness of a membrane shall not be less than 1.2mm. The thickness of a PY type membrane shall not be less than 4mm.

5.3.2 The technical specifications of nonwoven geotextiles used in tunnels may be used as those specified in Table 5.3.2.

Table 5.3.2 Technical specifications of nonwoven geotextiles used in tunnels

Item		Unit	Specifications			Remarks
Mass per unit area		g/m²	300	400	500	Deviation: ±5%
Breaking strength	Longitudinal, transverse	kN/m	⩾15	⩾20	⩾25	
Elongation at break	Longitudinal, transverse	%	⩾40			
CBR puncture strength		kN	⩾2.9	⩾3.9	⩾5.3	
Tear strength	Longitudinal, transverse	kN	⩾0.42	⩾0.56	⩾0.70	
Equivalent aperture O_{90} (O_{95})		mm	0.05 ~ 0.2			
Vertical permeability		cm/s	K × (10^{-1} ~ 10^{-3})			K = 1.0 ~ 9.9
Thickness		mm	⩾2.2	⩾2.8	⩾3.4	

5.3.3 Technical specifications of rubber waterstops used in tunnels may be used as those specified in Table 5.3.3.

Table 5.3.3 Technical specifications of rubber waterstops used in tunnels

Item			Unit	Specification
Hardness			Shore A	60 ± 5
Tensile strength ≥			MPa	10
Elongation at break ≥			%	380
permanent compression deformation		70℃, 24h	%	30
		23℃, 168h	%	20
Tear strength ≥			kN/m	30
Brittleness temperature ≤			℃	−45
Hot air aging	70℃, 168h	Hardness change ≤	Shore A	6
		Tensile strength ≥	MPa	9
		Elongation at break ≥	%	320
Alkali resistance	Saturated solution of calcium hydroxide at 23℃ for 168h	Hardness change ≤	Shore A	6
		Tensile strength ≥	MPa	9
		Elongation at break ≥	%	320
Ozone aging 50×10^{-8}: 20%, 40℃, 48h				Without crack
Rubber-metal bonding				Rubber damage

5.3.4 The required appearance quality, size deviation and physical properties of rubber waterstops shall conform to the *Polymer Water-proof Materials—Part 2: Waterstop* (GB 18173.2).

5.3.5 The properties of drainage pipes shall comply with the relevant provisions of the *Flexible Permeable Hose* (JC 937) and the *Polyethylene Structure Wall Pipeline System for Underground Usage—Part 1: Polyethylene Double Wall Corrugated Pipes* (GB/T 19472.1).

6 Load

6.1 General

6.1.1 Loads on tunnel structures shall be classified as given in Table 6.1.1.

Table 6.1.1 Classification of loads on tunnel structures

S/N	Load classification		Description
1	Permanent loads		Rock load
2			Soil pressure
3			Dead weight of structure
4			Superimposed dead load of structure
5			Influence of concrete shrinkage and creep
6			Water pressure
7	Live loads	Basic variable loads	Highway vehicle load and crowd load
8			Highway vehicle load and the resultant impact loading and soil pressure at flyover
9			Railway train live load and the resultant impact loading and soil pressure at flyover
10			Flowing water pressure at flyover
11		Other variable loads	Influence of temperature variation
12			Swelling pressure
13			Construction load
14	Accidental load		Impact loading of falling rock
15			Seismic force

Note: Serial numbers 1-10 denote main loads. Serial numbers 11, 12 and 14 represent additional loads. Serial numbers 13 and 15 represent special loads.

6.1.2 Rock load shall be determined based on such factors as the terrain, geological conditions, depth, support conditions, construction methods and the distance between adjacent tunnels. It can be calculated as per releasing load or relaxation load. Rock load shall be corrected forthwith once it is found inconsistent with actual site condition during construction and field measurement.

6.1.3 Possible loads that act on tunnel structures simultaneously shall be combined respectively in order to meet the requirements of load bearing capacity and normal services; and designed by considering the most unfavorable combination.

6.1.4 Loads on cut-and-cover tunnels shall be combined as follows:

1 For calculation of the backfill pressure on the roof of a cut-and-cover tunnel, if the impact loading needs to be checked and calculated in case of rockfall hazards, it is possible to only calculate the actual self weight of fill on the roof of tunnel and the impact loading of falling rock, and the self weight of earth and rock accumulated as a result of collapse does not need to be considered.

2 If a highway passes over a cut-and-cover tunnel, the highway vehicle loads shall be considered. The highway vehicle loads shall be calculated in compliance with the relevant provisions in the current *Technical Standard of Highway Engineering* (JTG B01).

3 If a railway passes over a cut-and-cover tunnel, the live loads of trains shall be considered. The live loads of trains shall be calculated in accordance with the provisions concerning live loads in railway standards.

6.1.5 Special loads that are not covered in the Specifications shall be calculated and combined in a special manner.

6.2 Permanent load

6.2.1 The dead weight of tunnel structures shall be calculated based on the design dimensions of structures and the standard unit weight of materials. The superimposed dead load of structures shall be calculated as per actual conditions.

6.2.2 The uniformly-distributed vertical and horizontal pressure due to rock relaxation load in a deep tunnel may be calculated as follows if no obvious eccentric load and expansive force are generated in surrounding rocks:

1 The uniformly-distributed vertical pressure may be calculated as per Eq. (6.2.2-1) and

Eq. (6.2.2-2):

$$q = \gamma h \tag{6.2.2-1}$$

$$h = 0.45 \times 2^{S-1}\omega \tag{6.2.2-2}$$

Where,

q—Uniformly-distributed vertical pressure (kN/m^2);

γ—Unit weight of surrounding rock (kN/m^3);

h—Rock load calculation height (m);

S—Surrounding rock class; to be taken as integers 1, 2, 3, 4, 5 and 6;

ω—Width influence coefficient, calculated as per Eq. (6.2.2-3);

$$\omega = 1 + i(B - 5) \tag{6.2.2-3}$$

B—Tunnel width (m);

i—Rock load increase/decrease rate for each 1m increase/decrease in tunnel width; to be subject to the uniformly-distributed vertical pressure of surrounding rock when $B = 5$m; its value shall be chosen as per Table 6.2.2-1.

Table 6.2.2-1 Values of surrounding rock increase/decrease rate *i*

Tunnel width B (m)	$B < 5$	$5 \leqslant B < 14$	$14 \leqslant B < 25$	
Rock load increase/decrease rate i	0.2	0.1	Excavation by drifts during construction	0.07
			Heading and bench or full-face heading	0.12

2 If the BQ or $[BQ]$ of surrounding rock is available, S in Eq. (6.2.2-2) can be replaced with $[S]$. $[S]$ can be calculated as per Eq. (6.2.2-4) or Eq. (6.2.2-5):

$$[S] = S + \frac{\dfrac{[BQ]_{\text{Upper}} + [BQ]_{\text{Lower}}}{2} - [BQ]}{[BQ]_{\text{Upper}} - [BQ]_{\text{Lower}}} \tag{6.2.2-4}$$

Or

$$[S] = S + \frac{\dfrac{BQ_{\text{Upper}} + BQ_{\text{Lower}}}{2} - BQ}{BQ_{\text{Upper}} - BQ_{\text{Lower}}} \tag{6.2.2-5}$$

Where:

$[S]$—Modified value of surrounding rock class (accurate to one decimal place); BQ or $[BQ]$ will be taken as 800 if they are greater than 800;

BQ_{Upper}, $[BQ]_{\text{Upper}}$—The respective upper limits of Rock Quality Designation BQ and modified Rock Quality Designation $[BQ]$ for this surrounding rock class, which shall conform to Table 6.2.2-2;

BQ_{Lower}, $[BQ]_{\text{Lower}}$—The respective lower limits of Rock Quality Designation BQ and modified Rock Quality Designation $[BQ]$ for this surrounding rock class, which shall conform to Table 6.2.2-2;

Table 6.2.2-2 Upper and lower limits of Rock Quality Designation *BQ* and modified Rock Quality Designation [*BQ*]

Surrounding rock class	Ⅰ	Ⅱ	Ⅲ	Ⅳ	Ⅴ
BQ_{Upper}, $[BQ]_{Upper}$	800	550	450	350	250
BQ_{Lower}, $[BQ]_{Lower}$	550	450	350	250	0

3 The uniformly-distributed horizontal rock load may be determined in accordance with Table 6.2.2-3.

Table 6.2.2-3 Uniformly-distributed horizontal rock load

Surrounding rock class	I, II	III	IV	V	VI
Uniformly-distributed horizontal pressure e	0	$<0.15q$	$(0.15 \sim 0.3)q$	$(0.3 \sim 0.5)q$	$(0.5 \sim 1.0)q$

6.2.3 The rock load in a shallow tunnel may be determined as described in Appendix D.

6.2.4 Where eccentric load is likely to occur in a tunnel, corresponding control measures shall be taken based on the eccentric load state and value. If it is projected that the influence of eccentric load cannot be eliminated, it shall be considered in load combinations and distribution. Eccentric load acting on tunnel lining shall be determined based on the terrain, geological conditions and the covering thickness of surrounding rock. The rock load in a shallow tunnel under eccentric load may be determined as described in Appendix E.

6.2.5 VThe rock load in twin tunnels with small clearance may be calculated as described in Appendix F.

6.2.6 The rock load in twin-arch tunnel may be calculated as described in Appendix G.

6.2.7 The backfill load on a cut-and-cover tunnel may be calculated as described in Appendix H. The physical or mechanical parameters of backfill shall be taken in accordance with the actual backfill materials and the designed compaction requirements.

6.2.8 The active soil pressure acting on the back of portal walls may be calculated based on Coulomb theory. When the back of the wall is inclined acutely or upright, the soil pressure shall be in horizontal direction. Soil pressure on portal walls may be determined in accordance with Appendix J.

6.3 Variable loads

6.3.1 Highway vehicle loads, any resultant impact loading and soil pressures over a cut-and-cover tunnel shall be calculated in accordance with relevant provisions of the current *General Specifications for Design of Highway Bridges and Culverts* (JTG D60).

6.3.2 The railway train live load and the resultant impact loading and soil pressure over a cut-and-cover tunnel shall be calculated in accordance with relevant provisions of the current *Fundamental Code for Design on Railway Bridge and Culvert* (TB 10002.1).

6.3.3 For structures that are constrained from expansive and/or contractive deformation, the influence of temperature variation, concrete shrinkage and creep on the structural elements shall be considered.

6.3.4 The swelling pressure in tunnels in cold regions may be determined according to the local natural conditions, moisture content of surrounding rock in winter, depth of freezing and drainage conditions.

6.3.5 Construction loads shall be evaluated and applied based on construction phases, methods and conditions.

6.4 Accidental loads

6.4.1 If impact loading of potential falling rock needs to be checked and calculated, it may be verified through field survey or relevant calculation.

6.4.2 Seismic load shall be determined in accordance with Chapter 16 and as described in Appendix K.

7 Portal

7.1 General

7.1.1 The portal design shall follow the principle of "proper prolongation of portal and tunnel" without large-scale break of rocks.

7.1.2 The location of portal shall be determined by financial and technical comparisons according to topographical conditions, geological conditions, relevant engineering works outside the tunnel, construction conditions, environmental protection requirements, and operational requirements.

7.1.3 Drainage shall be provided taking account of the terrain at portals, protection of portals and subgrade drainage.

7.1.4 The portal structure shall be able to prevent crushed stone, rolling rock, collapsed material etc. of the acute slopes around and above the portals from falling onto the pavement.

7.1.5 At portals where snow is susceptible to accumulate, snow protection measures should be considered.

7.1.6 Portal and portal structures should be designed in a way that enables easy inspection and maintenance.

7.1.7 Portal and portal structures shall be designed to be compatible with surrounding natural environment.

7.2 Portal Opening

7.2.1 The location of opening shall be determined as specified below:

1 The opening shall be located in a place with stable geological conditions.

2 The tunnel axis should intersect topographic contours at a large angle.

3 If the route approaches the tunnel across or along a gully, the location of the opening shall be determined by comprehensive analysis considering hydrological conditions, protection engineering, waterproofing and drainage engineering.

4 Where the route approaches the tunnel from a gentle slope, the location of opening shall be determined by comprehensive analysis considering tunnel approach conditions, cutting conditions outside the tunnel, side and heading slope protection, drainage, construction, farmland occupation, etc.

7.2.2 The design of the opening shall comply with the following requirements:

1 Minimise excavation of slopes around and above the opening to avoid formation of steep slopes or slopes with steep angles and minimise disturbance to the existing ground surface.

2 Slopes around and above the opening shall be stabilised by sloping, shotcrete and rockbolt, erection of retaining structures, extending the cut-and-cover tunnel, etc., as applicable. Landscaping should be adopted for slope protection.

3 Flood control facilities shall be provided where the opening is within the zone of influence of storm, flood and debris flow.

4 For the opening under a cliff, overhanging rocks shall be removed; the mountain slope should not be cut; the cut-and-cover tunnel should be extended.

5 Precautions shall be taken where nearby surface buildings and subsurface structures interact with the portal.

7.3 Portal engineering

7.3.1 The form of a portal shall be determined by topographical and geological conditions at the opening and surrounding environment.

7.3.2 The portal should be orthogonal to the tunnel axis.

7.3.3 The end-wall tunnel portal shall be designed as follows:

1 End and wing walls at the portal shall be designed as retaining wall structure such that the soil pressure from slopes on the side and above the portal shall be withstand. The thickness of portal and wing walls shall not be less than 0.5 m and 0.3 m respectively.

2 The horizontal distance from the intersection of acute slope and the crest of the backfill above the portal to the back of the end-wall should not be less than 1.5m. The clearance between the invert of drain above the portal to the outer edge of arch lining shall not be less than 1.0m. The top of the end-wall shall be 0.5m above the backfill surface behind the wall.

3 Expansion joints, settlement joints and drainage holes shall be provided, as needed, to the end-wall.

4 The end-wall base shall be placed on stable ground and embedded in to the ground surface. It shall be embedded not less than 0.2m into rock, or not less than 1.0m into soil. The wall base shall be placed deeper than the bottom of various ditches and trenches provided beside the wall. If in a frost swelling soil layer, the elevation of the wall base shall be at least 0.25m below the maximum frozen depth.

5 Any ground with insufficient bearing capacity shall be suitably reinforced.

6 The portal structure design shall meet seismic design requirements.

7.3.4 The cut-and-cover tunnel portal shall be designed as follows:

1 The lining in the approach section shall be constructed of reinforced concrete.

2 The lining in the approach section shall extend not less than 500mm beyond existing mountain slope or the design backfill slope.

3 The end face of linings in the approach section may be of vertical cut, inclined in the shape of inclined bamboo cutting, arch inclined portal in the shape of inverted bamboo cutting, inclined portal in the shape of a trumpet.

4 Where the portal is inclined in the shape of bamboo cutting, the slope gradient shall be steeper than or equal to existing mountain slope gradient or the designed backfill slope gradient.

5 The designed backfill slope should be backfilled per the natural slope gradient. If rock and soil are used as backfill material, the slope gradient should not be steeper than 1 : 1 and the slope surface should be turfed.

8 Structural Design of Lining

8.1 General

8.1.1 Linings shall be provided for highway tunnels. Linings may be constructed as a system of shotcrete and rockbolts, monolithic lining, or composite lining, and shall be selected based on assessment of the surrounding rock class, construction conditions and functional requirements. Expressway, Class I and Class-2 highway tunnels shall be provided with composite linings. For highways of Class III or below, the approach section of the tunnel and the tunnel proper surrounded by Class IV-VI rock masses shall be provided with composite or monolithic linings; the tunnel proper surrounded by Class I-III rock masses may be provided with shotcrete and rockbolt linings.

8.1.2 The tunnel lining shall be designed to take advantage of the rock's self-supporting capacity, taking into account surrounding rock geological conditions, cross-section shape, shoring structures, construction conditions, etc. The lining shall have sufficient strength, stability and durability to ensure the long-term operational safety of the tunnel.

8.1.3 The lining type and support parameters shall be determined by engineering analogy and structural calculation, taking account of functional requirements, surrounding rock class, engineering geological and hydrogeological conditions, tunnel depth, structural stress characteristics, surrounding environment, support type and construction method. During construction, support parameters shall be adjusted based on field monitoring and measurement results during construction as part of the observational design method. They may be determined by test and analysis if necessary.

8.1.4 The tunnel lining shall be designed as follows:

1 The lining should be designed to have a curved wall and arched cross-section.

2 In zones with poor surrounding rock, high lateral pressure and large groundwater inflow, an arch invert may be provided. The radius of the arch invert shall be determined based on geological conditions, groundwater pressure, clear section shape, tunnel width, etc. The space between pavement and the arch invert may be filled with concrete or concrete with rubbles. An arch invert is not required if the rock surrounding the tunnel invert is good and the bearing capacity and stability of the side wall base are sufficient.

3 The approach section requires reinforced linings, of which the length shall be determined based on topographic, geological and environmental conditions. This length shall not be less than 10m for a two-lane tunnel or less than 15m for a three-lane tunnel.

4 The lining in a zone with poor surrounding rock shall extend 5-10m toward the zone with good surrounding rock.

5 The asymmetrically loaded lining shall extend toward conventional lining by a length that shall be determined based on eccentric load and should not be less than 10m.

6 At the intersection of main tunnel and a cross passage with clear width greater than 3.0m, both the main tunnel linings and the cross passage linings shall be reinforced. The reinforced linings shall extend by not less than 5.0m in length into the main tunnel and by not less than 3.0m in length into the cross passage. No movement joints should be placed within the extended length.

8.2 Shotcrete and rockbolt lining

8.2.1 The shotcrete shall have a strength grade of not lower than C20 and a thickness of not less than 50mm.

8.2.2 The shotcrete reinforcement mesh shall be designed as follows:

1 The rebar diameter shall not be less than 6mm and not greater than 12mm.

2 The grid shall be rectangular; the spacing between rebars should be 150mm ~ 300mm.

3 The lapping length of rebars shall not be less than 30d, where d is the rebar diameter.

4 The thickness of shotcrete cover to the reinforcement mesh shall not be less than 20mm. The spacing between two layers of reinforcement mesh, where applicable, shall not be less

than 80mm.

5 The shotcrete shall not be less than 80mm in thickness in case of a single layer of reinforcement mesh, or not less than 150mm in case of two layers of reinforcement mesh.

6 Reinforcement mesh may be used along with rockbolts or temporary short rockbolts, in which case it should be securely fastened to rockbolts or other fixtures.

8.2.3 In weak surrounding rock with large deformation and poor stability or swelling rock, fibre reinforced shotcrete may be employed for protection. Its design shall meet the following requirements:

1 The design strength grade of fibre reinforced shotcrete shall not be lower than C25.

2 The amount of steel fibres added to steel fibre reinforced shotcrete should be 1.5% ~4% (by weight) of the dry mixture.

3 The amount of fibres added to synthetic fiber reinforced concrete shall be determined by test.

4 Where stringent waterproofing performance is required, high-performance shotcrete with a strength grade above C30 shall be used.

8.2.4 In rockbolt support design, the type and parameters of rockbolts shall be selected based on surrounding rock conditions, cross-sectional dimensions, functions and construction conditions, and in accordance with the following provisions.

1 For permanent support, fully bonded rockbolts shall be used. If end-anchorage type rockbolts are used for permanent support, the rockbolt hole must be fully filled with mortar or resin with a strength grade of not less than M20.

2 To support surrounding rock with a short stand up time, fully bonded, resin anchored rockbolts or early strength end anchored bolts with cement mortars should be used.

3 In zones of soft, highly deformable surrounding rock, prestressed rockbolts may be applied. The pre-applied force for such rockbolts shall not be less than 100kPa. Prestressed rockbolts must be anchored into stable rock layer.

4 To support broken surrounding rock wherein hole formation is difficult, self-drilling rockbolts should be used.

5 The diameter of rockbolts should be 20 ~ 28mm.

6 The anchor head shall be equipped with a bearing plate of at least 150mm (length) x 150mm (width) x 8mm (thickness).

8.2.5 Systematic bolting shall be designed as follows:

1 Rockbolts should be arranged radially around the tunnel perimeter. They should be designed to intersect the main joint sets and structural planes within the rock mass at the largest possible angle.

2 Rockbolts should be arranged in quincunx form, as illustrated in Fig. 8.2.5.

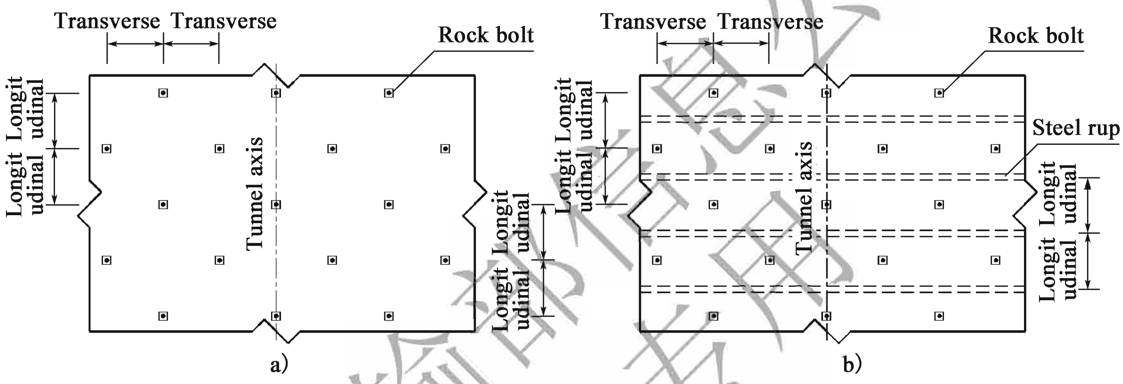


Fig. 8.2.5 Layout of systematic rock bolting

3 The length and spacing of systematic rock bolting shall be determined by calculation or engineering analogy based on surrounding rock conditions and tunnel width.

4 The spacing of rockbolts should not exceed 1/2 the length of the rockbolt, or 1.5m, whichever is the lesser. Long and short rockbolts may be arranged in a staggered manner if their spacing is small.

5 The systematic rock bolts for a two-lane tunnel should not be less than 2.0m long while that for a three-lane tunnel should not be less than 2.5m long.

6 Where systematic rock bolting is not provided for surrounding soil, other forms of support shall be adopted for reinforcement.

8.2.6 Locally unstable rock blocks should be stabilized by local rockbolts. Fully bonded rockbolts, end anchored rockbolts or prestressed rockbolts may be used. The anchored end shall be placed into stable rock mass. Rockbolt parameters may be determined by engineering analogy or calculation.

8.2.7 In zones of poor rock, approach, shallow depth or where ground settlement is strictly restricted, steel frame support may be added in the shotcrete layer. The steel frame support shall be designed as follows:

1 The steel frame support shall be of sufficient stiffness and strength to withstand all possible loads during tunneling.

2 Steel lattice girders should be used.

3 Steel frame supports should be spaced 0.5-1.2m apart.

4 The number of steel rib supports used continuously shall not be less than 3.

5 Lateral connections are required between adjacent steel frame supports. If rebars are used for lateral connections, the diameter of each rebar should not be less than 20mm, the spacing of rebars shall not be greater than 1m and the rebars shall be arranged in a staggered manner on inner and outer edges of the steel frame support.

6 Steel frame supports shall be fabricated in segments, which shall be connected by steel plates.

7 The concrete cover between steel frame support and surrounding rock shall not be less than 40mm thick while that on the free side shall not be less than 20mm thick. If a single layer of shotcrete and rockbolt lining is adopted, the concrete cover on the free side shall not be less than 40mm thick.

8 The shape and dimensions of steel frame supports shall depend on excavated cross-section; the steel frame support shall not intrude into design clearance or secondary lining after all induced deformation.

8.2.8 The steel lattice girders shall be designed as follows:

1 The main bar shall be of HRB400 steel; the web bar may be of HRB400 or HPB300 steel.

2 The diameter of main bar should be 18 ~ 25mm; the diameter of web bar should be 10 ~ 20mm.

3 The cross-sectional dimensions shall be determined by engineering analogy or calculation; the cross-sectional height may be 120 ~ 220mm.

4 The plane of the connecting steel plate should be perpendicular to the axis of steel rib support. U-shaped bars shall be added to aid the welding of the connecting steel plate to the main bar of the steel lattice frame.

8.2.9 The plane of connecting steel plates at both ends of structural steel rib support segments shall be perpendicular to the axis of the steel rib support.

8.2.10 Where forepoling is installed, a steel rib support shall be provided as the rear fulcrum for forepoling. The sectional height of the steel rib support should not be less than 160mm.

8.2.11 Shotcrete and rockbolt lining parameters may be determined by engineering analogy or numerical calculation, subject to adjustments based on field tunnel construction monitoring and measurement. If the method of engineering analogy is used, refer to Table P.0.3 in Appendix P hereto.

8.3 Monolithic linings

8.3.1 Monolithic linings may be designed to have a constant or variable cross-section. The thickness of the arch invert, if applicable, shall not be less than that of side wall.

8.3.2 Where a monolithic lining is employed, reinforced concrete structure should be used in the following circumstances.

1 In zones of obvious eccentric load;

2 At intersections of main tunnel and a cross passage, ventilating duct or refuge that is greater than 3m in clear width;

3 In Class V surrounding rock;

4 In single tunnels with four traffic lanes;

5 At approach areas with seismic peak ground acceleration greater than 0.20g.

8.3.3 The reinforced concrete monolithic lining shall meet the following requirements:

1 The concrete strength grade shall not be lower than C30.

2 The structural thickness should not be less than 300mm.

3 The spacing of load-carrying main bars should not be less than 100mm.

8.3.4 Monolithic linings shall be provided with movement joints as specified below:

1 Settlement joints shall be provided at the boundary between cut-and-cover tunnel and mined tunnel lining and at 5-12m from the portals for tunnels without cut-and-cover section.

2 Settlement joints should be provided at obvious changes in geological conditions and the boundary between different types of lining.

3 One settlement joint should be placed every 30-100m in continuous weak rock.

4 Expansion joints should be installed within a range of 100-200m from the portals in regions characterized by significant temperature fluctuations, particularly in cold areas where the average temperature during the coldest month falls below -15℃.

5 Settlement joints and expansion and contraction joints shall not be less than 20mm wide and may be filled with asphalt board or bitumastic oakum. Settlement joints and Expansion joints should be vertically placed perpendicular to tunnel axis. Settlement joints and expansion joints of arch, walls and arch invert shall be placed within the same cross-section.

6 Settlement joints and expansion joints may also be used as construction joints. Where settlement joints and expansion joints are necessary, these shall be designed in conjunction with construction joints.

8.3.5 For monolithic linings without an arch invert, the base of lined side wall shall be as specified below:

1 The base shall be placed on stable ground and have a sufficient bearing capacity that meets design requirements;

2 The base bottom shall not be higher than the design excavation bottom of cable trench. Where the excavation bottom of the side drain is below the base bottom, the side drain excavation boundary shall be more than 500mm away from the side wall base.

3 Within the thickness of portal wall, the side wall base shall extend to the level of the portal

wall base.

4 The cross-section of side wall base should be enlarged appropriately.

8.4 Composite lining

8.4.1 Composite linings shall be designed as specified below:

1 Primary support shall be designed as permanent support structure and should consist of shotcrete, rockbolts, reinforcement mesh, or steel rib support or a combination thereof. It shall also be in accordance with Section 8.2 herein.

2 Secondary linings shall consist of moulded concrete or moulded reinforced concrete, and comply with Section 8.3 herein.

3 When determining the excavation cross-section, in addition to the required clear section and structural dimensions, consideration shall be given to surrounding rock and primary support deformations that may occur due to loading. The deformation allowance shall be determined by calculation and analysis based on surrounding rock class, cross-section size, depth, construction method, support configuration, etc. Alternatively, it can be predicted by engineering analogy according to Table 8.4.1. The deformation allowance shall also adjusted according to field tunnel construction monitoring and measurement results.

Table 8.4.1 Deformation allowance (mm)

Surrounding rock class	Two-lane tunne	Three-lane tunnel	Surrounding rock class	Two-lane tunnel	Three-lane tunnel
Ⅰ	—	—	Ⅳ	50 ~ 80	60 ~ 120
Ⅱ	—	10 ~ 30	Ⅴ	80 ~ 120	100 ~ 150
Ⅲ	20 ~ 50	30 ~ 80	Ⅵ	Dependent on field measurements	

Notes:

1. Higher values shall be adopted for weak, crushed rock and lower values for intact rock.
2. Values for a four-lane tunnel shall be determined by engineering analogy and calculation and analysis.

8.4.2 Composite linings may be designed by engineering analogy method, and if necessary, may be checked by theoretical analysis. Support parameters for two-lane or three-lane tunnels may be selected from Tables P.0.1 and P.0.2 in Appendix P. Values for a four-lane tunnel shall be determined by engineering analogy and calculation and analysis. During construction, necessary adjustments shall be made to design support parameters based on advance geological forecast and field tunnel construction monitoring and measurements.

8.4.3 In poor surrounding rock mass and geological conditions or where construction of a large span tunnel requires staged excavation, the excavation method shall be designed to define excavation sequence, temporary support measures and parameters.

8.4.4 For weak, flowing rock, swelling surrounding rock or special rock in high in-situ stress conditions, the tunnel support parameters may be determined by field test, taking into account the effect of increasing distortional pressure on the surrounding rock.

8.5 Cut-and-cover tunnel lining

8.5.1 Cut-and-cover tunnel lining should be employed where:

1 The overburden on top of the tunnel is shallow and the tunnel is difficult to build by conventional mining methods. That is, large excavation and/or cuttings should not be made.

2 The subgrade or portals are susceptible to adverse geology such as cave-in, talus, rockfall and debris flow.

3 Excavation of the cut would put adjacent critical buildings (structures) at risk.

4 With a highway, railway, canal or other man made structure passing above the tunnel, mining and flyover are inappropriate.

5 Tunnel length extension is needed to reduce excavation and preserve natural landscape at portals.

8.5.2 The type of cut-and-cover tunnel structure shall be determined based on topographical, geological and construction conditions and taking into account structural safety, economy, practicality, aesthetics and other factors. In addition, the following provisions shall be observed:

1 Arch cut-and-cover tunnel should be employed where the backfill layer on top of the tunnel is thick or the amount of landslide or rockfall in one occurrence is large.

2 An arch structure should be employed where the cut-and-cover tunnel needs to withstand the landslide thrust from the heading slope.

3 A rectangular frame structure may be employed in zones with height restrictions.

8.5.3 Cut-and-cover tunnel linings shall be designed as specified below:

1 The cut-and-cover tunnel shall have a reinforced concrete structure.

2 For a semi-cut arch cut-and-cover tunnel, eccentric load shall be considered. The outer side wall of an arch cut-and-cover tunnel should be thickened appropriately. Where topographic conditions permit, counter pressure backfill or counter pressure wall may be employed.

3 An arch invert shall be provided where side pressure on the arch cut-and-cover tunnel is high or ground bearing capacity is insufficient.

4 When intended as a measure for landslide control, the cut-and-cover tunnel shall be designed as a retaining structure, incorporating comprehensive control measures.

5 Settlement joints should be implemented in areas where there are noticeable shifts in geological conditions. Additionally, the provision of expansion joints should be based on the length of cut-and-cover tunnels in regions with fluctuating air temperatures.

6 For a cut-and-cover tunnel intended for rockfall protection, its structural safety shall be checked for impact load from rockfall.

8.5.4 The cut-and-cover tunnel foundation shall be designed as specified below:

1 The cut-and-cover tunnel foundation with no arch invert shall be in accordance with 8.3.5 herein.

2 When bedrock is exposed or at a shallow depth, the foundation may be placed on the bedrock. When in soft ground, the foundation may be an arch invert, monolithic reinforced concrete slab, or it can also adopt the measures of pile foundation, enlarged foundation, deepened foundation or ground improvements, etc.

3 The cut-and-cover tunnel foundation shall be embedded into rock for certain depth and have certain protection width. When on a slope terrain, the ground may be excavated into benches. Where freezing damage is possible, the base shall extend at least 250mm below the maximum frozen depth.

4 If the outside of foundation is susceptible to the influence of current scour, stabilization and protection measures shall be taken.

5 On a transverse slope terrain, if the outer side foundation is buried more than 3.0m below the pavement, then horizontal reinforced concrete tie shall be installed below the pavement and anchored into inner side foundation or rock mass.

8.5.5 The backfilling on top of cut-and-cover tunnel, treatment on the back of an arch shall be determined according to its purpose, function, terrain and side and heading slope hazards, and in accordance with the following:

1 When posing a serious threat, overhanging rocks from slopes around and above the tunnel shall be removed or stabilized. To protect against general rockfall and collapse hazards, the backfill roof of the arched cut-and-cover tunnel should not be less than 1.2m thick and shall be sloped appropriately for drainage.

2 When cut-and-cover portals are employed, the arch roof may be partially exposed. The exposed part should be covered with a screed or finish layer not less than 20mm thick.

3 In the case of cut-and-cover tunnel flyover, backfill thickness shall be determined considering elevations of highway, railway, canal or other man-made structures, natural environment, aesthetics requirements, structural design, etc. For an arch cut-and-cover tunnel, strengthening of the arch foot may be provided if necessary.

4 Where a water canal, debris flow aqueduct or other structures are envisaged on top of the cut-and-cover tunnel, their impact shall be considered. The bottom of a general water canal or common drain ditch shall not be less than 1.0m from the outer edge of the tunnel roof. For an aqueduct carrying mountain torrents or debris flow, its bottom should be not less than 1.5m from the outer edge of the tunnel roof.

8.5.6 Backfilling of the space behind side walls of the cut-and-cover tunnel shall be determined based on the type of tunnel, geological conditions, design requirements and construction methods, and in accordance with the following:

1 Where elastic resistance of strata at the side walls is taken into account, the space behind the side walls shall be backfilled with concrete, stone pitching or dry rubble.

2 Where calculation of soil pressure on side walls of the cut-and-cover tunnel is based on backfill material, the internal friction angle of the backfill material behind the side walls

shall not be less than the calculated friction angle of existing strata or that of design backfill material.

8.6 Structural requirements

8.6.1 Minimum sectional thickness of each component of the tunnel structure shall be in accordance with Table 8.6.1. The masonry structure lining of two-lane tunnel, three-lane tunnel and Tunnel ventilation fan (TVF) room should not be less than 300mm in thickness.

Table 8.6.1 Minimum sectional thickness (mm)

Type of building material	Lining of tunnel and cut-and-cover tunnel			End wall, wing wall and retaining wall at portals
	Arch ring	Side wall	Arch invert	
Concrete	200	200	200	300
Rubble concrete	—	—	—	500

8.6.2 The rigid angle of concrete foundation step shall not exceed 45°. This shall not exceed 35° in the case of masonry foundation.

8.6.3 Minimum thickness of concrete cover to longitudinal load-bearing bars in reinforced concrete members shall be as specified in Table 8.6.3.

Table 8.6.3 Minimum thickness of concrete cover (mm)

Member thickness	Minimum thickness of cover	
	Non-aggressive environment	aggressive environment
<150	As per site conditions	As per site conditions
150 ~ 300	30	40 ~ 55
301 ~ 500	35	40 ~ 60
>500	40	50 ~ 60

Note: Take higher values for highly aggressive environment and lower values for slightly aggressive environment.

8.6.4 Minimum sectional reinforcement ratio of longitudinal load-bearing main bars in reinforced concrete members shall be as specified in Table 8.6.4.

Table 8.6.4 Minimum sectional reinforcement ratio of longitudinal load-bearing main bars in reinforced concrete members (%)

Load-bearing type	Minimum reinforcement ratio				
Compression member	Full load bearing main bar	0.6			
	One-side load bearing main bar	0.2			
One-side tensile bars in flexural member and eccentric or axial tensile member	Type of rebar	Concrete strength grade			
		C25	C30	C40	C50
	HPB300	0.25	0.30	0.35	0.40
	HRB400	0.20	0.20	0.25	0.30

Notes:

1. When using HRB400 bars, minimum reinforcement ratio of full load bearing main bar in compression member shall be 0.1% less than the value specified in the table above.
2. Compression bars in eccentric tensile member shall be considered as one-side load bearing main bars in compression member.
3. The reinforcement ratio of full load bearing main bars and one-side load bearing main bars in compression member as well as one-side tensile bars in axial tensile member and small-eccentricity tensile member shall be calculated based on the total cross-section area of the member. The reinforcement ratio of one-side tensile bars in flexural member and large-eccentricity tensile member shall be calculated based on the total cross-section area less the compression flange area.
4. Where bars are arranged around the perimeter of the member cross-section, the "one-side load bearing main bar" refers to the load bearing main bar placed along one of two opposite edges in force direction.

8.6.5 Bent-up bars in reinforced concrete member (Fig. 8.6.5) shall be designed as specified below:

1 If a load bearing main bar needs bending, anchorage length shall be reserved at the bend end point B. This length shall not be less than 20d (d is bar diameter) in tensile zone, or 10d in compression zone. A hook shall be designed at the end in the case of plain round bar.

2 The bend angle of a bent-up bar should be 45° or 60° for beam, and should not be less than 30° for slab.

3 When the bent-up bar is HPB300, minimum bending radius R shall be 10d (d is bar diameter). When the bent-up bar is HRB400, minimum bending radius R shall be 12d.

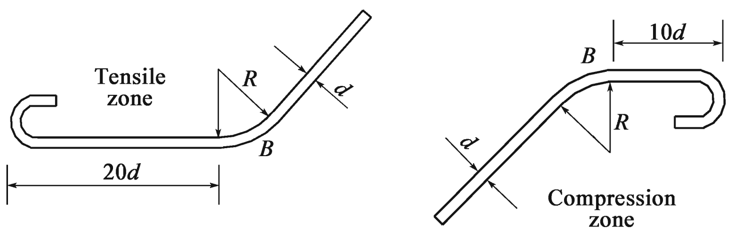


Fig. 8.6.5 Configuration of bent-up bar end

8.6.6 Bar anchorage in reinforced concrete member shall be as specified below:

1 The bar anchorage length shall be as specified in Table 8.6.6.

Table 8.6.6 Bar anchorage length

Anchorage condition		Bar type	
		HPB300	HRB400, HRB500
Anchorage length of compression bar measured from the unstressed point	≥30*d*	No hook	—
	<30*d*	10*d* + *straight hook*	—
	≥20*d*	—	No hook
	<20*d*	—	10d + straight hook
Anchorage length of bar in tensile member, calculated per adhesion stress	In a zone without transverse pressure	30*d* + *semicircular hook*	20*d* + *straight hook*
	In a zone with transverse pressure	15d + semicircular hook	10d + straight hook
Anchorage length of tensile bar in flexural member and eccentrically compressed member, measured from the unstressed point	In compression zone	10d + straight hook	10*d with no hook*
	In tensile zone (in difficult conditions)	20*d* + *semicircular hook*	20d + straight hook
Length of bent-up bar stretching into the compression zone	≥20*d*	Without straight section in parallel with longitudinal bar; with straight hook at the end	Without straight section in parallel with longitudinal bar; and without hook
	<20*d*	With a straight section 10d long in parallel with longitudinal bar; and with straight hook	With a straight section 15d long in parallel with longitudinal bar; and without hook

Notes:

1. If their diameters are greater than 25mm, the anchorage length of bars HRB400 and HRB500 shall be multiplied by a safety factor of 1.1.
2. The anchorage length of HRB400 and HRB500 epoxy resin coated bars shall be multiplied by a safety factor of 1.25.
3. Where the bar is susceptible to disturbance during concreting (such as sliding formwork), its anchorage length shall be multiplied by a safety factor of 1.1.
4. Where the thickness of concrete cover to HRB400 and HRB500 in anchorage zone is more than 3 times the bar diameter and stirrups are provided, its anchorage length may be multiplied by a factor of 0.8.

2 Where a hook is required at the end of load-bearing main bar, the inner diameter of the hook shall be 4d (d is bar diameter) and the straight section shall be 5d long.

8.6.7 Connection between load-bearing main bars shall be made as specified below:

1 Connections of load-bearing main bar should be placed at a location of minimum stress. The number of joints on the same bar should be minimised.

2 Plain round bars of more than 25mm in diameter and all deformed bars shall be connected by welding or mechanically. The tensile strength of the welded joint or mechanical joint shall not be lower than the strength of the bars themselves.

3 Welded joints shall be staggered with each other. The connection section of welded joints shall be 35d (d is the diameter of the larger bar) long and not less than 500mm. Any point with its midpoint within the length of a connection section is considered as in the same connection section.

4 In the same connection section, the area of welded joints and mechanical joints as a percentage of load-bearing bar shall not be greater than 50% for tensile main bar whereas the area of joint area as a percentage of compression main bar is exempt from this restriction.

5 Connection between plain bars of small diameter may be made through lap joint, in which case the end of bars shall be made into semi-circular hooks. The distance between points of tangency on two hooks shall not be less than 30d for tensile bars nor less than 20d for compression bars. Within the lap range, the bars shall be tied with wires or welded.

6 Bar connections under other circumstances shall comply with applicable provisions in the *Code for Design of Concrete Structures* (GB 50010).

8.6.8 Reinforcement configuration for members subject only to axial compression and complete with load-bearing main bar and general stirrups shall be as specified below:

1 The cross-sectional area of the load-bearing main bar shall not be less than that of the member by 0.6%, nor should it be greater by 3%.

2 The diameter of the load-bearing main bar should not be less than 12mm.

3 The diameter of stirrups shall not be less than 1/4 of the diameter of load-bearing main bar, nor less than 6mm.

8.6.9 Tunnel lining reinforcement configuration shall be as specified below:

1 Minimum diameter of the load-bearing bar shall be 16mm.

2 The cross-sectional area of the load-bearing main bar shall not be less than that of the member by 0.6%, nor should it be greater by 3%.

3 Distribution bars perpendicular to load-bearing main bar shall be provided inside and outside the lining. The diameter of the distribution bars should not be less than 12mm and their spacing should not be greater than 300mm.

4 The inner and outer layers of load-bearing bars in the lining shall be connected by connecting stirrups of not less than 6mm diameter. Both ends of the stirrup shall be made into a hook whose inner diameter shall not be less than the diameter of load-bearing bar; the length of straight section shall not be less than 5d (d is bar diameter).

5 Lining stirrups shall be arranged at the intersection of circumferential load-bearing bar and distribution bar, at intervals not greater than twice the spacing of distribution bars. The circumferential load-bearing bar shall be tied or welded to stirrups.

6 Constructional position limit bars of not less than 16mm diameter should be arranged at intervals not less than 2.0m × 2.0m between inner and outer layers of circumferential load-bearing bars. The position limit bars shall be located at the intersection of circumferential load-bearing bars and distribution bars. Where position limit bars are provided, stirrups are not required. The position limit bars shall be welded to circumferential load-bearing bars and distribution bars.

8.6.10 Maximum calculated width of cracks on the surface of reinforced concrete members for tunnel shall be 0.2mm, or 0.15mm in highly corrosive environment, and comply with relevant codes.

8.6.11 Where tight control on crack width is required, the spacing of distribution bars in tunnel lining may be reduced appropriately, but should not be less than the spacing of main bars.

8.6.12 For a member in highly corrosive environment, hoisting rings, attachments and other ironwork embedded in concrete and partially exposed shall be isolated from steel bars in the concrete member, or the exposed ironwork shall be protected against corrosion.

8.6.13 Configuration of other bars in tunnel structure member and bars with seismic consideration shall comply with the current *Code for Design of Concrete Structures* (GB 50010).

8.6.14 The diameter and spacing of bars in cover plates for cable trench and water ditch shall be as stated in Table 8.6.14.

Table 8.6.14 Diameter and spacing of bars in cover plates (mm)

Category	Diameter d	Plate thickness h	Spacing
Longitudinal load-bearing bar (main bar)	6, 8 or 10 commonly used for load-bearing bar	≤150mm	≤200
		$h > 150$mm	≤1.5h *and* ≤300
Constructional bar	d≥6 commonly used for distribution bar		≤200

9 Structural Calculation

9.1 General

9.1.1 The sectional strength of a members forming the tunnel structure shall be checked by the ultimate limit state design method. Where the structure's crack resistance is specified, the concrete member shall be checked for crack resistance. For a reinforced concrete member, its crack width shall be checked.

9.1.2 This chapter applies to analysis of static forces.

9.2 Lining calculation

9.2.1 In the case of monolithic lining for deep tunnel, monolithic lining or secondary lining of composite lining for shallow tunnel and the lining for cut-and-cover tunnel, calculation should be performed using the load-structure method. This method may also be used for secondary lining of composite lining for deep tunnel. The calculation principle of this method is presented in Appendix L, hereto.

9.2.2 In calculating internal forces and deformation of tunnel lining using the load-structure method, elastic resistance shall be taken into account. For a densely backfilled lining, the size and distribution of elastic resistance may be determined based on local deformation theory and from Eq. (9.2.2).

$$\sigma = k\delta \tag{9.2.2}$$

where

σ—Strength of elastic resistance (MPa);

k—Elastic resistance coefficient of surrounding rock, which may be selected from Table A.0.7-1 in the absence of measured data;

δ—Lining deformation toward surrounding rock (m), taken as zero if it is toward the tunnel.

9.2.3 Calculation for a lining with arch invert shall take into account the effect of the arch invert on internal force of the structure.

9.2.4 In checking the sectional strength of a member using the ultimate limit state design method, different safety factors not less than values given in Tables 9.2.4-1 and 9.2.4-2 shall be applied depending on load combinations. In checking the strength for construction stage, the safety factors can be obtained by multiplying the value under "permanent load + basic variable load + other variable load" in Tables 9.2.4-1 and 9.2.4-2 by a reduction coefficient of 0.9.

Table 9.2.4-1 Strength safety factor for concrete and masonry structures under various load combinations

Cause of failure	Concrete			Masonry		
	Permanent load + basic variable load	Permanent load + basic variable load + other variable load	Permanent load or permanent load + accidental load	Permanent load + basic variable load	Permanent load + basic variable load + other variable load	Permanent load + accidental load
Concrete or masonry reaching ultimate compressive strength	2.4	2.0	1.8	2.7	2.3	2.0
Concrete reaching ultimate tensile strength	3.6	3.0	2.7			

Table 9.2.4-2 Strength safety factor for reinforced concrete structure under various load combinations

Cause of failure	Permanent load or permanent load + basic variable load	Permanent load + basic variable load + other variable load	Permanent load + accidental load
Rebar reaching ultimate strength or concrete reaching compressive or shear ultimate strength	2.0	1.7	1.5
Concrete reaching ultimate tensile strength	2.4	2.0	1.8

9.2.5 Primary support for composite lining shall be designed mainly by engineering analogy method. Support parameters may be determined by the ground-structure method presented in Appendix M hereto and checked for service and construction stages respectively.

9.2.6 During analysis of surrounding rock stability, the safety factor of surrounding rock during construction may be checked by strength reduction finite element method; the safety factor of surrounding rock after primary support is installed may be used as the basis for its stability assessment.

9.2.7 Where the secondary lining of composite lining and primary support jointly bear the pressure from surrounding rock and other external loads, internal force and deformation may be calculated by ground-structure method and checked by load structure method, using load values obtained under 6.2.2 herein.

9.2.8 During the design and analysis phase, lining design calculations shall adopt the characteristic parameters of surrounding rock, per available geological data or from Table A.0.7-1 when such data are unavailable. During construction, the design parameters shall be verified and the analysis updated according to in-situ geology encountered, tunnel construction monitoring and measurement results.

9.2.9 When designed per bearing capacity, deformation of primary support for composite lining shall not exceed the design deformation allowance.

9.2.10 For an eccentrically compressed concrete member of monolithic lining or cut-and-cover tunnel lining, eccentricity of its axial force should not be greater than 0.45 times the sectional thickness. This value should not be greater than 0.3 times the sectional thickness for an eccentrically compressed member of exterior wall of semi-cut open tunnel, shed tunnel, side wall of cut-and-cover tunnel and masonry. The base eccentricity shall comply with Table 9.4.1 herein.

9.2.11 Compressive strength of axial and eccentrically compressed member of concrete and masonry with a rectangular cross-section shall be calculated from Eq. (9.2.11):

$$KN \leqslant \varphi \alpha R_a bh \tag{9.2.11}$$

where

K—Safety factor, to be obtained from Table 9.2.4-1 herein;

N—Axial force (kN);

φ—Factor of longitudinal bending of a member, taken as $\varphi = 1$ for tunnel lining, arch ring of cut-and-cover tunnel and side wall with dense backfill behind it; obtained from Table 9.2.11-1 per slenderness ratio for other members;

α—Eccentric effect factor of axial force, obtained from Table 9.2.11-2.

R_a—Ultimate compressive strength of concrete or masonry, obtained from Tables 5.2.4 and 5.2.15 herein;

B—Sectional width (m);

H—Sectional thickness (m);

Table 9.2.11-1 Factor of longitudinal bending of concrete and masonry members

H/h	<4	4	6	8	10	12	14	16
Factor of longitudinal bending φ	1.00	0.98	0.96	0.91	0.86	0.82	0.77	0.72
H/h	18	20	22	24	26	28	30	
Factor of longitudinal bending φ	0.68	0.63	0.59	0.55	0.51	0.47	0.44	

Notes:

1. H is member height and h is the length of short edge of cross-section (in the case of concentric compression) or sectional edge length within the plane subject to bending moment (in the case of eccentrically compressed).
2. Intermediate values of H/h may be obtained by interpolation method.

Table 9.2.11-2 Eccentric effect factor α

e_0/h	α	e_0/h	α	e_0/h	α	e_0/h	α	e_0/h	α
0.00	1.000	0.10	0.954	0.20	0.750	0.30	0.480	0.40	0.236
0.02	1.000	0.12	0.923	0.22	0.698	0.32	0.426	0.42	0.199
0.04	1.000	0.14	0.886	0.24	0.645	0.34	0.374	0.44	0.170
0.06	0.996	0.16	0.845	0.26	0.590	0.36	0.324	0.46	0.142
0.08	0.979	0.18	0.799	0.28	0.535	0.38	0.278	0.48	0.123

Notes:

1. e_0 is eccentricity of axial force.
2. $\alpha = 1.000 + 0.648\,(e_0/h) - 12.569\,(e_0/h)^2 + 15.444\,(e_0/h)^3$.

9.2.12 According to crack resistance requirements, the tensile strength of eccentrically compressed concrete member of rectangular cross-section shall be calculated from Eq. (9.2.12):

$$KN \leqslant \frac{1.75R_l bh}{\dfrac{6e_0}{h} - 1} \tag{9.2.12}$$

where

K—Safety factor, to be obtained from Table 9.2.4-1 herein;

N—Axial force (kN);

R_l—Ultimate tensile strength of concrete, obtained from Table 5.2.4 herein;

B—Sectional width (m);

H—Sectional thickness (m);

e_0—Eccentricity of axial force.

9.2.13 If the concrete for monolithic lining is placed intermittently or when side wall is masonry and arch ring is made from concrete, the eccentricity of arch foot or elephant foot section shall not be greater than 0.3 times its sectional thickness; values specified in Table 9.2.4-1 herein for masonry shall be used in calculating the safety factor of sectional compressive strength.

9.2.14 Sectional strength of flexural and eccentrically compressed reinforced concrete members may be calculated per Appendix N hereto.

9.2.15 For flexural members, maximum deflection value calculated based on basic load combination shall not be greater than the permissible values presented in Table 9.2.15.

Table 9.2.15 Permissible deflection of flexural members

Type of member		Permissible deflection
Beam and slab	$l_0 \leq 5m$	$l_0/250$
	$5m < l_0 \leq 8m$	$l_0/300$
	$l_0 > 8m$	$l_0/400$

9.2.16 Deformation of reinforced concrete flexural members under various load combinations may be calculated by Material Mechanics method according to given stiffness.

9.2.17 Where the tunnel is constructed by staged excavation method and the support structure is completed in steps over a long time, the support structure shall take into account stress during construction.

9.2.18 The structural calculation for linings near portals should consider the interaction effect between the portal slopes and the tunnel structure. Depending on the magnitude of this effect, the lining structure may need to be reinforced or the number of longitudinal rebars increased.

9.3 Calculation for cut-and-cover tunnel

9.3.1 The internal force in cut-and-cover tunnel lining may be calculated using load structure model. According to backfill requirements, lateral load may take into account the effect of elastic resistance or soil pressure.

9.3.2 For cut-and-cover tunnel lining, the member's sectional strength shall be calculated by ultimate limit state design method, and safety factors given in Table 9.2.4-2 herein shall be used depending on load combinations.

9.4 Calculation for portals

9.4.1 Where end-wall tunnel portal is employed, the end wall and wing wall may be regarded as retaining wall and their strength shall be checked for ultimate state. The stability against overturn around wall toe and sliding along wall base shall also be checked. The checking shall be in accordance with Table 9.4.1 as well as applicable provisions in the current *Code for Design of Highway Subgrades* (JTG D30), *Code for Design of Highway Masonry Bridges and Culverts* (JTG D61) and *Code for Design of Ground Base and Foundation of Highway Bridges and Culverts* (JTG D63). For a high portal wall, the tensile stress in control section shall be calculated.

Table 9.4.1 Checking specifications for portal wall

Wall section load effect value S_d	≤ structural resistance effect value R_d (in ultimate state)
Wall section eccentricity e	≤0.3 times sectional thickness
Base stress σ	≤ permissible bearing capacity of ground
Base eccentricity e	Rock foundation ≤B/5 ~ B/4; soil foundation ≤B/6 (B is wall base thickness)
Anti-sliding stability safety coefficient K_c	≥1.3
Anti-stability against overturning safety coefficient K_0	≥1.6

10 Water-proofing and Drainage

10.1 General

10.1.1 Tunnel waterproofing and drainage design shall follow the principle of ‘comprehensive control according to local conditions through a combination of prevention, drainage, interception and plugging’ to properly handle surface water and groundwater. Waterproofing and drainages both inside and outside the tunnel shall be complete and unobstructed.

10.1.2 Waterproofing and drainage for expressway, Class-1 highway and Class-2 highway tunnels shall be such that:

1 There is no seepage from the arch, side wall or equipment cavern, or wet stain on the pavement.

2 There is no ponding behind the tunnel lining in sections with freezing hazard or no freezing of drains.

3 There is no water dripping from the arch of vehicular and pedestrian cross passages or from side walls.

10.1.3 Waterproofing and drainage for Class III and IV highway tunnels shall be such that:

1 There is no water dripping from the arch, no water flowing on side walls or no seepage water in equipment caverns, no water ponding or flowing water on pavement surface

2 There is no ponding behind the tunnel lining in - sections with freezing hazard or no freezing of drains.

10.1.4 Waterproofing and drainage measures adopted for tunnels shall protect the natural environment. If seepage in the tunnel is likely to cause reduction in surface water and affect people's work and life, water plugging shall be applied to surrounding rock.

10.2 Waterproofing

10.2.1 Where surface water is likely to infiltrate into the tunnel, prevention and control measures should be taken; abandoned pits, boreholes, etc. shall be filled and sealed.

10.2.2 If composite lining is employed, a waterproofing layer shall be placed between primary support and secondary lining. The waterproofing layer should be a combination of waterproofing membrane and non-woven fabric and shall be as specified below:

1 The waterproofing membrane should be made of weldable sheet material. Its thickness shall not be less than 1.0mm and its joint lap length not less than 100mm.

2 The density of non-woven fabric shall not be less than $300g/m^2$.

3 The non-woven fabric should not be bonded to the waterproofing membrane.

10.2.3 The cast-in-situ concrete lining shall meet seepage resistance requirement. Seepage resistance grade of the concrete should not be less than P8.

10.2.4 Reliable waterproofing measures shall be applied to construction joints, settlement joints and expansion joints in cast-in-situ concrete lining.

10.2.5 Where aggressive groundwater is present, corrosion and erosion resistant waterproofing materials shall be used depending on corrosion type; the concrete waterproofing grade may be raised as appropriate.

10.2.6 In zones with high volumes of seepage and inflow from the surrounding rock, rock mass grouting may be adopted to plug water.

10.2.7 Waterproofing for reserved caverns should be consistent with that for main tunnel.

10.3 Drainage

10.3.1 In the tunnel a longitudinal drainage should be installed such that groundwater is discharged separately from discharge of operational sewage and fire-fighting sewage.

10.3.2 Drainage in the tunnel shall be as specified below:

1 Side drain shall be provided on both sides of pavement.

2 Drainage gradient of the side drain should be consistent with the tunnel's longitudinal gradient.

3 The side drain should have a rectangular cross-section. Where these drains are buried conduits, filter grate and settling ponds shall be provided at 25-30m intervals.

4 Where a center drain is not provided in the tunnel, groundwater behind the lining may be channeled to the side drain whose invert should not be less than 50mm below the base of pavement structure.

5 Measures shall be taken to prevent water accumulation in cable ducts.

10.3.3 When designing center drain below the pavement structure layer, the following shall be observed:

1 The center drain should be separate from side drain.

2 The center drain may be located in the center or on the sides of tunnel, its location, quantity and depth shall be dependent on tunnel length, pavement width, type of arch invert, frozen depth, etc.

3 The center drain should have a rectangular cross-section. Its dimensions shall be dependent on tunnel length, longitudinal gradient and groundwater inflow.

4 The center drain should be provided with settling ponds at 50-200m intervals and with manholes as needed. The location and configuration of the manholes shall be easy for cleaning and inspection, and their spacing should not be greater than 200m.

5 The manhole cover may be covered by pavement surface course.

10.3.4 Drainage of pavement structure base shall be as specified below:

1 The surface of pavement cushion or arch invert fill layer shall be made into transverse

drainage slope not less than 1.5% tilting toward the center drain, if any.

2 Where there is water seepage at the tunnel bottom, a transverse drainage pipe should be placed along the tunnel length every 3-8m. Such drain should be located at construction joint of cushion or arch invert fill or the point of water inflow at the tunnel bottom.

3 In a tunnel without a center drain, the drainage gradient of transverse drainage pipe should be consistent with pavement transverse slope and shall connect to the lower side drain through an interface not lower than the bottom of the side drain.

4 In a tunnel with center drain, the drainage gradient of transverse drainage pipe shall not be less than 1.5% and shall tilt toward and link to the center drain.

5 The transverse drainage pipe should be highly permeable pipe whose diameter shall not be less than 50mm.

10.3.5 The tunnel lining drainage design shall be as specified below:

1 At the bottom of secondary lining behind side walls, longitudinal drainage pipe shall be arranged without encroaching upon secondary lining space. Its drainage gradient shall be consistent with the longitudinal gradient of the tunnel and its diameter shall not be less than 100mm.

2 Circumferential drainage pipe shall be arranged between waterproofing layer and primary support at intervals that should not be greater than 10m and the spacing shall be decreased in zones of high inflow. Where there is concentrated seepage of water from the surrounding rock, vertical drainage pipe may be added. Circumferential and vertical drainage pipes shall be connected to the longitudinal drainage pipe and shall not be less than 50mm in diameter.

3 Transverse pipes shall pass through secondary lining at the foot of lined side walls to longitudinal drainage pipe at one end, and connect to center drain, if any, at the other end, or to side drain if there is no center drain. Transverse pipes should have a diameter not less than 80mm, drainage gradient not less than 1% and longitudinal spacing not greater than 10m. This spacing shall be decreased in zones of high inflow.

10.3.6 If developed groundwater in obvious aquifers fully recharged poses potential hazard to the tunnel where water plugging is not efficient, adit and parallel heading may be used for drainage, or drainage tunnel and other interception and drainage facilities may be provided.

10.3.7 In a tunnel with high volumes of predicted inflow, drainage cross-section of the center drain and side drain shall be enlarged.

10.4 Waterproofing and drainage for portals and cut-and-cover tunnel

10.4.1 Interception drains shall be provided as needed 3-5m beyond the excavation line of tunnel and adit portals and side and heading slopes of cut-and-cover tunnel, such that the landscape effect of side and heading slopes is not affected.

10.4.2 Where the cutting outside the tunnel portal is on an upgrade, reverse side drain or drainage guidance measures may be adopted on both sides of subgrade outside the portal so that water outside the tunnel does not flow into the tunnel.

10.4.3 Waterproofing and drainage design for cut-and-cover tunnel shall be as specified below:

1 Externally bonded waterproofing layer shall be applied to the outer edge of cut-and-cover tunnel lining.

2 At the connection between cut-and-cover tunnel and subsurface-cut tunnel, waterproofing layers shall be lapped and sealed.

3 Clay drainage layer should be placed on top of backfill and connected to side and heading slopes. This layer should be covered with planting soil not less than 20cm thick.

4 Drainage ditches shall be provided, as appropriate, on top of cut-and-cover tunnel backfill.

5 The extrados of the cut-and-cover tunnel, if exposed, shall be provided with waterproof screed or ceramic tiles.

6 Longitudinal and vertical drainage pipes should be arranged at the foot of or behind the side wall next to the mountain to direct water to side wall drainage hole for discharge.

10.5 Waterproofing and drainage for tunnels in cold regions

10.5.1 In addition to specifications under 10.3 and 10.4 herein, waterproofing and drainage for tunnels in cold regions shall meet the following specifications:

1 Where groundwater may cause freezing condition, a center drain should be provided. The bottom of the center drain shall be below the level of freezing.

2 If the center drain cannot meet drainage and anti-freezing requirements, cold-proof drainage tunnel may be provided. It shall be placed 3 ~ 5m beneath the pavement. Its arch and side walls shall be provided with drainage channel.

3 Drainage pipe shall be provided in the portal subgrade approach section linking center drain in the tunnel to cold-proof drainage tunnel. The drainage pipe shall be buried below the frozen depth and provided with manhole at its corner. The drainage pipe exit shall be provided with cold-proof gravelly outfall.

4 The seepage resistance grade of tunnel lining concrete may be raised as appropriate.

10.5.2 Longitudinal drainage pipe behind the lining and transverse pipes shall be resistant to freezing.

11 Special Types of Tunnels

11.1 General

11.1.1 Tunnels may take special forms such as twin tunnels with small clearance, twin-arch tunnels, branching-out tunnels and shed tunnels. The type of tunnel will be dependent on topographical and geological restrictions, the impact of structures around the tunnel and the requirement for overall route design.

11.1.2 The form of tunnel shall be properly determined taking into account geology, terrain, structural safety, construction conditions, environmental protection and other factors.

11.2 Twin tunnels with small clearance

11.2.1 Twin tunnels with small clearance may be adopted at portals where the space is narrow and route arrangement is restricted, or at localised locations of short, medium or long tunnel portals where land occupation for portal needs to be reduced.

11.2.2 The design of twin tunnels with small clearance shall meet the following requirements:

1 Composite lining structure shall be employed. Support parameters shall be determined according to engineering analogy, construction method, analysis and calculation, subject to adjustments based on field monitoring and measurement results during tunnel construction.

2 Requirements on construction sequence, excavation method and temporary support measures shall be specified, depending on surrounding rock geological conditions and the clear distance between the two tunnels. In addition, measures shall be specified to protect

or strengthen the rock pillar for stability, if required.

3 Where the clear distance between the two tunnels is less than 0.8 times excavation span, the length of twin tunnels with narrow pillar should be limited to 1,000m.

11.3 Twin-arch tunnels

11.3.1 Twin-arch tunnels may be adopted in special areas of narrow terrain or with constraints on alignment.

11.3.2 The twin-arch tunnels shall be designed as specified below:

1 The middle wall for twin-arch tunnels should be of composite type.

2 Composite lining structure shall be employed. Support parameters shall be determined according to engineering analogy, construction method, analysis and calculation, subject to adjustments based on field monitoring and measurement results during tunnel construction.

3 Secondary linings shall be constructed of reinforced concrete.

4 Specific requirements on construction sequence, excavation method and temporary support measures shall be stipulated, according to surrounding rock geological conditions as well as structural stress and rock stability analysis under unfavorable conditions during construction.

5 Where the twin-arch tunnels are subjected to eccentric load, the impact of such load shall be taken into account in determining support parameters, construction method and construction sequence.

6 If monolithic middle wall is employed, longitudinal construction joints on both sides of the middle wall shall be above the level of longitudinal drain on the top of middle wall, and appropriate waterstop measures shall be taken. Vertical drain shall be embedded in the middle wall. Its diameter shall not be less than 100mm and its longitudinal spacing should not be greater than 10m.

7 Where composite middle wall is employed, heading axis during construction should deviate from the middle wall centerline.

8 Deformation joints shall be placed according to structural need. Deformation joints for main tunnels and for the rock pillar shall be placed within the same cross-section.

9 Effective measures shall be taken to prevent adverse effect of horizontal thrust from the tunnel arch on the rock pillar structure during construction.

10 The twin-arch section should not be greater than 500m in length.

11.4 Branching-out tunnel

11.4.1 Branching-out tunnel may be adopted at portals with narrow space or special requirements, or at portals of long or extra-long tunnels where it is very difficult to arrange two halves of the route separately outside the tunnel. In Class V and VI surrounding rock, branching-out tunnels should not be adopted.

11.4.2 Branching-out tunnels shall be designed as specified below:

1 Horizontal alignment of route centerline in the approach section to branching-out tunnel should be curved. Where this alignment is straight, the horizontal alignment of left and right tunnels may be separated in the form of small-deflection "S" curve that meets operational safety requirements and the requirement of alignment consistency within 3s travelling length at design speed inside and outside the portal. Where left and right tunnels are separated using small-deflection "S" curve, design longitudinal gradient should not be greater than 2% in general circumstances or 2.5% under special circumstances.

2 The length of branching-out section and the location of structural variation in the branching-out section shall be determined based on surrounding rock geological conditions, distance between routes in left and right tunnels and construction methods. The length of branching-out section should not be greater than 600m.

3 The branching-out tunnel shall be provided with composite lining. Its support parameters may be determined by engineering analogy or calculation and analysis.

4 Specific requirements on construction sequence, excavation method and temporary support measures shall be placed, according to surrounding rock geological conditions. Structural stress and rock stability analysis under unfavorable conditions during construction shall be carried out.

5 Within 10-15m of initial section of twin-arch lining and neighborhood lining in the branching-out tunnel, the structure shall be reinforced appropriately. The tunnel structural should be designed per surrounding rock of one class lower.

6 At the portal of a branching-out tunnel that is at shallow depth, a cut-and-cover tunnel structure may be adopted for bidirectional large-span sections and some twin-arch tunnel sections.

7 Deformation joints shall be arranged at changes in structural form.

8 Bulkhead shall be provided at changes in lining structure; it shall be designed per geological conditions and waterproofing.

11.4.3 Where full longitudinal ventilation is adopted for both left and right tubes of a branching-out tunnel, the form of portal shall take account of the air supply and exhaust channeling effect at portals of left and right tubes, and the following measures shall be taken as applicable:

1 If the tunnel has a large span at the portal, a median partition wall shall be provided in the large span tunnel and extension at the portal.

2 If twin-arch tunnel is at the portal, portals of left and right tubes should be staggered or isolated.

3 Where twin tunnels with small clearance is at the portal and there is air and exhaust fluid channeling effect, portals of left and right tubes may be staggered or isolated.

4 Air inflow and outflow at the portal shall be adjusted.

11.5 Shed tunnel

11.5.1 Shed tunnel may be provided in river and mountain side or steep sections and skew portal with high side and heading slopes.

11.5.2 The shed tunnel may be in arch, semi-arch or rectangular shape depending on topographical condition, geological condition, climate condition, protection and environmental requirements.

11.5.3 Construction gauge of the shed tunnel shall meet basic requirements for tunnel construction gauge. For a shed tunnel connected to tunnel portal, its construction gauge shall be the same as tunnel construction gauge. For an independent shed tunnel on expressway, its construction gauge width should be the same as subgrade construction gauge width. The outline and dimensions of shed tunnel shall be proposed according to topographical conditions, its purpose and structural form.

11.5.4 Loadings on the shed tunnel shall take account of mutual effect between side slope and shed tunnel. Backfill load and surcharge load shall be considered as permanent load.

11.5.5 Overall stability of the shed tunnel shall be analysed and its key structural members shall be subjected to sectional strength check and crack check, according to the form of shed tunnel, stress conditions and constraints.

11.5.6 The shed tunnel structure shall be designed as specified below:

1 The shed tunnel shall have a reinforced concrete structure.

2 Main structure of arch and semi-arch shed tunnel shall be monolithic.

3 Rectangular shed tunnel shall have an integrated framework structure or simply supported structure.

4 Settlement joints shall be placed depending on geology and structural form. Expansion joints should be placed if the shed tunnel is more than 40m long.

11.5.7 Design of shed tunnel foundation shall be as specified below:

1 Shed tunnel foundation shall be placed on solid ground. If the foundation is on soft subsoil, monolithic reinforced concrete floor, pile foundation, extended foundation, deepened foundation and other measures shall be implemented to ensure adequate bearing.

2 The elevation of shed tunnel base shall be at least 200mm below the excavated elevation of side drain bottom. In zones of possible freezing damage, the base shall extend at least 250mm below the freezing line.

3 If the outside of foundation is susceptible to the influence of current scour, stabilization and protection measures shall be taken.

4 On a transverse slope terrain, if the outer side foundation of the shed tunnel is buried more than 3m below the pavement, then reinforced concrete horizontal tie rod shall be installed

below the pavement and anchored into inner side foundation or rock mass. If column is outside the shed tunnel, stringer may be added to connect adjacent columns.

11.5.8 The side of shed tunnel next to mountain and the outer surface of its top shall be provided with waterproofing layer. Construction joints, settlement joints and deformation joints shall be waterproofed.

11.5.9 The back of the side next to mountain shall be provided with drainage pipe. At the basement on the side next to mountain, drainage holes shall be arranged at 5 ~ 10m intervals.

12 Auxiliary Channel

12.1 General

12.1.1 Auxiliary channels may be provided for operational ventilation, disaster prevention and rescue or increasing tunnel faces and improving construction ventilation and drainage conditions.

12.1.2 The site of auxiliary channel shall be selected considering topographical and geological conditions and the needs for construction and operation. It should avoid karst and abundant groundwater zones.

12.1.3 The form, length and number of auxiliary channels shall be determined by technical and economic comparison based on tunnel length, terrain, geology and hydrology coupled with ventilation, disaster prevention and rescue, drainage, muck removal, duration and environmental requirements.

12.1.4 The cross-section of auxiliary channel for operation purpose shall be determined by geological conditions, function and construction conditions. When the operational auxiliary channel also functions as construction auxiliary channel, it shall meet minimum cross-section requirements of construction auxiliary channel.

12.1.5 The operational auxiliary channel shall be designed as permanent structure and should be provided with composite lining or shotcrete and rockbolt lining. It shall also be provided with sound waterproofing and drainage facilities.

12.1.6 The cross-section of construction auxiliary channel shall be determined according to geological conditions, construction machinery, construction length of length of auxiliary channel serving the main tunnel construction, construction ventilation and working environment requirements.

12.1.7 For a construction auxiliary channel, appropriate lining structure shall be selected according to surrounding rock geological conditions so as to meet surrounding rock stability and lining safety requirements during construction. For a temporary construction auxiliary channel that is not intended to be used after completion of main tunnel, the following shall be considered during design:

1 Where the temporary ground support cannot achieve the design life of the adjacent operational tunnel, the auxiliary channel shall be reinforced or backfilled. If it is backfilled, drainage shall be provided. If it is not backfilled, the long term stability of surrounding rock and lining shall be ensured and an access maintained for inspection and maintenance personnel.

2 At the intersection of construction auxiliary channel and main tunnel and at portals, a safety door shall be provided to prevent unauthorized access.

12.1.8 The selection of the location of auxiliary channel (shaft)'s entrance/exit leading to the outside of the tunnel, site layout and muck disposal shall meet environmental requirements, prevent muck from blocking river, canal or road and minimize the impact on farmland, water conservancy facilities and supply of domestic water. The channel (shaft)'s entrance/exit shall be protected against surface water flowing into the channel to prevent flooding.

12.1.9 The principle of waterproofing and drainage design for inclined shaft and vertical shaft should be a combination of sealing and draining measures.

12.2 Vertical shaft

12.2.1 The location of vertical shaft shall depend on topographical and geological conditions. It should be on either side of the tunnel.

12.2.2 The shaft site shall accommodate the layout of hoisting system and shaft structures and facilitate ventilation and muck removal.

12.2.3 The cross-sectional shape and size shall meet functional requirement, taking into consideration the space required for construction equipment and operations. The cross-section should be circular in shape.

12.2.4 The structural form of vertical shaft lining shall be determined based on surrounding rock class and service requirements. Composite lining should be used. Support parameters may be determined by engineering analogy or selected from Table P.0.7 in Appendix P hereto.

Waterproofing layer may not be provided for vertical shaft lining.

12.2.5 The top of the shaft shall be provided with concrete or reinforced concrete capping beam. The underside of the capping beam should be provided with enlarged foundation that is integrated with the capping beam.

12.2.6 Composite or monolithic lining shall be employed for connections with the connecting passage at the formation level of the shaft.

12.2.7 Shaft cribs should be provided below the capping beam, along the shaft wall where the geological condition is poor and at the cast-in-situ concrete lining above chamfered section between shaft and tunnel.

12.2.8 Safety protection facilities and step ladder or cat latter for inspection shall be provided. Safety step ladder shall be provided in the shaft at construction stage.

12.3 Inclined shaft

12.3.1 The layout and length of inclined shaft shall be determined based on its intended purpose, terrain, geology, hoisting method, shaft site, etc. and shall meet the following specifications:

1 The rail haulage section should be straight; the straight section should not be longer than 1,200m.

2 The trackless haulage length should not be greater than 2,000m.

3 The shaft site shall accommodate the layout of hoisting system and shaft structures and facilitate air venting and muck removal.

12.3.2 The cross-sectional shape and size of the inclined shaft shall meet its functional requirements. Its cross-section should be of a horseshoe shape. During construction, it shall meet the space requirements for construction equipment and operations as well as the following specifications:

1 A sidewalk not less than 0.75m wide shall be provided on one side in the cross-section; a gap not less than 0.25m wide shall be provided on the other side.

2 In the case of rail haulage, the distance between centerlines of two tracks shall not be less than 0.7m. In a yard with attachment/detachment operations, the clearance between two

trains at the closest point shall not be less than 0.2m.

12.3.3 The hoisting method shall be selected according to hoisted amount, length of inclined shaft and shaft top terrain. The dip angle of inclined shaft:

1 should not be greater than 35° in the case of hoisting by rail skip;

2 should not be greater than 25° in the case of hoisting by rail mining car;

3 should not be greater than 15° in the case of hoisting by belt conveyor;

4 should not be greater than 7° in the case of trackless haulage.

12.3.4 The distance between the bottom of inclined shaft and main tunnel shall be determined based on functional requirement. Horizontal angle of intersection of the inclined shaft and tunnel centerline should not be less than 40°.

12.3.5 Vertical curves shall be set off an inflection point at shaft top and bottom. Its radius should be 12-20m. In the case of rail haulage, longitudinal gradient of the shaft in haulage sections should be consistent.

12.3.6 If rail hoisting is adopted, refuge shall be provided. If dip angle is greater than 15°, landing shall be provided. If trackless haulage is adopted, passing bays shall be provided as needed.

12.3.7 The structural form of the tunnel lining shall be determined based on functional requirement. Support parameters may be determined by engineering analogy or selected from Table P.0.8, within Appendix P. When the shaft is used for operational ventilation, its intrados shall be flat and smooth.

12.3.8 Composite or monolithic lining should be employed at shaft top, in zones of poor geology and at the connection of shaft bottom and horizontal cross passage.

12.3.9 If monolithic or composite lining is employed, deformation joints shall be placed vertically depending on the length of continuous lining.

12.3.10 For an inclined shaft with a dip angle greater than 30°, its lining foundation should be made into steps or provided with foundation support.

12.3.11 If rail haulage is adopted in an inclined shaft with a dip angle greater than 15°, appropriate safety measures must be taken by providing stop device at appropriate locations; rails shall be fitted with sufficient anti-slip measures for the degree of inclination.

12.4 Parallel adit and cross passage

12.4.1 Parallel adits and/or cross passages may be provided according to construction and operation needs.

12.4.2 For a single-tube tunnel, any parallel adits should be located on the side of groundwater recharge source. For a twin-tube tunnel, parallel adit shall be determined by the spacing between the two tubes and topographical conditions.

12.4.3 The clear distance between any parallel adit and the tunnel(s) shall be determined based on geological conditions, construction methods, operation, evacuation, rescue and other factors. For a single-tube tunnel whose parallel adit may be expanded as an access tunnel under planning, the clear distance shall be beneficial to such expansion.

12.4.4 Any parallel adit should be consistent with main tunnel in longitudinal gradient; its invert elevation should be 0.2 ~ 0.6m below that of the main tunnel. Parallel adit drainage design shall be coordinated with main tunnel drainage design.

12.4.5 The cross-section of any parallel adit shall be determined based on functional requirement; passing bays shall be provided according to means of haulage.

12.4.6 Tunnels alongside river and mountainous terrain, or where there may be a valley and/or low-lying ground available for use on one side of the tunnel, a cross passage may be constructed to connect main tunnel to the ground surface. If the cross passage has an upward gradient to the ground surface, reliable cut-off, extraction and drainage measures shall be put in place.

12.4.7 Portal and approach section of cross passage shall be designed in accordance with relevant specifications under Chapters 7 and 8 herein.

12.4.8 The cross passage should be positioned to avoid passing through or running along faults, highly fractured zones and other unfavorable ground conditions.

12.4.9 Lining parameters for parallel adit and cross passage may be determined by engineering analogy or selected from Table P.0.8 in Appendix P hereto.

12.5 Duct and underground machine room

12.5.1 Duct shall be designed as specified below:

1 Duct should be provided with monolithic or composite lining; its intrados shall be smooth.

2 Where the cross-section changes, such as at bends, tapers, enlargements and branches, the change in cross section should be formed via a continuous variable cross-section curve. If abrupt connection between different cross-sections is made, a transition wall shall be provided.

3 The duct bulkhead should be constructed of concrete and integrated with the structural tunnel lining.

12.5.2 Duct lining parameters may be determined by engineering analogy and functional requirement, or selected from P.0.8 in Appendix P hereto.

12.5.3 Design of tunnel ventilation fan (TVF) rooms shall meet the following requirements:

1 TVF rooms should be located near the tunnel provided that the surrounding rock and lining structure remain stable.

2 The TVF room shall provide sufficient space necessary for equipment layout, erection, handling and service channel.

3 If a crane is to be fitted for equipment erection, the longitudinal gradient of the cavern should be level.

4 Where the fan is to be installed in phases, space shall be reserved.

12.5.4 The TVF room lining may be of shotcrete-and-rockbolt or composite type. The lining parameters may be determined by engineering analogy or calculation according to the size of the TVF room. If lifting equipment is to be used, composite lining shall be employed and the secondary lining shall designed to withstand the equipment lifting loads. Where there are special requirements for equipment installation, bespoke designs may need to be developed.

12.5.5 Waterproofing and drainage for duct and TVF room shall meet functional requirements;

the drainage gradient of side drains shall not be less than 0.3%.

12.6 Intersection

12.6.1 The intersection of main tunnel and cross passage used for construction and operation, the intersection of cross passage and inclined shaft or parallel heading, the intersection of duct and main tunnel, etc. should avoid poor geological conditions.

12.6.2 The lining at intersections should be of monolithic or composite type and shall be provided with settlement joints as necessary.

13 Auxiliary Engineering Measures

13.1 General

13.1.1 Where the tunnel passes through shallow sections, severe eccentrically loaded sections, soft stratigraphy with poor self-stability, faults, heavily fractured zones and large areas of water spray or inflow, the following auxiliary engineering measures may be employed:

1 Surrounding rock stabilization measures including forepoling, advanced tremie, advance rockbolt, advance drilling and grouting, advance horizontal jet grouting pile, advance glass fiber rockbolt, surface mortar rockbolt, surface grouting, rockbolt at foot of wall, radial grouting through advance pipes and temporary support.

2 Inflow treatment measures including, but not limited to, advance pre-grouting into surrounding rock, radial grouting into surrounding rock, drainage holes drilled ahead of the face, diversion of groundwater through drainage tunnels and well point dewatering.

13.1.2 Appropriate auxiliary engineering measures shall be taken depending on topographical and geological conditions, tunnel cross-section size, depth and construction methods. Engineering measures adopted shall be effective, reliable, durable, economical and suited to field realities.

13.2 Surrounding rock stabilisation measures

13.2.1 For tunnel portal sections in poor ground, low cover, high risk of rockfall, sections requiring high settlement control, or sections within highly fractured rock or soil formations, forepoling may be used. The forepoling shall be designed as specified below:

1 Forepoling shall be 100 ~ 200mm beyond the tunnel excavation profile, with a defined

'look-out' angle with respect to the tunnel axis. The magnitude of the angle should ensure the forepoling steel pipes do not encroach into the tunnel excavation profile.

2 The circumferential spacing of steel pipes should be between 350 ~ 500mm.

3 One round of forepoling should be 10 ~ 45m long. A horizontal overlap not less than 3.0m long shall be provided between two consecutive rounds of pipes, and between forepoling steel pipes and other advance supports.

4 Hot-rolled seamless steel pipes of 80 ~ 180mm in outer diameter should be used. Steel pipes should be joined by "V" butt welding or threaded connection. Each pipe should be 1.6 ~ 4.0m long. Each joint of the pipes shall be staggered by not less than 500mm with adjacent steel pipe joints.

5 Reinforcing steel cage or rebar bundle shall be inserted into the steel pipe before injecting the steel pipe with cement mortar not less than M20 in strength grade.

6 Grouting holes may be drilled in the steel pipe wall. These holes should be 6-10mm in diameter and arranged at a spacing of 200 ~ 300mm in quincunx form.

7 At the tunnel portal, the end of the forepoles shall be supported by an arch. The arch shall be formed by monolithic reinforced concrete or steel rib frame. Steel guide tubes shall be embedded in the arch. The arch foundation shall ensure its stability.

13.2.2 In sections where the tunnel face is not self-standing after tunnel excavation, or the arch is susceptible to raveling or local collapse as well as sections with the potential to cave-in, shallow section and portal section with poor geological condition, advance anchors may be used. The advanced tremie shall be designed as specified below:

1 Seamless steel tube of 42 ~ 50mm diameter and 3.0 ~ 5.0m length should be used.

2 Grouting holes shall be drilled in the tube wall. These holes should be 6-8mm in diameter and arranged at 150-250mm intervals in quincunx form. A length of at least 500mm at the tail end shall be free of holes.

3 Circumferential spacing between elements should be 300-400mm, with an angle of 5° ~ 12° to horizontal. Longitudinal horizontal lap length shall not be less than 1.0m.

4 One end shall be supported by the steel frame.

5 Surrounding rock shall be grouted through the conduit.

13.2.3 In soft stratigraphy with no groundwater, thin horizontal layered rock formations and zones where rock arch may unravel within hours of excavation or local collapse is possible, advance rockbolts may be used. Advance rockbolts shall be designed to meet the following requirements:

1 Ordinary mortar rockbolts of 22 ~ 28mm diameter should be used. Where the rock mass is highly fractures in which the drill hole is not stable, self-drilling rockbolts of 28-76mm diameter may be used.

2 The length should be 3.0 ~ 5.0m. The length of self-drilling rockbolts, if used, should be 5.0 ~ 10m.

3 Circumferential spacing should be 300-400mm, with an angle of 5°-15° to the tunnel axis. Longitudinal horizontal lap length shall not be less than 1.0m.

4 One end shall be supported by steel frames.

6 Rapid hardening mortar should be used. Its strength grade shall not be lower than M20.

7 Self-driving rockbolts shall be grouted with grout whose strength grade shall not be lower than M20.

13.2.4 In weak rock masses, fractured zones within faults, cohesionless soil strata and where the tunnel excavation may cause mud burst and flowing at the face, the surrounding rock or excavated face may be reinforced by advance drilling and grouting. Advance drilling and grouting shall be designed to meet the following requirements:

1 Determination of extent of strengthening and selection of grout material shall be based on geological and groundwater conditions.

2 Grouting holes shall be arranged according to extent of strengthening, grout material, grouting radius and engineering requirements, such that the grouted areas overlap with each other.

3 Grouting should be limited to 3.0m beyond the excavation line.

4 The grout hole diameter shall not be less than 75mm; grouting pressure shall be determined by field test.

5 Longitudinal length of one grouting operation may be 30 ~ 50m.

13.2.5 In silty clay, clayey material, silt and sandy soil with high moisture content, advance horizontal jet grouting piles may be used. Such piles shall be designed as specified below:

1 Circumferential or full-face reinforcement may be employed as needed.

2 The diameter of jet grouting piles should be 0.3 ~ 1.0m for single tube method, 0.6 ~ 1.4m for double tube method and 0.7 ~ 2.0m for triple tube method.

3 In the case of circumferential reinforcement, external dip angle of jet grouting pile should be 3° ~ 10°, and circumferential spacing shall be such that grout in adjacent holes can overlap to form an arch structure.

4 The length for one operation should be 10 ~ 20m; the overlap length for each round shall not be less than 2.0m.

5 If tensile and bending strength of jet grouting piles need to be increased, structural steel, reinforcing steel cage, rebar bundle or steel pipe may be inserted into the jet grouting piles.

13.2.6 In soft stratigraphy where large- or full-face excavation is adopted and shallow tunnel sections where tight control of surface settlement is required, advance glass fiber rockbolt may be installed ahead of the tunnel face for forepoling. The advance glass fiber rockbolt shall be designed as follows:

1 If forepoling or advanced tremie has already been installed, the extent of strengthening should be within the face.

2 Rockbolt spacing should be 1.0 ~ 3.0m within the face and 300 ~ 600mm in the tunnel surrounding rock area, subject to adjustment depending on surrounding rock stability.

3 Longitudinal reinforced length should be 10 ~ 30m; the overlap length for each round shall not be less than 6.0m.

4 The diameter should be 18 ~ 32mm for fully threaded solid rockbolt and 18 ~ 60mm for fully threaded hollow rockbolt.

5 In poor geological conditions, hollow grouted rockbolts with grout or cement mortar should

be used.

6 The face shall be properly drained and monitored for longitudinal squeezing displacement.

13.2.7 In shallow sections and approach sections with poor stability, strata may be reinforced with surface mortar rockbolts whose design shall be as specified below:

1 They should be placed vertically and may also be inclined depending on terrain and main structure plane conditions.

2 They should be one deformed bar of 16 ~ 22mm diameter or formed by welding several such bars side by side and arranged in quincunx form at 1.0 ~ 1.5m intervals.

3 The hole diameter shall be 30mm greater than the bolt diameter; the grade of mortar injected shall not be lower than M20.

4 The rockbolt shall extend 5-10m beyond unfavorable geological zone in longitudinal direction and should extend to a distance determined by calculated fracture plane or 1-2 times the excavation width in transverse direction.

5 The rockbolts should not encroach on tunnel excavation profile.

6 Excavation of the tunnel below shall start after the anchoring mortar has exceeded 70% of design strength.

13.2.8 In shallow or portal sections with loose strata, unstable surrounding rock, low self-stability of the face and the possibility of cave-in during tunneling, pre-excavation grouting may be performed from the surface to stabilize the ground. Surface grouting design shall be as specified below:

1 Grouting holes shall be vertical.

2 Grouting hole diameter should not be less than 110mm.

3 Grouting hole depth shall be at least 1.0m below the tunnel excavation invert level.

4 The extent of stabilization shall be 5-10m beyond unfavorable geological zone in longitudinal direction and may extend 1.5 ~ 2.0 times the tunnel width in transverse direction.

5 Hole spacing should be 1.4 ~ 1.7 times the grout radius of a single hole; grouting holes may be arranged in quincunx or rectangular array.

6 Grouting pressure may be determined by field test.

13.2.9 In sections supported by steel ribs, rockbolts (anchoring pipes) should be installed at the foot of the walls to improve stability of the bearing surface; its design shall be as specified below:

1 Groups of 2no. rockbolts (anchoring pipes) at the foot of wall shall be arranged at the bottom or joint of steel rib support and welded to the steel ribs.

2 The direction of resultant force of 2no. rockbolts (anchoring pipes) at the foot of walls shall form an angle of 15 ~ 30° with primary support axis.

3 The rockbolts should be φ22 ~ φ32mm deformed bar. The anchor pipes should be φ42 ~ φ54mm seamless steel pipe with a wall thickness not less than 3.0mm. The rockbolts (anchoring pipes) should be 2.5 ~ 4.0m long.

4 The rockbolt holes or anchoring pipes shall be fully grouted with grout of the same strength grade as that for ordinary mortar rockbolt.

13.2.10 In highly fractured surrounding rock or sections with poor bonding between rock strata, radial grouting through small pipes may be performed to stabilize surrounding rock. The pipes should not be less than 3.5m long and should be at 1.0 ~ 2.5m intervals.

13.2.11 In the case of large deformation during construction, complex transition between construction sequences or emergency rescue, temporary closure and support measures may be taken. Such measures shall be effective, easy to construct and remove later.

13.3 Inflow treatment measures

13.3.1 Inflow treatment in tunneling shall follow the principle of "sealing as primary consideration, in combination of sealing and draining with attention on environment protection". Inflow treatment measures shall be selected based on topographical and geological conditions, tunnel structural type, size of cross-section and environmental requirements.

13.3.2 In fractured or weathered zones where high groundwater inflow is expected, draining such flows could lead to loss of fines from the rock joints, leading to its instability of the ground, or where such drainage may have a significant impact on groundwater or surface water around the tunnel, pre-excavation grouting into surrounding rock mass may be utilized to control ground water inflow This measure shall be designed to meet the following requirements:

1 Depending on engineering, hydrogeological conditions and other factors, advance full-face curtain grouting, advanced peripheral pre-grouting or advance localized pre-grouting may be selected.

2 The grouting ring thickness for curtain grouting and peripheral grouting should reach 3 ~ 6m beyond the theoretical excavation line; the length of one grouting operation may be 10 ~ 30m.

3 The spacing of grouting hole bottoms from center to center should be 1.5 ~ 3.0m or 1.5 ~ 1.7 times the grout radius.

4 Grout penetration should be determined by field test according to stratum porosity, fissure and its connectivity, grouting pressure and grout type, or selected by engineering analogy method, subject to modification during construction.

13.3.3 In situations where stable rock masses experience high groundwater inflow that primarily affects the aquifer surrounding the tunnel and surface water, radial grouting into the surrounding rock can be carried out after excavation to mitigate water inflow. The design specifications for this grouting process are outlined below:

1 Full-face radial grouting, local radial grouting or supplementary grouting may be selected depending on surrounding rock geological conditions, inflow type, inflow rate and waterproofing and drainage requirements.

2 Grouting ring thickness should be between 2.0m and 6.0m from the excavation line.

13.3.4 In zones where high pressure groundwater or inrush with sufficient recharge source is present ahead of the face and groundwater discharge, but has little influence on the surrounding rock stability and aquifer, drainage boreholes may be drilled. Its design shall be as specified below:

1 Hole diameter shall not be less than 76mm; the number of boreholes in each section shall not be less than 3.

2 Depth should not be less than 10m;

3 Where the end of the borehole is less than 1-2 round lengths ahead of the face and advance borehole drainage remains necessary, advance borehole drainage for the next round shall be implemented.

4 The borehole head shall be provided with protective device against inrush.

13.3.5 In a tunnel with high ground water head and recharge, seasonal variations in groundwater inflow, insufficient discharge capacity, or where any of the above groundwater control measures are insufficient to manage groundwater inflow, a drainage tunnel may be used for drainage. Its design shall be as specified below:

1 It may be located on either side of the tunnel or below the tunnel. If there are several inflow zones along the tunnel length, the drainage tunnel should be parallel or nearly parallel to main tunnel. For a tunnel with underground river or concentrated inflow points, it may be arranged transversely where conditions allow. Its arrangement shall not undermine the stability of the surrounding rock and tunnel structure.

2 The longitudinal gradient shall allow gravity drainage and should not be less than 0.5%.

3 The outfall shall be located such that no damage is done to the downstream area.

4 Its invert elevation shall be lower than that of main tunnel.

13.3.6 During the construction of shallow tunnels in cohesionless soil strata, where the groundwater table is within 3.0m above the invert of tunnel excavation and there is a known recharge source, well point dewatering can be employed. The design specifications for this dewatering process are outlined below:

1 Well point location, depth and number shall be determined based on stratum permeability, dewatering zones, groundwater volume and other factors.

2 Well point dewatering boreholes shall be arranged on the ground surface along two sides of the tunnel; more boreholes may be arranged on the side with recharge source.

3 After dewatering, the water table shall be lowered to 0.5 ~ 1.0m below the invert of the tunnel excavation line.

14 Design of Tunnels in Special Geology

14.1 General

14.1.1 When tunneling through special geological conditions such as swelling surrounding rock, karst, goaf, running sand, methane gas and other toxic gases, loess, high in-situ stress zone and permafrost, appropriate auxiliary engineering measures shall be taken to ensure structural and construction safety.

14.1.2 For a tunnel passing through special ground, apart from special design, observation of groundwater table and monitoring of rock deformation and support lining deformation or stress are also required during construction. In case of discrepancy between design assumptions and actual observed condition, the design shall be corrected promptly.

14.2 Swelling surrounding rock

14.2.1 The cross-section should be in circular or nearly circular shape.

14.2.2 The support structure design shall follow the philosophy of "installing flexible support before rigid support, allowing deformation before jacking and applying support in layers".

14.2.3 In zones of large swelling deformations, a two-pass system of primary support may be employed; retractable steel rib support may also be used within primary support; both long and short rockbolts should be used; the length of rockbolts should be increased and their spacing decreased.

14.2.4 Deformation allowance in tunnel excavation shall be dependent on the amount of swelling surrounding rock deformation and larger than that in normal surrounding rock. The size of steel rib support shall increase with increasing excavation cross-section to fit excavation outline.

14.2.5 Acomposite lining shall be employed. Secondary lining should have a reinforced concrete structure. Both primary support and secondary lining shall be provided with arch invert.

14.2.6 Water interception and drainage measures shall be taken to minimize swelling deformation of rock in contact with water.

14.3 Karst

14.3.1 Depending on the location of karst with respect to the tunnel, cavity-crossing, cavity strengthening, cavity-backfilling, cavity water channeling, draining or intercepting, cavity infill removal or strengthening, backfill surface sinkhole or surface water drainage or a combination thereof may be adopted.

14.3.2 When the tunnel alignment crosses a large solution cavity or underground river, it may pass over this cavity or river.

14.3.3 Where a large hollow solution cavity is present above the arc crown, the cavity wall may be reinforced by shotcrete-and-rockbolt depending on the degree of its stability; lining extrados shall be backfilled or provided with forepoling or elephant foot, depending where it is used. Where hollow cavities are present on either side of the tunnel, such measures as increasing lined side wall thickness and applying concrete or stone pitching against the wall may be taken.

14.3.4 Infilled solution cavities at the tunnel bottom shall be addressed by pile foundation, grouting, replacement, crossing and other measures depending on infill characteristics and the relative location of the cavities to the tunnel.

14.3.5 Karst water shall be intercepted, directed and discharged as appropriate. The original karst water channel shall be protected, dredged and restored.

14.4 Goaf

14.4.1 Mutual impact between goaf and tunnel shall be analyzed according to the conditions of surrounding rocks in which the goaf is located, the type, size, stability of the goaf and its relationship with the tunnel. On this basis, appropriate support structure and engineering measures shall be selected.

14.4.2 When tunneling through goaf, such measures as crossing, reinforcing surrounding rock, reinforcing support structure for the goaf, sealing and backfilling the goaf and discharging ponding may be taken. When tunneling through a goaf containing toxic gases, treatment measures shall be taken according to requirements for disposal of toxic gases.

14.4.3 The tunnel lining structure within the zone of influence from goaf should be reinforced. The tunnel lining in a goaf with toxic gases shall be air-tight.

14.4.4 For an unmined zone overlaid or crossed by the tunnel, the boundary of no mining shall be specified.

14.5 Running sand

14.5.1 When tunneling through running sand strata, appropriate tunnel support structure and engineering measures shall be selected based on running sand characteristics, scale, penetration test results, relative density, grain size distribution, plasticity index, load-bearing capacity of the ground, aquitard distribution, groundwater pressure, permeability and other factors.

14.5.2 Design of tunneling through running sand strata shall be as specified below:

1 Ground drainage shall be enhanced. Groundwater table should be lowered to 0.5m below the tunnel invert.

2 Plugging measures shall be provided in case of overflow of running sand.

3 Surrounding rocks near the running sand overflow shall be reinforced.

4 Surrounding rocks containing running sand shall be advance-supported.

5 Steel rib support used for primary support should be closed into a ring. In the case of partial excavation, temporary floor beam or temporary arch invert shall be provided.

6 To control arch settlement in running sand strata, timber support or steel truss supports may be used as temporary vertical support.

7 Secondary lining shall be constructed of reinforced concrete.

14.6 Gas and other toxic gases

14.6.1 When tunneling through ground containing methane gas and other toxic gases, extraction, isolation, sealing and reinforcement measures shall be taken according to gas content, inflow and pressure; advanced observation, construction ventilation, gas detection, etc. shall be designed.

14.6.2 The tunnel lining shall have a composite closure structure with arch invert, with increased impermeability of secondary cast-in-situ concrete lining. The composite closure structure shall extend not less than 20m into zones without gas or other toxic gases.

14.6.3 Construction joints in secondary cast-in-situ concrete lining shall be as at least gas-tight as the concrete lining. Where a two-pass system of cast-in-situ concrete linings is employed, construction joints in the two layers of lining shall be staggered by at least 2.0m.

14.6.4 In gas stratum, the shotcrete thickness shall not be less than 150mm; secondary cast-in-situ concrete thickness shall not be less than 400mm.

14.6.5 Embedded parts and reserved caverns shall be arranged such that the impervious performance of the lining structure is not compromised.

14.6.6 To address other toxic gases, design should be based on specific situations and comply with sealing and plugging principles and ventilation requirements for gassy tunnels.

14.7 Loess

14.7.1 The lining structure of a tunnel in loess shall be dependent on the type, physical and mechanical properties, natural water content of loess, tunnel cross-section size, construction methods, etc.

14.7.2 Loess tunnels should be provided with a composite lining structure with curved wall and arch invert. Where systematic rock bolting is not installed, steel rib support shall be enhanced and rockbolts at foot of the walls added. The secondary lining should be constructed of reinforced concrete.

14.7.3 In the case of inadequate ground bearing capacity, such measures as installing rockbolt (pipe anchors) at the foot of the walls, increasing arch footing or elephant foot section and installation of steel pipe piles may be taken to prevent overall settlement of the lining structure.

14.7.4 Surface gully, sink holes and cracks affecting the tunnel shall be backfilled and paved. Surface water drainage facilities shall also be provided.

14.7.5 For a tunnel below groundwater table, a combination of lowering, drainage and plugging measures shall be taken according to loess property and groundwater characteristics.

14.7.6 Collapsible loess ground may be stabilized by such measures as replacement with lime earth, compaction piles, jet grouting piles, root piles and steel tube piles.

14.7.7 Loess tunnel portal design shall be as specified below:

1 The toe of portal side and heading slopes and foundations susceptible to scouring shall be paved. Side and heading slopes shall be connected by an arc.

2 Collapsible loess ground shall be stabilized by replacement or compaction piles depending on its physical and mechanical properties and portal type. Where non-collapsible loess ground has insufficient bearing capacity, replacement, extended foundation and other stabilization measures may be taken.

14.8 High in-situ stress zone

14.8.1 Design of tunnels in high in-situ stress zones shall be as specified below:

1 The horizontal projection angle of tunnel axis and maximum principal stress direction should be less than 30°.

2 The cross-section of lined tunnel shall be in a nearly circular shape.

14.8.2 Tunnels in high in-situ stress zones shall be classified by possible rock burst in hard rock and large deformations in soft rock taking into account in-situ stress intensity, hydrogeological and surrounding rock conditions. Appropriate excavation method and control measures shall be selected according to the classification. Rock burst and large deformation classification may be determined per Tables 14.8.2-1 and 14.8.2-2.

Table 14.8.2-1 Rock burst classification

Rock burst class	Description	Criteria
I	Slight rock burst	$0.3 \leq \sigma_{\theta\max}/R_b < 0.5$
II	Moderate rock burst	$0.5 \leq \sigma_{\theta\max}/R_b < 0.7$
III	Strong rock burst	$0.7 \leq \sigma_{\theta\max}/R_b < 0.9$
IV	Violent rock burst	$0.9 \leq \sigma_{\theta\max}/R_b$

Note: $\sigma_{\theta\max}$ is maximum tangential stress in tunnel wall; R_b is uniaxial compressive strength of rock.

Table 14.8.2-2 Large deformation classification

Large deformation class	Description	Criteria (%)
Class I	Slight large deformation	$2 \leq U_a/a < 3$
Class II	Moderate large deformation	$3 \leq U_a/a < 5$
Class III	Strong large deformation	$5 \leq U_a/a$

Note: U_a is amount of deformation; a is tunnel width.

14.8.3 Rock burst treatment shall follow the principle of "prevention as primary consideration in combination with control". Zones of possible rock burst shall be monitored and probed. The following measures shall be taken according to rock burst classification:

1 In zones with slight and moderate rock bursts, primary support may consist of reinforcement mesh shotcrete or fiber reinforced shotcrete, systematic rock bolting, advance rockbolts, etc.

2 In zones with moderate rock burst, the tunnel face and surrounding rock may be sprayed with water or injected with water after drilling water injection holes; steel lattice girder may be added.

3 In zones with strong rock burst, the tunnel face and surrounding rock may be sprayed with water or injected with water after drilling water injection holes; stress relief holes may be drilled in the face; comprehensive control measures such as reinforcement mesh shotcrete or fiber reinforced shotcrete, systematic rock bolting, advance rockbolt rows and enhancing steel rib support may be taken.

4 In zones with violent rock burst, yielding support system shall be employed, and such measures as advanced stress relief and water injection under high pressure shall be taken to reduce In-Situ Stress magnitude.

14.8.4 Large deformation prevention and control shall follow the principle of "reinforcing surrounding rock, allowing for deformation, flexible before rigid, release before resistance, phased support, early closure and invert strengthening". The following measures shall be taken depending on large deformation classification:

1 In zones with slight large deformation, such measures as combination of long and short rockbolts, reinforcement mesh shotcrete or fiber reinforced shotcrete, installing steel rib support and reinforcing secondary lining may be taken.

2 In zones with moderate large deformation, such measures as long rockbolts, reinforcement

mesh shotcrete or fiber reinforced shotcrete, retractable steel rib support and secondary lining may be taken.

3 In zones with strong large deformation, such measures as ground pre-stabilization, partial excavation, long rockbolts, reinforcement mesh shotcrete or fiber reinforced shotcrete, shotcrete layer with longitudinal joints, retractable steel rib support, adding cushion and secondary lining may be taken.

4 In zones with moderate or strong large deformation, such measures as two or more pass shotcrete-and-rockbolt support, adding anchor rope and increasing deformation allowance may be taken as appropriate.

14.9 Permafrost

14.9.1 Tunnels in permafrost zones shall be arranged as specified below:

1 Where possible, the tunnel should be located in a location with low groundwater table, dry surrounding rock and small impact of freeze and thaw on surrounding rocks.

2 The portal site should avoid ice cone, ice mound, permafrost swamp, etc.

3 The tunnel alignment along an underground ice layer shall be avoided.

14.9.2 The portal foundation shall be placed at 1.0m below the frost heave or thaw collapse line. Thick underground ice behind the portal wall, if present, shall be removed and replaced.

14.9.3 The ratio of portal side and heading slopes shall be determined based on frost conditions; side slopes excavated surface shall receive thermal protection treatment.

14.9.4 For a tunnel in permafrost, composite lining with curved wall and arch invert shall be employed. Based on frost property and analysis of frost heave hazard, the clear section may be enlarged appropriately; structural reinforcement space may be reserved or two pass cast-in-situ concrete lining may be employed.

14.9.5 Cast-in-situ concrete lining should be constructed of low-temperature, early strength and frost resistant concrete that meets impervious performance, frost resistance and durability requirements. Admixtures shall not corrode reinforcement bars. The impermeability grade of concrete should not be below P10.

15 Tunnel Subgrade and Pavement

15.1 General

15.1.1 Tunnel subgrade shall be stable, dense and homogeneous to provide pavement structures with uniform support.

15.1.2 Tunnel pavement shall have sufficient strength, be flat, durable, skid-resistant and wear-resistant.

15.1.3 Tunnel pavement structure shall be determined through economical and technical comparison based on such factors as traffic volume, design speed, horizontal and vertical alignment parameters, local environmental conditions, material supply and life cycle cost analysis.

15.1.4 Drainage shall be provided below the tunnel pavement structure.

15.2 Tunnel subgrade

15.2.1 For a tunnel provided with arch invert, the arch invert filling layer may be the subgrade layer. The filling materials and filling requirements shall comply with relevant provisions of Chapters 5 and 8 herein.

15.2.2 For a tunnel without arch invert, the subgrade shall be a stable stone foundation.

15.3 Tunnel pavement

15.3.1 Expressway and Class-1 highway tunnels should have composite pavement composed of upper asphalt mixture course and lower concrete course. For other classes of highways, the tunnels may have composite pavement or cement concrete pavement.

15.3.2 The tunnel pavement structure shall be determined in the light of tunnel structure and geological conditions. For a tunnel without arch invert, its pavement shall consist of both base course and surface course, and a leveling course may be added as demanded. For a tunnel with arch invert, its pavement may consist of only base and surface courses.

15.3.3 The base course of pavement shall be designed as follows:

1 In a tunnel without arch invert, the base course of pavement shall be placed on solid foundation.

2 A base course should be constructed of plain concrete. It should have a thickness of 150 ~ 200mm, a compressive strength grade of not lower than C20 or flexural-tensile strength of not lower than 1.8MPa. It shall be provided with transverse contraction joints corresponding to the concrete surface course.

3 If a paving width is greater than 7.5m, longitudinal contraction joints shall be provided.

4 When a leveling course is added, the average thickness of the leveling course should not be less than 150mm.

15.3.4 Tunnel pavement to be provided with cement concrete surface course shall conform to the following provisions:

1 Classes II, III and IV highways should be provided with jointed cement concrete surface courses. The thicknesses of cement concrete surface courses should be within a range of 200 ~ 220mm on Classes III and IV highways and 220 ~ 240mm on Class-2 highways. The concrete should have a strength grade of C35 ~ C40 and flexural strength of 4.0 ~ 4.5MPa on Classes III and IV highways and a strength grade of not less than C40 and flexural strength of 4.5 ~ 5.0MPa on Class-2 highways.

2 Expressways and Class-1 highways shall be provided with continuously reinforced concrete surface courses or steel fiber reinforced concrete surface courses. The thickness of a cement concrete surface course should be within a range of 240 ~ 260mm. The concrete should have a strength grade of C40 ~ C50 and flexural strength of not less than 5.0MPa.

3 The thickness of surface course, the joint construction and spacing, the amount of steel

fibers added to concrete, and the reinforcement of special parts of surface course shall meet relevant provisions of the current *Specifications for Design of Highway Cement Concrete Pavement* (JTG D40). Expansion joints shall be provided in portal section. Where the lining structures change, transverse joints shall be set uniformly in combination with the lining deformation joints.

4 The design criteria for structural reliability, the material properties and structural parameters and their variable levels, the design methods, the standard axial loads, the material composition and the property parameters of cement concrete pavement of all grades shall conform to relevant provisions of the current *Specifications for Design of Highway Cement Concrete Pavement* (JTG D40). The coarse and fine aggregates for cement concrete pavement of expressway and Class-1 highway tunnels should be Grade I.

5 The texture depth of pavement surface shall meet the relevant provisions on special road sections in the *Specifications for Design of Highway Cement Concrete Pavement* (JTG D40) currently in force during handover and acceptance. The texture depth of pavement under unfavorable conditions shall be given a big value. Surface texture shall be wear-resistant. If grooving method is used, longitudinal grooves should be made. Longitudinal and transverse grooves should be made simultaneously for expressway, Class-1 highway, tunnel entrance section and tunnel with a steep slope. If composite pavement is used, the cement concrete pavement structure as the lower surface course is not subject to this clause.

6 For overlay of concrete pavement, cement concrete overlay structure or asphalt concrete overlay structure shall be selected and used through technical and economical comparison based on use requirements and old concrete pavement conditions. The overlay structure shall be designed in accordance with relevant provisions in the current *Specifications for Design of Highway Cement Concrete Pavement* (JTG D40) and *Specifications for Design of Highway Asphalt Pavement* (JTG D50).

15.3.5 The reinforcement for the continuously reinforced concrete surface course should meet the following provisions:

1Cold-rolled ribbed rebar welded mesh (rebar diameter: 8 ~ 12mm) should be used, and longitudinal and transverse cold-rolled ribbed rebar with a diameter of 12 ~ 20mm also can be used. Reinforcement may be determined as per Eq. (15.3.5) and the minimum reinforcement ratio should not be less than 0.15%.

$$A_s = \frac{16 L_s h \mu}{f_{sy}} \tag{15.3.5}$$

Where,

A_s—Area of rebar required per linear meter of width or length of concrete surface course (mm^2).

L_s—Distance between transverse joints for longitudinal rebar (m); distance between free edges or between longitudinal joints without tie bars for transverse rebar (m).

h—Thickness of surface course (mm).

μ—Coefficient of friction resistance between surface course and base course, which can be taken as 1.8.

f_{sy}—Yield strength or standard strength of rebar (MPa).

2 When longitudinal and transverse rebars are placed at the upper part of surface course, they shall be arranged in a single layer. The net cover to longitudinal rebar shall not be less than 50mm in thickness and the transverse rebar shall be placed under the longitudinal rebar.

3 The diameter of longitudinal rebar should be the same as or close to that of transverse rebar, and the difference in their diameters shall not be greater than 4mm. The spacing of longitudinal rebars shall not be greater than 200mm and the spacing of transverse rebars shall not be greater than 800mm. The spacing shall not be less than 100mm or 2.5 times the maximum particle size of aggregate. The distance from edge rebar to longitudinal joint or free edge should be 100 ~ 150mm.

4 The weld length of longitudinal rebar should not be less than 10 times (welding by one side) or 5 times (welding by both sides) the rebar diameter. The welding positions of adjacent rebars shall be staggered. The angle between the connecting line at welding end and the longitudinal rebar shall be smaller than 60°.

15.3.6 The asphalt concrete surface course of composite pavement shall meet the following provisions:

1 Asphalt concrete surface course shall have good performance, including firm adhesion to cement concrete face slab, water seepage prevention, skid- and wear-resistance, anti-crack, anti-rutting and antistripping. Relevant performance requirements shall conform to corresponding provisions of the current *Specifications for Design of Highway Asphalt Pavement* (JTG D50).

2 Asphalt concrete surface course should be arranged in two layers, each with a thickness of 80 ~ 100mm.

3 The type of mixture for asphalt surface course should be the same as that for road sections outside the tunnel. For extra-long tunnels, warm mix asphalt mixture can be used. The

mixing of various additives shall not affect the pavement performance of mixtures.

4 A tack coat shall be placed between asphalt surface course and concrete face slab. The tack coat should consist of modified emulsified asphalt or hot-sprayed SBS modified asphalt + premixed asphalt macadam.

5 At the deformation joints of tunnel structure, the joints and expansion joints of cement concrete surface course not continuously reinforced and without tie bars, and the tunnel sections in soft ground susceptible to late differential settlement, measures retarding the development of reflection cracks such as provision of reinforced geomaterials or stress absorbing layer shall be taken at corresponding position of cement concrete face slab.

15.3.7 When the upper asphalt surface course is laid above the leveling course, the concrete leveling course should not be less than 80mm in thickness, and reinforcement mesh shall be provided. The fiber reinforced concrete leveling course should not be less than 60mm in thickness. The concrete for leveling course shall have the same strength as and be closely bonded to the lower reinforced concrete pavement slab.

15.3.8 If cement concrete pavement is used in a tunnel and asphalt pavement is used outside the tunnel, in-tunnel transition sections consistent with the road sections outside the tunnel shall be provided and conform to the following provisions:

1 For medium, long and extra-long tunnels of expressways and Class-1 highways, the length of in-tunnel entrance transition section shall not be less than the total length of tunnel lighting entrance section and transition section, and not be less than 300m. The length of in-tunnel exit transition section shall not be less than the running length at design speed for 3s.

2 For the short tunnels of expressways and Class-1 highways and the tunnels of Classes II, III and IV highways, the length of in-tunnel entrance/exit pavement transition section shall not be less than the running length at design speed for 3s, and shall not be less than 50m.

15.3.9 The connection of different pavement structures of a tunnel shall meet the following provisions:

1 When it is impossible to set a dowel bar at the expansion joint connecting tunnel and bridge or fixed structure, double-layer reinforcement mesh can be provided in the cement concrete pavement structure 10 ~ 15m away from the joint.

2 When concrete pavement surface course and asphalt pavement surface course join in a tunnel, a transition section not less than 3m in length shall be provided on the side of asphalt pavement surface course. In the transition section, the two types of pavement are arranged in a ladder pattern, with one superimposed onto the other, and the lower thickened cement concrete transition slab shall not be less than 200mm in thickness. Tie bars, each with a diameter of 25mm and a length of 700mm, and spaced at 400mm should be set in the joint of transition slab and concrete surface course.

16 Seismic Design

16.1 Seismic fortification classification and standard

16.1.1 The seismic fortification for mountain road tunnels is divided into three categories, i. e. B, C and D based on highway class, tunnel importance and the difficulty in repair (emergency repair), as listed in Table 16.1.1.

Table 16.1.1 Scope of application for each seismic fortification category of highway tunnel

Seismic fortification category	Scope of application
B	1. Expressway and Class-1 highway tunnels; 2. Three-lane and four-lane tunnels; 3. Twin-arch tunnel; and 4. Underground fan cavern groups under complicate geological conditions
C	1. Single-tube two-lane tunnels of Class II and Class III highways; and 2. Inclined ventilation shaft, vertical shaft and air channel, parallel adit.
D	1. Class-4 highway tunnel; and 2. Auxiliary caverns.

16.1.2 The seismic fortification objectives of tunnels with different seismic fortification categories shall be in accordance with those listed in Table 16.1.2.

Table 16.1.2 Seismic fortification objectives of tunnels with different seismic fortification categories

Seismic fortification category	Fortification objectives	
	E1 earthquake effect	E2 earthquake effect
B	After an earthquake, the lining structure stress is lower than the elastic limit and the structure is in an elastic state; the structure is free from damage and the structural functions remain in the states before the earthquake (performance requirement 1)	After an earthquake, the lining structure stress is beyond the elastic limit, but within the yield strength, and the structure is in a transitional state from elasticity to elasto-plasticity; the structure is subject to local minor damage but can be used continuously without repair or through simple reinforcement (performance requirement 2)

continued

Seismic fortification category	Fortification objectives	
	E1 earthquake effect	E2 earthquake effect
C	After an earthquake, the lining structure stress is lower than the elastic limit and the structure is in an elastic state; the structure is free from damage and the structural functions remain in the states before the earthquake (performance requirement 1)	After an earthquake, the lining structure stress exceeds the yield strength, but does not reach the maximum bearing capacity of structure; the structure is in an elastic-plastic state and does not lose stability; the structure is damaged but shall not partially or wholly collapse and can recover its functions through repair and reinforcement (performance requirement 3)
D	After an earthquake, the lining structure stress is lower than the elastic limit and the structure is in an elastic state; the structure is free from damage and the structural functions remain in the states before the earthquake (performance requirement 1)	—

16.1.3 Categories B and C tunnels should be subject to seismic analysis and checking calculation under E1 and E2 earthquake effects. Category D tunnels may be subject to seismic analysis and checking calculation only under E1 earthquake effect. The tunnels shall be provided with required seismic measures. Categories B, C and D tunnels in regions with the basic seismic peak ground acceleration of 0.05g and 0.10g may be designed with seismic measures only.

16.1.4 The fortification criteria of seismic measures for different tunnels shall meet the following provisions:

1 Seismic measures for Category B tunnels shall be reinforced to make its seismic peak ground acceleration one level higher than that in this region.

2 Seismic measures for and earthquake effects on Category C tunnels shall be determined according to the ground motion parameters in this region.

3 Seismic measures for Category D tunnels are allowed to be reduced appropriately compared to the ground motion parameter requirements in this region, but they shall not be reduced if the seismic peak ground acceleration equals to 0.05g.

16.2 Earthquake effect

16.2.1 For highway tunnels that have not gone through specific seismic safety evaluation for the project sites, the earthquake effect to be considered in seismic design shall be represented by the design basic seismic parameters corresponding to fortification and ground motion in this region and the seismic importance coefficient C_i.

16.2.2 The design basic ground motion parameters in tunnel site area shall be determined in accordance with the ground motion parameters stated in the current *Seismic Ground Motion Parameters Zonation Map of China* (GB 18306). The range of seismic peak ground acceleration of each zone and the seismic intensity corresponding to the seismic peak ground acceleration zonation, shall be in accord with those listed in Table 16.2.2.

Table 16.2.2 Correspondence between seismic peak ground acceleration zonation and range of ground motion acceleration in each zone as well as seismic intensity

Seismic peak ground acceleration A(g)	0.05	0.10	0.15	0.20	0.30	0.40
Seismic fortification ground motion grading (g)	[0.04, 0.09)	[0.09, 0.14)	[0.14, 0.19)	[0.19, 0.28)	[0.28, 0.38)	[0.38, 0.75)
Seismic level of fortification	VI	VII		VIII		IX

16.2.3 The seismic importance coefficient C_i of tunnel shall be in accord with those listed in Table 16.2.3.

Table 16.2.3 Seismic importance coefficient C_i of tunnel

Seismic fortification category	E1 earthquake effect	E2 earthquake effect
B	0.43	1.3
C	0.34	1.0
D	0.26	—

16.2.4 For an extra-long tunnel located in a zone with seismic peak ground acceleration of 0.40g, specific seismic safety evaluation for the project sites shall be made in accordance with relevant provisions to determine the earthquake effect. For tunnels that have gone through seismic safety evaluation for the project sites, the seismic peak ground acceleration under different earthquake effect shall not be lower than that determined based on the seismic importance coefficient of Table 16.2.3 herein.

16.3 Seismic design check

16.3.1 The checking calculation of structural strength, deformation and tunnel portal stability shall be conducted in view of the seismic fortification objectives. Earthquake effect shall be combined with permanent and variable loads.

16.3.2 For strength checking calculation under E1 earthquake effect, the safety factors of structural strength shall be in accordance with those listed in Table 16.3.2.

Table 16.3.2 Safety factor of structural strength

Load-bearing characteristic	Type of material	
	Reinforced concrete	Concrete
Concrete reaches its ultimate compressive strength.	—	1.8
Concrete reaches its ultimate tensile strength.	—	2.5
Rebar reaches its design strength or concrete reaches its ultimate compressive strength.	1.5	—
Concrete reaches its ultimate tensile strength (major tensile stress).	1.8	—

16.3.3 When checking the deformation performance as an integral structure as a whole, the maximum convergence for secondary lining structure shall be used as the checking calculation criterion. *When the required seismic resistance is 2h* (check the original text!), the maximum convergence threshold shall not be greater than 5.0‰ of tunnel span. When the required seismic resistance is 3h (checking the original text!), the maximum convergence threshold shall not be greater than 15.0‰ of tunnel span.

16.3.4 During seismic design check, the transverse seismic resistance calculation should be conducted using static method for a drill-and-blast mountain road tunnel. For longitudinal or 3D calculation, if necessary, the calculation methods shall be used as those listed in Table 16.3.4.

Table 16.3.4 Seismic calculation methods for tunnel

Seismic calculation	Calculation method	Structural performance state
Transverse seismic calculation	Static method	Elasticity
	Transverse response displacement method	Elasticity
	Dynamic analysis method (time history) (2D or 3D)	Elasticity or elastoplasticity
Longitudinal seismic calculation	Longitudinal response displacement method	Elasticity
	Dynamic analysis method (time history)	Elasticity or elastoplasticity
Seismic calculation with 3D space model	Dynamic analysis method (time history)	Elasticity or elastoplasticity

16.4 Seismic resistance measures

16.4.1 The tunnel should be arranged at sites favorable for seismic resistance, and should not be arranged at site with unfavorable geologies such as rock piles, landslide mass, debris flow gully, collapsible rocks and surrounding falling rocks, and at low-lying place with difficulty in drainage or under the unstable cliff.

16.4.2 The portal location shall be determined in combination with the topographic and geological conditions. Measures shall be taken to control the excavation heights of heading slope and side slopes at a portal, so as to prevent seismic hazards such as collapse and landslide.

16.4.3 In areas with high seismic level of fortification, if the side and heading slopes at a portal are steep, cut-and-cover tunnel portal should be used, and measures shall be taken to prevent impact of falling rocks.

16.4.4 Cut-and-cover tunnel lining shall be constructed of reinforced concrete structure and provided with seismic joints longitudinally along the tunnel.

16.4.5 In areas with design basic seismic peak ground acceleration greater than or equal to 0.20g, seismic connection measures like short rebars or tendons shall be added or arranged at construction joint between end wall at tunnel portal and lining ring frame, as well as between end wall and opening retaining wall or wing wall.

16.4.6 For structures in such sections as the portal section, shallowly-embedded section under eccentric load, deeply-embedded weak surrounding rock section and fracture zone of a fault, the seismic fortification length shall be determined based on topographic and geological conditions. Both ends of the seismic fortification section shall extend to sections with good surrounding rock quality. They should extend 5 ~ 10m in tunnels with no more than two lanes and extend 10 ~ 20m in tunnels with no less than three lanes.

16.4.7 The lining structures in seismic fortification sections shall conform to the following provisions:

1 Tunnel lining in weak surrounding rock sections shall be curved wall lining with arch invert.

2 At the boundary between cut-and-cover tunnel and bored tunnel, the boundary between soft

rock and hard rock and the fault fracture zone, seismic joints should be arranged through overall consideration of settlement joints and expansion joints.

3 When composite support structures are used at the tunnel intersections and in the fault fracture zones not strengthened by grouting, the secondary lining structures shall be constructed of reinforced concrete.

4 When a tunnel passes through active faults, the transverse section of the lining should be designed with over-excavation according to the assessed maximum dislocation of the fault. Fault fortification section should be provided with seismic joints.

16.5 In-tunnel facilities

16.5.1 Seismic design shall be carried out for in-tunnel facilities, including in-tunnel auxiliary structures and auxiliary electrical and mechanical equipment, and their connection with main structures.

16.5.2 Auxiliary structures and electrical and mechanical equipment shall be securely connected to main structures, so as to prevent them from falling during an earthquake.

16.5.3 The supports and connections for electrical and mechanical facilities installed in the tunnel shall meet the functional requirements during an earthquake and shall not lead to damage of related parts.

17 Design for Alteration and Expansion

17.1 General

17.1.1 If an existing tunnel or existing tunnel alignment is utilized, the existing tunnel may be reconstructed or expanded or additional tunnel be constructed to improve the tunnel traffic conditions, increase the traffic capacity or raise the highway class and standards. For local sections restricted by topographic and geological conditions, the original tunnel may be retained for traffic after evaluation and verification.

17.1.2 For tunnel reconstruction and expansion, the necessary investigations specified in Chapter 3 herein shall be conducted. In addition, the design, construction, repair & maintenance and operation of the existing tunnel shall be investigated.

17.2 Tunnel reconstruction and expansion scheme design

17.2.1 For tunnel reconstruction and expansion design, economical and technical comparisons shall be made in combination with the overall route design, tunnel connection conditions, engineering geology, current condition of the existing tunnel, traffic arrangement and construction conditions. The existing tunnel shall be fully utilized to reasonably determine the forms and technical standards of reconstruction and expansion.

17.2.2 The alignment and cross-section design of additional and expanded tunnels shall conform to relevant provisions in the current *Technical Standard of Highway Engineering* (JTG B01) and in Chapter 4 herein.

17.2.3 For expansion of the existing twin tunnels with four-lanes into a six-lane tunnels, expansion should be carried out in the original locations.

17.2.4 For expansion of the existing twin tunnels with four-lanes into an eight-lane tunnel, expansion may be carried out in the original location or by utilizing the original tunnel and building an additional tunnel.

17.2.5 For expansion of the existing twin arch tunnels with four-lanes into a twin tunnels with six-lanes, the existing twin-arch tunnels should be retained and traffic in the same direction will be separated by building a new single three lane tunnel.

17.2.6 For expansion of the existing twin arch tunnels with four lanes into a twin tunnels with eight lanes, the existing twin-arch tunnels can be retained, and a new four-lane tunnel or twin two-lane tunnels may be constructed; alternatively, the twin-arch tunnels can be changed into a single four-lane tunnel, and a new single four-lane tunnel may be constructed.

17.2.7 For separation of traffic in the same direction with two tunnels, necessary traffic safety facilities shall be provided.

17.2.8 For expansion of a single two-way two-lane tunnel into a twin four-lane tunnel, the existing tunnel should be utilized and an additional two-lane tunnel may be constructed.

17.2.9 The existing tunnel that will no longer be open to traffic after reconstruction and expansion should serve as a repair or maintenance service channel and emergency evacuation and rescue channel. The evacuation and rescue channel shall be capable of ensuring the long-term stability of tunnel structures.

17.2.10 Tunnel expansion should be carried out after construction of new parallel tunnels is completed. Construction of new tunnels shall minimize the influence on existing tunnel structures. If necessary, the existing tunnel structures shall be temporarily protected or reinforced.

17.2.11 The tunnel reconstruction and expansion design shall contain design for construction schemes and traffic arrangement design, and a construction scheme without traffic interruption should be adopted.

17.3 Tunnel expansion

17.3.1 Expansion of existing tunnels shall comply with the following requirements:

1 The tunnel alignment shall be consistent with that of the existing tunnel, and the existing tunnel clearance shall be used wherever possible as the expanded tunnel clearance.

2 The design elevation of tunnel pavement should be consistent with that of existing tunnel.

3 The locations, distances and sizes of emergency stop zone, vehicle cross passage and pedestrian cross passage in the existing tunnel shall be utilized wherever possible.

17.3.2 Tunnel structure expansion design shall include design of demolition scheme and temporary support of existing tunnel structures. For tunnel expansion, the structural stress state and surrounding rock stability shall be calculated under multiple concurrent construction works of the expansion, and the lining structure and construction shall comply with relevant provisions in other chapters herein.

17.3.3 The rock load in tunnel expansion shall be calculated as follows:

1 If construction of existing tunnel causes little disturbance to surrounding rock, no large deformation or collapse occurs during construction, and there are no sections of the tunnels with obvious variation of the condition of the surrounding rock after a certain period of operation, the rock load can be calculated in reference to newly-built tunnels.

2 For sections which once collapsed during construction of the existing tunnel, the surrounding rock class and pressure shall be determined based on the height and the transverse extent of collapse.

17.3.4 Expansion of an existing tunnel should be done by expanding excavation on a single side within a range covering the full excavation range of the existing tunnel.

17.3.5 If tunnel expansion causes obvious vibration of adjacent tunnels due to blast and has adverse impacts on surrounding rock stability and structural internal force, reliable technical measures shall be taken to reduce the impacts.

17.4 Tunnel reconstruction

17.4.1 Prior to reconstruction, the current condition of the existing tunnel shall be investigated and tested comprehensively to analyze and evaluate the health status of the existing tunnel.

17.4.2 Tunnel reconstruction shall conform to the following provisions:

1 Tunnel reconstruction shall comply with the current *Technical Standard of Highway Engineering* (JTG B01). If it is restricted by technical and economical conditions, it

shall comply with the original technical standards.

2 The civil structures of the existing tunnel shall be used as much as possible provided that the traffic capacity and operation safety can be ensured. The drainage on the back of existing structures and lining should not be changed.

3 After reconstruction, the difference between the design traffic speed in the reconstructed tunnel and the design traffic speeds in the sections before and after the reconstructed tunnel shall not be greater than 20km/h.

17.4.3 If a new emergency stop zone is to be added to the existing tunnel, the method of local expanded excavation shall be adopted.

17.4.4 If an additional parallel tunnel is to be constructed during tunnel reconstruction, the tunnel reconstruction should be conducted after completion of additional tunnel.

17.5 Additional tunnel

17.5.1 The structural design of the additional tunnel shall conform to relevant provisions on construction of new tunnels.

17.5.2 The additional tunnel shall be aligned so as to reduce the impacts on the existing tunnel.

17.5.3 If both the additional tunnel and the existing tunnel are tunnels with narrow clear space, the rock load shall be considered based on twin tunnels with narrow pillar.

17.5.4 If a cross passage is to be provided between an additional tunnel and the existing tunnel, the design elevation of the additional tunnel shall meet the design requirements for longitudinal slope of cross passage. The cross passage opening location shall be kept at least 2m away from the movement or construction joints of the existing tunnel lining structures.

17.5.5 According to such factors as the operation requirements of the existing tunnel, the structure status, the clear distance to the existing tunnel and the surrounding rock class, consideration shall be given to the impact of blasting construction for additional tunnel on the adjacent existing tunnel structures.

18 In-tunnel Reserved Caverns, Embedment and Other

18.1 General

18.1.1 The in-tunnel advance provision, embedment and other structures shall be designed in accordance with the requirements of tunnel traffic engineering and facilities and coordinated with relevant disciplines.

18.1.2 Reserved caverns and embedded parts shall be capable of ensuring the stability and structural strength of tunnel structures and shall not undermine the support capability of tunnel lining structures.

18.2 Reserved caverns and embedment

18.2.1 The proposed size of reserved caverns shall meet the requirements of equipment placement space and maintenance & operation space.

18.2.2 The reserved caverns shall be arranged at least 1.5m away from movement or construction joints of lining structures.

18.2.3 Reserved caverns shall be designed with proper waterproof measures and free from seepage or leakage.

18.2.4 As to embedded parts for suspension and installation of facilities in tunnels, their strength and corrosion-resistant design shall be conducted based on their load bearing and durability requirements, and they shall meet the following requirements.

1 The design service life of embedded parts in tunnel shall be the same as that of structures.

2 Embedded parts required to bear loads shall meet the bearing capacity requirements.

3 The strength of embedded parts for hanging fans shall be capable of bearing loads not less than 15 times the fan mass.

18.2.5 Pipelines embedded in lining shall be placed in the middle of lining section, and the pipe wall shall not be less than 100mm from the inner and outer edges of lining.

18.2.6 In tunnels where high tension cable installations are placed, the earthing flat steel shall be embedded according to requirements of electrical design requirements, and the specific embedment earthing requirements can be designed by the designers of electrical discipline.

18.3 Cable trenches

18.3.1 Cable trenches shall be arranged according to the needs of electrical and mechanical works and firefighting works, and should be arranged below the maintenance walkway on both sides.

18.3.2 As to the size of a cable trench, its section form and size shall be proposed based on the cable and firefighting service pipe arrangement requirements in the tunnel, and shall facilitate the laying and maintenance of cables and fire service pipes.

18.3.3 The cable trench covers shall be capable of bearing pedestrian loads. The covers shall be provided with lifting hooks, holes or slots at a certain distance to open them. The tops of covers shall be flush with the curb tops.

18.3.4 The exterior wall of a cable trench shall be of reinforced concrete structure and shall ensure no damage to facilities in the trench after it is hit by a vehicle.

18.3.5 Cable trenches shall meet the gravity flow drainage requirements.

Appendix A
Provisions on Rock Mass Classification

A.0.1 Quantitative index of rock hardness is expressed by saturated uniaxial compressive strength of rock (R_c); measured value should be used. If measured value is not available, the converted value from measured point load strength index of rock ($I_{S(50)}$) may be used, i.e. calculated from Eq. (A.0.1):

$$R_c = 22.82 I_{S(50)}^{0.75} \tag{A.0.1}$$

A.0.2 Quantitative index of rock mass integrity is expressed by rock mass integrity index (K_v), as specified below:

1 K_v should be measured value of elastic wave; if no measured value is available, it may be determined from volumetric joint count of rock (J_v) per Table A.0.2.

Table A.0.2 Correspondence between J_v and K_v

J_v (joints/m^3)	<3	3 ~ 10	10 ~ 20	20 ~ 35	≥35
K_v	>0.75	0.75 ~ 0.55	0.55 ~ 0.35	0.35 ~ 0.15	≤0.15

2 To test and calculate rock mass integrity index K_v, points and zones representative of different engineering geological rock formations or lithology shall be selected to test the velocity of elastic longitudinal wave in rock mass. Samples shall be taken from the same rock mass to test longitudinal wave velocity in rock. This index shall be calculated from Eq. (A.0.2-1):

$$K_v = (v_{pm}/v_{pr})^2 \tag{A.0.2-1}$$

where

v_{pm}—elastic longitudinal wave velocity in rock mass (km/s);

v_{pr}—elastic longitudinal wave velocity in rock (km/s).

3 To test and calculate volumetric joint count of rock J_v (joints/m^3), outcrops or excavated wall surface representative of different engineering geological rock formations or lithology shall be selected to count joints (discontinuities). In addition to joint sets, scattered joints longer than 1m shall also be counted. Re-cemented joints with siliceous, ferruginous or calcareous infills may be excluded from the count. An area of not less than $2 \times 5\text{m}^2$ shall be counted for each measurement point. The value of J_v shall be calculated from Eq. (A. 0.2-2) based on joint count result:

$$J_v = S_1 + S_2 + \cdots + S_n + S_k \qquad \text{(A. 0. 2-2)}$$

where

S_n—The number of joints per meter of survey line in the nth joint set;

S_k—The number of joints not in sets per cubic meter of rock (joints/m^3).

A. 0. 3 Values of correction coefficient for Rock Quality Designation influence factor K_1, K_2 and K_3 may be taken as specified in Tables A. 0. 3-1, A. 0. 3-2 and A. 0. 3-3. The correction coefficient may be taken as zero if not specified in these tables.

Table A. 0. 3-1 Correction coefficient K_1 for groundwater influence

Groundwater ingress state	*BQ*			
	>550	550 ~ 451	350 ~ 251	<250
Damp or dripping, $p \leqslant 0.1$ or $Q \leqslant 25$	0	0	0.2 ~ 0.3	0.4 ~ 0.6
Rainy or surge flow, $0.1 < p \leqslant 0.5$ or $25 < Q \leqslant 125$	0 ~ 0.1	0.1 ~ 0.2	0.4 ~ 0.6	0.7 ~ 0.9
Rainy or surge flow, $p > 0.5$ or $Q > 125$	0.1 ~ 0.2	0.2 ~ 0.3	0.7 ~ 0.9	1.0

Note: Under the same groundwater state, the smaller the Rock Quality Designation BQ, the greater the value of correction coefficient K_1; in the same rock mass, the higher the groundwater inflow and pressure, the greater the value of correction coefficient K_1.

Table A. 0. 3-2 Correction coefficient for influence of main weak structure plane occurrence K_2

Structure plane orientation and its combination with tunnel axis	Angle between structure plane strike and tunnel axis <30°; structural plane dip angle at 30° ~ 75°	Angle between structure plane strike and tunnel axis >60°; structural plane dip angle >75°	Other combinations
K_2	0.4 ~ 0.6	0 ~ 0.2	0.2 ~ 0.4

Notes:

1. In general, the larger the angle between structure plane strike and tunnel axis and the structural plane dip angle, the smaller the correction coefficient K_2; the smaller the angle between structure plane strike and tunnel axis and the structural plane dip angle, the greater the correction coefficient K_2.
2. This table is specifically for the situation where one set of prevailing structure plane exists. It is not applicable to the situation where two or more sets of prevailing structure plane exist.

Table A.0.3-3 Correction coefficient K_3 for influence of initial stress

Initial stress	*BQ*				
	>550	550 ~ 451	450 ~ 351	350 ~ 251	≤250
High stress zone	1.0	1.0	1.0 ~ 1.5	1.0 ~ 1.5	1.0
High stress zone	0.5	0.5	0.5	0.5 ~ 1.0	0.5 ~ 1.0

Notes:

1. The smaller the value of *BQ*, the greater the correction coefficient K_3.
2. The high stress and high stress states may be evaluated in accordance with Table A.0.4 in Appendix A.

A.0.4 Dependent on main phenomena encountered in drilling and excavation of rock mass (surrounding rock) such as rock core disking or rock burst, the stress in surrounding rock may be evaluated in detail according to Table A.0.4.

Table A.0.4 Main phenomena encountered during excavation in high initial stress zones

Stress state	Main phenomena	R_c/σ_{max}
High stress	1. Hard rock: during excavation there is rock burst, popping, spalling from tunnel wall and many new fractures; poor tunnel formation 2. Soft rock: rock core commonly caked; spalling from tunnel wall, obvious displacement, even large displacement, for a long period of time during excavation; no easy to form tunnel	<4
High stress	1. Hard rock: possible rock burst, spalling and falling off from tunnel wall and many new fractures during excavation; poor tunnel formation 2. Soft rock: rock core occasionally caked; obvious displacement of rock at tunnel wall for a long period of time during excavation; poor tunnel formation	4 ~ 7

Note: σ_{max} is Maximum initial stress perpendicular to tunnel axis.

A.0.5 Qualitative classification of rock hardness shall be as specified below:

1 Rock hardness may be qualitatively classified as shown in Table A.0.5-1.

Table A.0.5-1 Qualitative classification of rock hardness

Description		Qualitative evaluation	Typical rock
Hard rock	Stiff rock mass	Producing clear sound when hammered, rebound, hand shake, difficult to break; mostly no absorption reaction when immersed in water	Unweathered-slightly weathered granite, syenite, diorite, diabase, basalt, andesite, gneiss, quartz schist, siliceous slate, quartzite, siliceous cemented conglomerate, quartz sandstone, cherty limestone, etc.
	Medium hard rock	Producing relatively clear sound when hammered, slight rebound, slight hand shake, relatively difficult to break; slight water absorption reaction when immersed in water	1 Moderately (weakly) weathered stiff rock; 2 Unweathered-slightly weathered fused tuff, marble, slate, dolomite, limestone, calcareous cemented sand-shale, etc.

continued

Description		Qualitative evaluation	Typical rock
Weak rock	Medium soft rock	Producing a toneless sound when hammered, no rebound, relatively easy to break; after immersed in water, fingernail can leave a mark	1 Highly weathered stiff rock; 2 Moderately (weakly) weathered relatively stiff rock; 3 Unweathered-slightly weathered tuff, phyllite, sandy mudstone, marlstone, argillaceous sandstone, siltstone, shale, etc.
	Soft rock	Dumb when hammered, no rebound, with indenture, easy to break; after immersed in water, it can be broken by hand.	1 Highly weathered stiff rock; 2 Moderately (weakly)-highly weathered relatively stiff rock; 3 Moderately (weakly) weathered medium soft rock; 4 Unweathered mudstone, argillaceous shale, chlorite schist, Pinal schist, etc.
	Extremely soft rock	Dumb when hammered, no rebound, with deep indenture, can be broken by hand; after immersed in water, it can be kneaded into a ball.	1 Various completely weathered rocks; 2 Highly weathered soft rock; 3 Various hypabyssal rocks

2 Rock weathering degree may be determined per Table A.0.5-2. When the wave velocity ratio k_v, weathering coefficient k_f and field characteristics do not correspond to those specified in the table, the rock weathering degree should be judged comprehensively.

Table A.0.5-2 Classification of rock weathering degree

Description	Field characteristics	Weathering degree indicators	
		Wave velocity ratio k_v	Weathering coefficient k_f
Unweathered	Rock structure unchanged, fresh	0.9 ~ 1.0	0.9 ~ 1.0
Slightly weathered	Rock structure, mineral composition and luster basically unchanged; some fissure planes with ferrimanganic rendering or slightly discolored	0.8 ~ 0.9	0.8 ~ 0.9
Moderately (weakly) weathered	Rock structure largely broken; mineral composition and luster considerably changed; feldspar, mica and sideromelane weathered and altered	0.6 ~ 0.8	0.4 ~ 0.8
Highly weathered	Rock structure largely broken; mineral composition and luster considerably changed; feldspar, mica and sideromelane weathered and altered	0.4 ~ 0.6	<0.4
Completely weathered	Rock structure completely broken; decomposed and disintegrated into loose soil or sand like material; all minerals discolored, luster lost; minerals other than quartz granules largely weathered into secondary minerals.	0.2 ~ 0.4	—

Notes:

1. The wave velocity ratio k_v is the ratio of elastic longitudinal wave velocity in weathered rock to that in fresh rock.

2. The weathering coefficient k_f is the ratio of saturated uniaxial compressive strength of weathered rock.

3 The degree of development of rock joints may be classified as presented in Table A.0.5-3.

Table A.0.5-3 Classification of degree of development of rock joints

Joint spacing d (mm)	$d>400$	$200<d\leqslant 400$	$20<d\leqslant 200$	$d\leqslant 20$
Degree of development of joints	Undeveloped	Developed	Highly developed	Extremely developed

4 The relationship between R_c and qualitative classification of rock hardness may be determined as shown in Table A.0.5-4.

Table A.0.5-4 Relationship between R_c and qualitative classification of rock hardness

R_c (MPa)	>60	60 ~ 30	30 ~ 15	15 ~ 5	<5
Hardness	Stiff rock	Relatively stiff rock	Medium soft rock	Soft rock	Extremely soft rock

A.0.6 Qualitative classification of rock mass integrity shall be as specified below:

1 Rock mass integrity may be qualitatively classified as shown in Table A.0.6-1.

Table A.0.6-1Qualitative classification of rock mass integrity

Description	Development degree of structure plane		Bond degree of main structure planes	Type of main structure planes	Corresponding structural type
	Number of sets	Average spacing (m)			
Intact	1 ~ 2	>1.0	Good or fair	Joints, fissures and bedding	Monolithic or extremely massive structure
Relatively intact	1 ~ 2	>1.0	Poor	Joints, fissures and bedding	Blocky or massive structure
	2 ~ 3	1.0 ~ 0.4	Good or fair		Blocky structure
Relatively fractured	2 ~ 3	1.0 ~ 0.4	Poor	Joints, fissures, bedding and minor fault	Fractured blocks or medium thick layers
		0.2 ~ 0.4			
	≥3		Good		Interlocked clastic structure
			Fair		Medium and thin layers
Fractured	≥3	0.2 ~ 0.4	Poor	Various types of structure plane	Fractured blocks
		≤0.2	Fair or poor		Clastic structure
Extremely fractured	Disorderly		Very poor		Loose structure

Note: Average spacing refers to the average spacing of main structure planes (1-2 sets).

2 The bond degree of structure planes may be classified as shown in Table A.0.6-2.

Table A.0.6-2 Classification of bond degree of structure planes

Bond degree	Structure plane features
Good	Aperture < 1mm, siliceous, ferruginous or calcareous cementation, or rough with no infill; Aperture at 1 ~ 3mm, siliceous or ferruginous cementation; Aperture > 3mm, rough, siliceous cementation
Fair	Aperture < 1mm, structure plane straight and flat, calcareous/argillaceous cementation or with no infill; Aperture at 1 ~ 3mm, calcareous cementation; Aperture > 3mm, rough, siliceous or ferruginous cementation
Poor	Aperture at 1 ~ 3mm, structure plane straight and flat, argillaceous or calcareous/argillaceous cementation; Aperture > 3mm, mostly argillaceous or debris cementation
Very poor	Argillaceous infill or mud and debris infill with a thickness greater than undulation difference

3 Rock formation thickness may be classified as shown in Table A.0.6-3.

Table A.0.6-3 Classification of rock formation thickness

Single layer thickness h (m)	$h > 1.0$	$0.5 < h \leq 1.0$	$0.1 < h \leq 0.5$	$h \leq 0.1$
Classification of rock formation thickness	Extremely thick	Thick	Medium thick	Thin

4 The correspondence between K_v and qualitative rock mass integrity may be determined per Table A.0.6-4.

Table A.0.6-4 Correspondence between K_v and qualitative rock mass integrity

K_v	>0.75	0.75 ~ 0.55	0.55 ~ 0.35	0.35 ~ 0.15	<0.15
Integrity	Intact	Relatively intact	Relatively fractured	Fractured	Extremely fractured

A.0.7 Physical and mechanical parameters and shear strength of structural plane of various classes of surrounding rock shall be obtained from lab or field tests. If measurement data are not available, they may be obtained from the following tables:

1 Physical and mechanical parameters of surrounding rock may be obtained from Table A.0.7-1.

Table A.0.7-1 Physical and mechanical parameters of surrounding rock

Surrounding rock class	Unit weight γ (kN/m^3)	Coefficient of elastic resistance k (MPa/m)	Modulus of deformation E (GPa)	Poisson's ratio μ	Internal friction angle Φ(°)	Cohesion c (MPa)	Calculated friction angle φ_c(°)
Ⅰ	>26.5	1800 ~ 2800	>33	<0.2	>60	>2.1	>78
Ⅱ		1200 ~ 1800	20 ~ 33	0.2 ~ 0.25	50 ~ 60	1.5 ~ 2.1	70 ~ 78
Ⅲ	26.5 ~ 24.5	500 ~ 1200	6 ~ 20	0.25 ~ 0.3	39 ~ 50	0.7 ~ 1.5	60 ~ 70

continued

Surrounding rock class	Unit weight γ (kN/m^3)	Coefficient of elastic resistance k (MPa/m)	Modulus of deformation E (GPa)	Poisson's ratio μ	Internal friction angle Φ(°)	Cohesion c (MPa)	Calculated friction angle φ_c(°)
Ⅳ	24.5 ~ 22.5	200 ~ 500	1.3 ~ 6	0.3 ~ 0.35	27 ~ 39	0.2 ~ 0.7	50 ~ 60
Ⅴ	17 ~ 22.5	100 ~ 200	< 1.3	0.35 ~ 0.45	20 ~ 27	0.05 ~ 0.2	40 ~ 50
Ⅵ	15 ~ 17	< 100	< 1	0.4 ~ 0.5	< 20	< 0.2	30 ~ 40

Notes:

1. Values in this table do not include loess strata.
2. If calculated friction angle is used, then internal friction angle and cohesion will not be considered.

2 The shear peak strength parameters of rock mass discontinuity may be selected from Table A.0.7-2.

Table A.0.7-2 Shear peak strength parameters of rock mass discontinuity

S. N.	Hardness of rock mass on both sides and bond degree of discontinuity	Internal friction angle Φ(°)	Cohesion c (MPa)
1	Hard rock, with good bond;	> 37	> 0.22
2	Hard-medium hard rock, with fair bond; Medium soft rock, with good bond	37 ~ 29	0.22 ~ 0.12
3	Hard-medium hard rock, with poor bond; Medium soft-soft rock, with fair bond	29 ~ 19	0.12 ~ 0.08
4	Medium hard-medium soft rock, with poor-very poor bond; Soft rock with poor bond; argillaceous surface of soft rock	19 ~ 13	0.08 ~ 0.05
5	Medium hard rock and all soft rocks, with poor bond; Argillaceous layer of soft rock	< 13	< 0.05

3 Physical and mechanical parameters of surrounding soils may be obtained from Table A.0.7-3.

Table A.0.7-3 Physical and mechanical parameters of surrounding soils

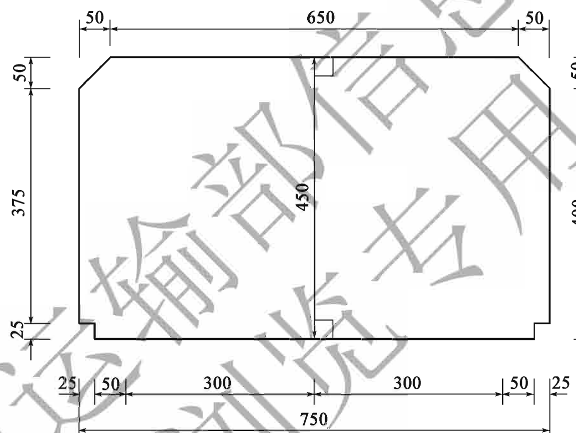
Surrounding rock class	Soil type	Unit weight (kN/m^3)	Coefficient of elastic resistance k (MPa/m)	Modulus of deformation E (GPa)	Poisson's ratio μ	Internal friction angle Φ (°)	Cohesion c (MPa)
Ⅳ	Clayey material	20 ~ 30	200 ~ 300	0.030 ~ 0.045	0.25 ~ 0.33	30 ~ 45	0.060 ~ 0.250
	Sandy soil	18 ~ 19		0.024 ~ 0.030	0.29 ~ 0.31	33 ~ 40	0.012 ~ 0.024
	Gravelly soil	22 ~ 24		0.050 ~ 0.075	0.15 ~ 0.30	43 ~ 50	0.019 ~ 0.030

continued

Surrounding rock class	Soil type	Unit weight (kN/m^3)	Coefficient of elastic resistance k (MPa/m)	Modulus of deformation E (GPa)	Poisson's ratio μ	Internal friction angle Φ (°)	Cohesion c (MPa)
V	Clayey material	16 ~ 18	100 ~ 200	0.005 ~ 0.030	0.33 ~ 0.43	15 ~ 30	0.015 ~ 0.060
	Sandy soil	15 ~ 18		0.003 ~ 0.024	0.31 ~ 0.36	25 ~ 33	0.003 ~ 0.012
	Gravelly soil	17 ~ 22		0.010 ~ 0.050	0.20 ~ 0.35	30 ~ 43	< 0.019
VI	Clayey material	14 ~ 16	< 100	< 0.005	0.43 ~ 0.50	< 15	< 0.015
	Sandy soil	14 ~ 15		0.003 ~ 0.005	0.36 ~ 0.42	10 ~ 25	< 0.003

Appendix B
Structure Gauge and Intrados Drawings

B.0.1 The construction gauge and intrados of a two-lane Class-4 highway tunnel are illustrated in Fig. B.0.1-1 and B.0.1-2.



B.0.1-1 Construction gauge of two-lane Class-4 highway tunnel (20km/h) (dimensions given in cm)

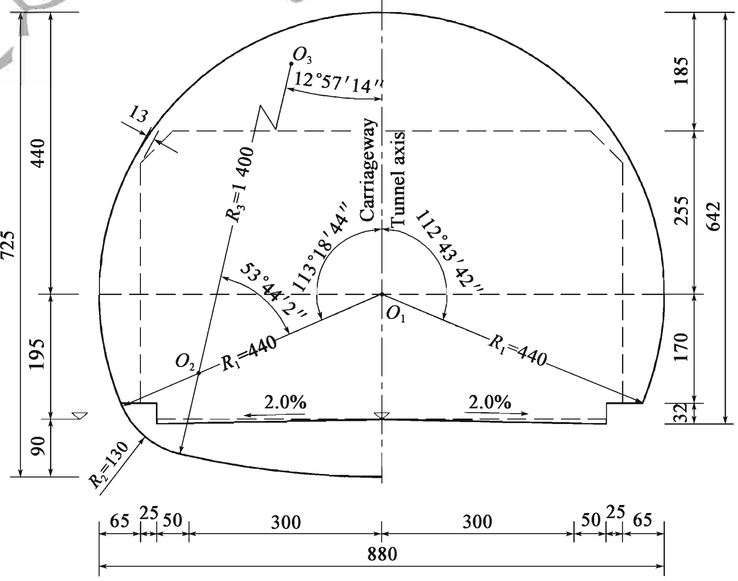


Fig. B.0.1-2 Intrados of two-lane Class-4 highway tunnel (20km/h) (dimensions given in cm)

B.0.2 The construction gauge and intrados of a two-lane Class III highway tunnel are illustrated in Fig. B.0.2-1 and B.0.2-2.

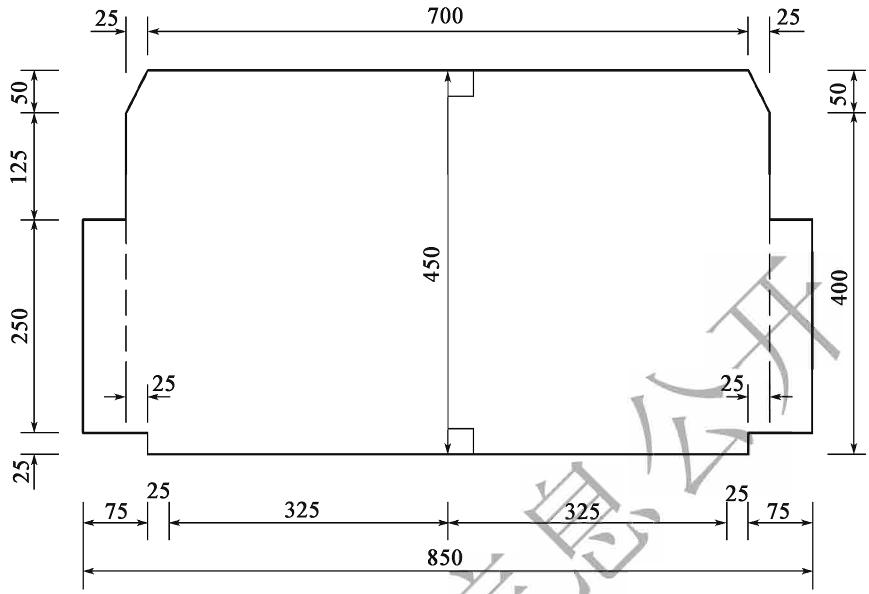


Fig. B.0.2-1 Construction gauge of two-lane Class III highway tunnel (30km/h) (dimensions given in cm)

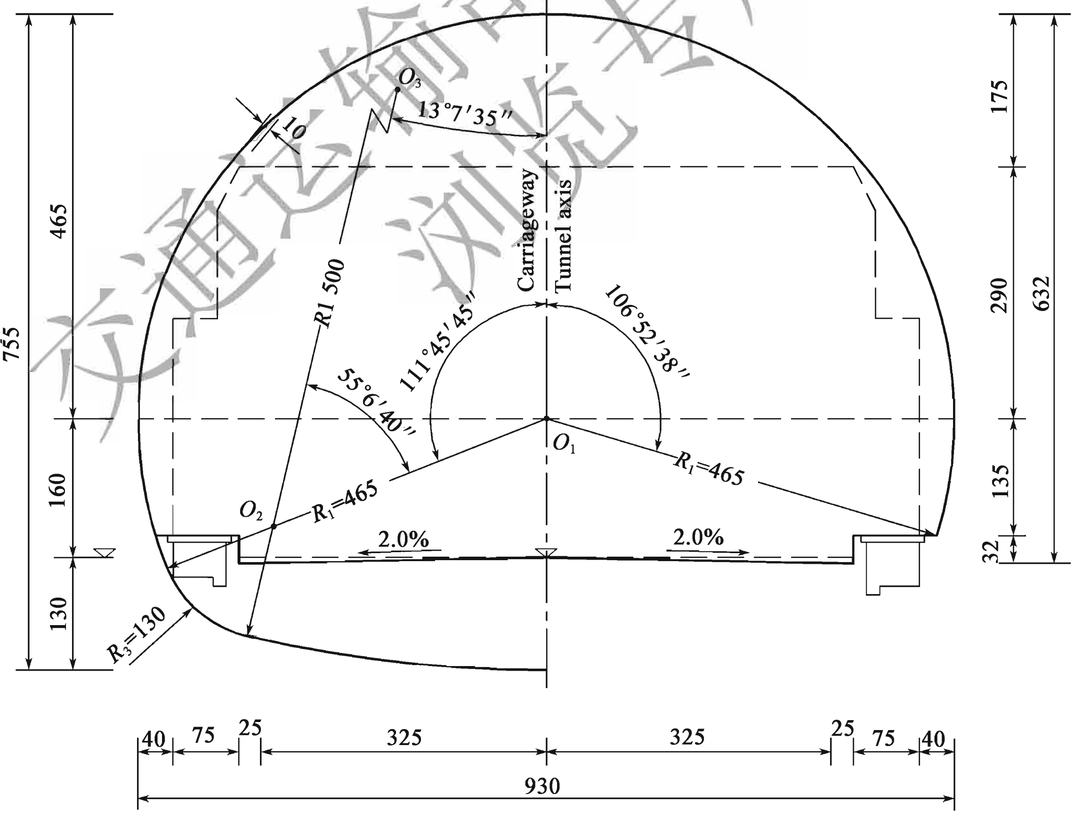


Fig. B.0.2-2 Intrados of two-lane Class III highway tunnel (30km/h) (dimensions given in cm)

B. 0. 3 The construction gauge and intrados of a two-lane Class III highway tunnel are illustrated in Fig. B. 0. 3-1 and B. 0. 3-2.

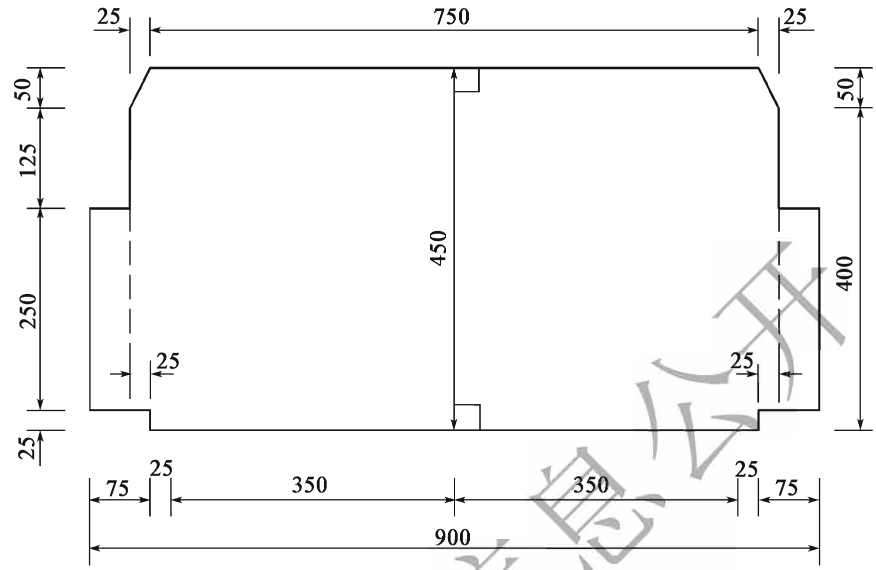


Fig. B. 0. 3-1 Construction gauge of two-lane Class III highway tunnel (40km/h) (dimensions given in cm)

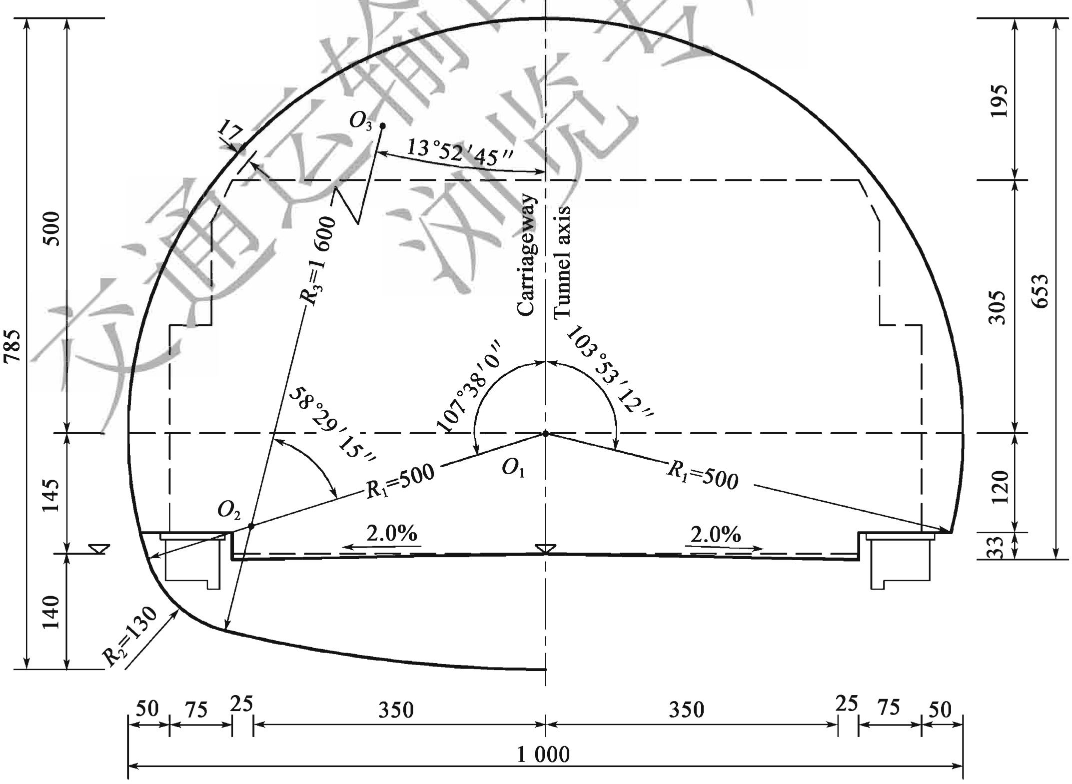


Fig. B. 0. 3-2 Construction gauge of two-lane Class III highway tunnel (40km/h) (dimensions given in cm)

B.0.4 The clearance of a two-lane Class-2 highway tunnel (60km/h) is illustrated in Fig. B.0.4-1 and B.0.4-2.

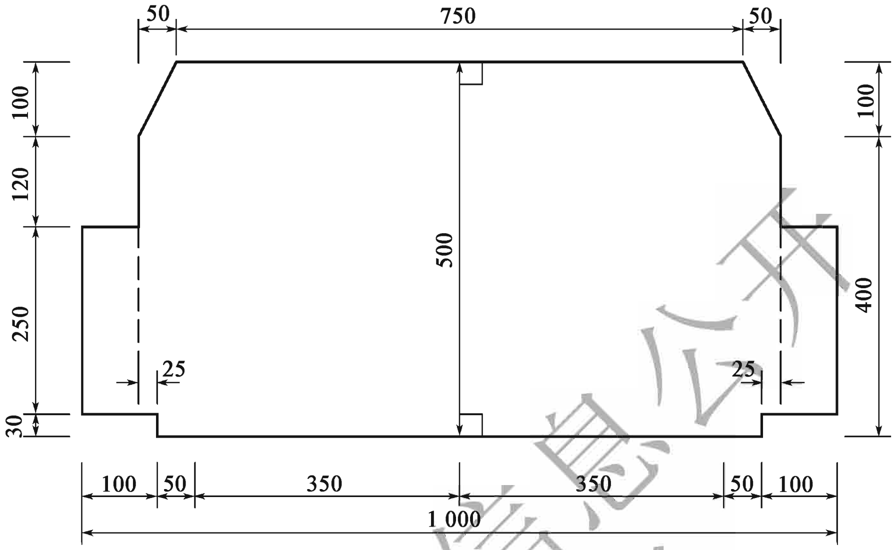


Fig. B.0.4-1 Clearance of two-lane Class-2 highway tunnel (60km/h) (dimensions given in cm)

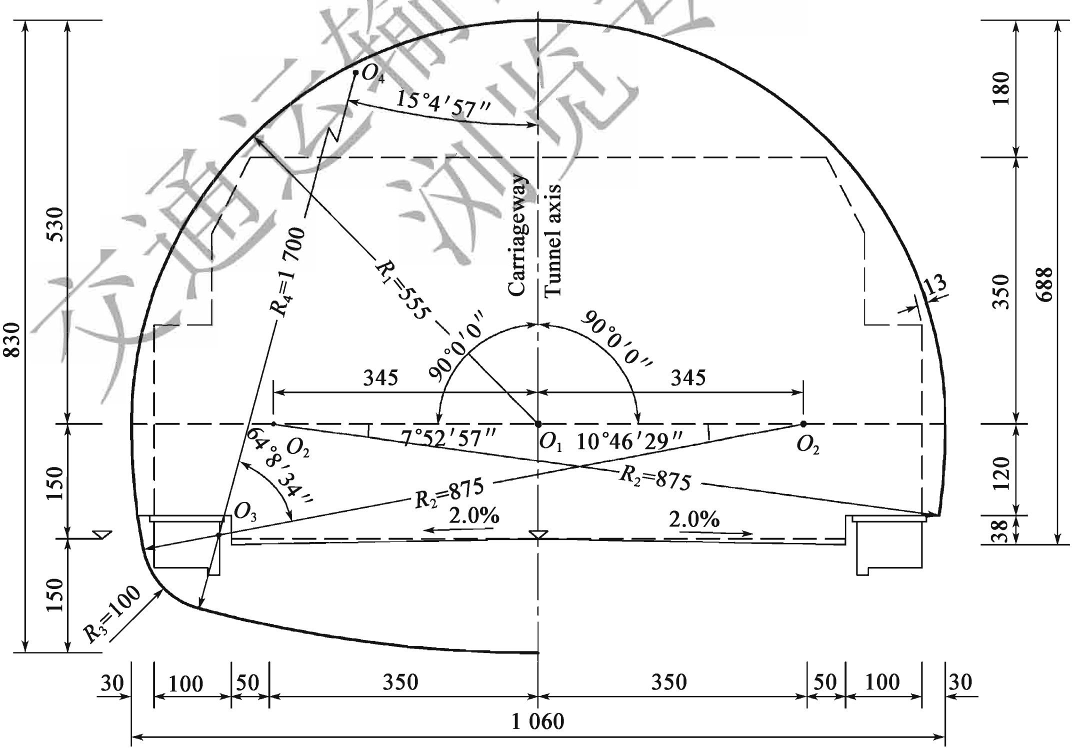


Fig. B.0.4-2 Intrados of two-lane Class-2 highway tunnel (60km/h) (dimensions given in cm)

B.0.5 The clearance of a two-lane Class-2 highway tunnel (80km/h) is illustrated in Fig. B.0.5-1 and B.0.5-2.

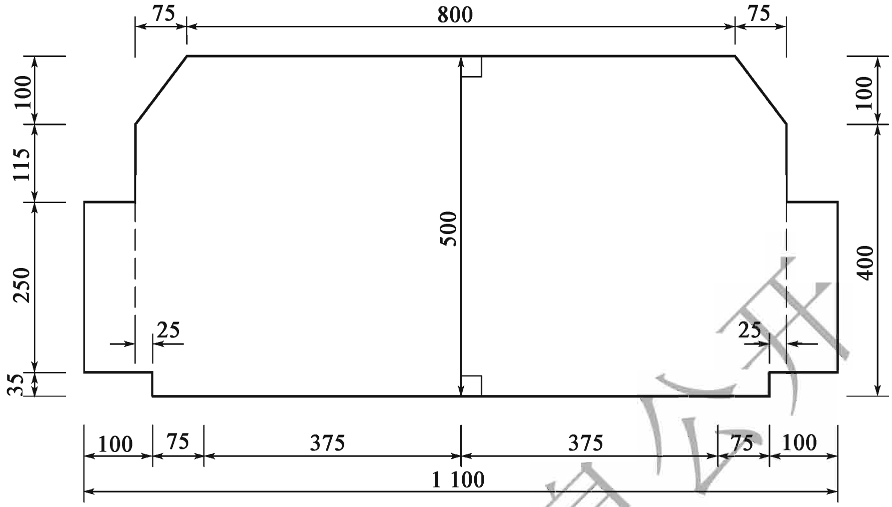


Fig. B.0.5-1 Clearance of two-lane Class-2 highway tunnel (80km/h) (dimensions given in cm)

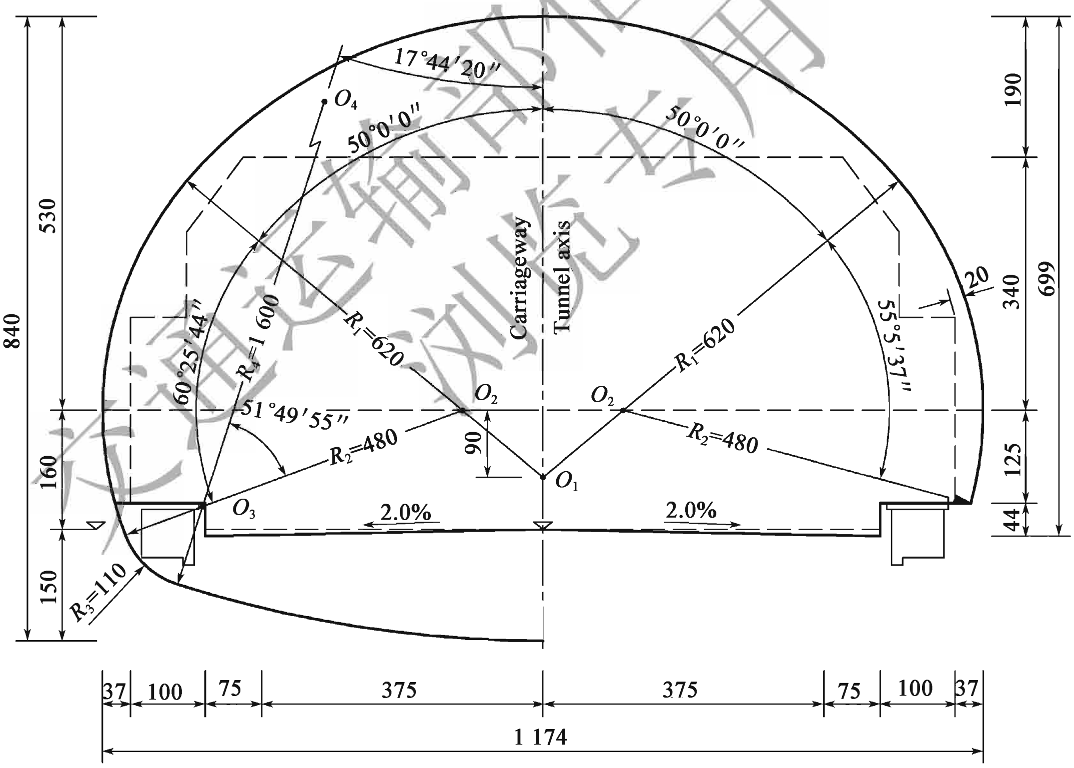


Fig. B.0.5-2 Intrados of two-lane Class-2 highway tunnel (80km/h) (dimensions given in cm)

B.0.6 The clearance of a two-lane Class-1 highway tunnel (60km/h) is illustrated in Fig. B.0.6-1 and B.0.6-2.

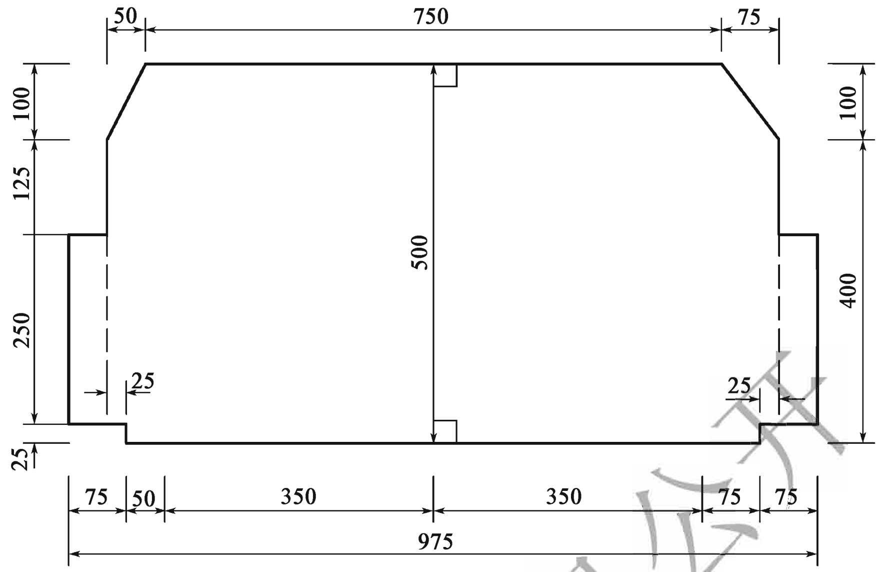


Fig. B.0.6-1 Clearance of two-lane Class-1 highway tunnel (60km/h) (dimensions given in cm)

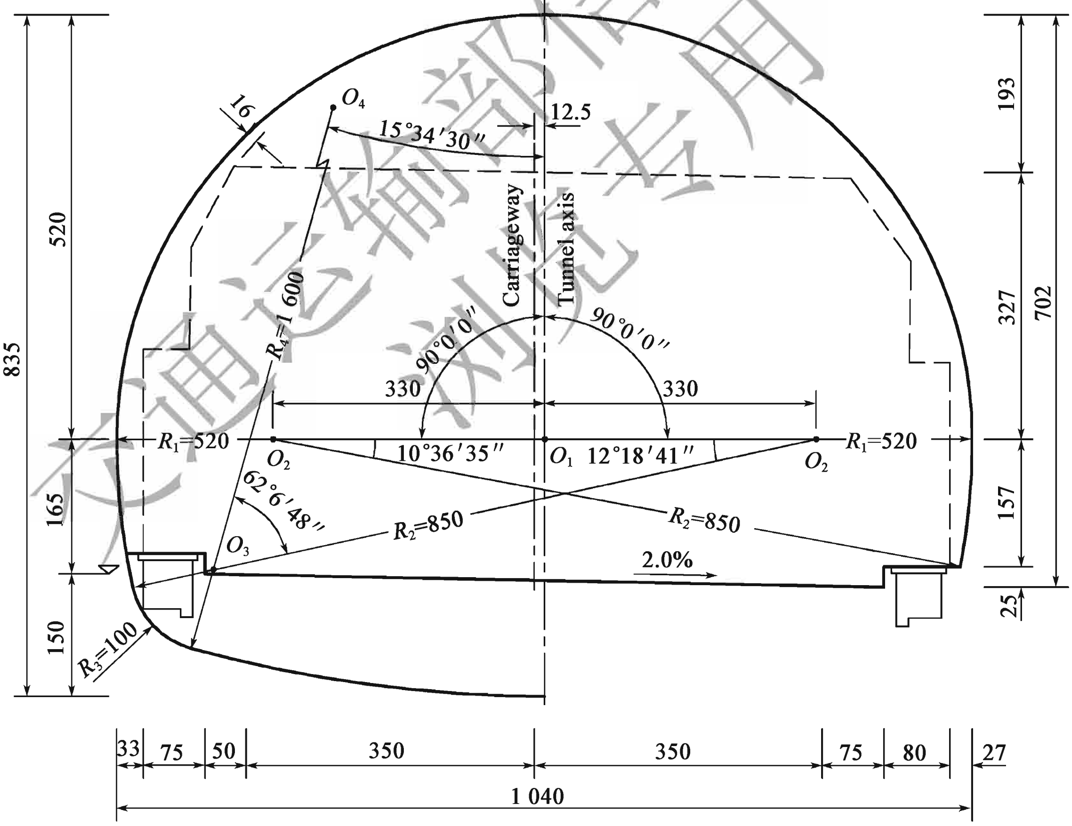


Fig. B.0.6-2 Intrados of two-lane Class-1 highway tunnel (60km/h) (dimensions given in cm)

B.0.7 The clearance of two-lane tunnels on expressway or Class-1 highway (80km/h) is illustrated in Fig. B.0.7-1 and B.0.7-2.

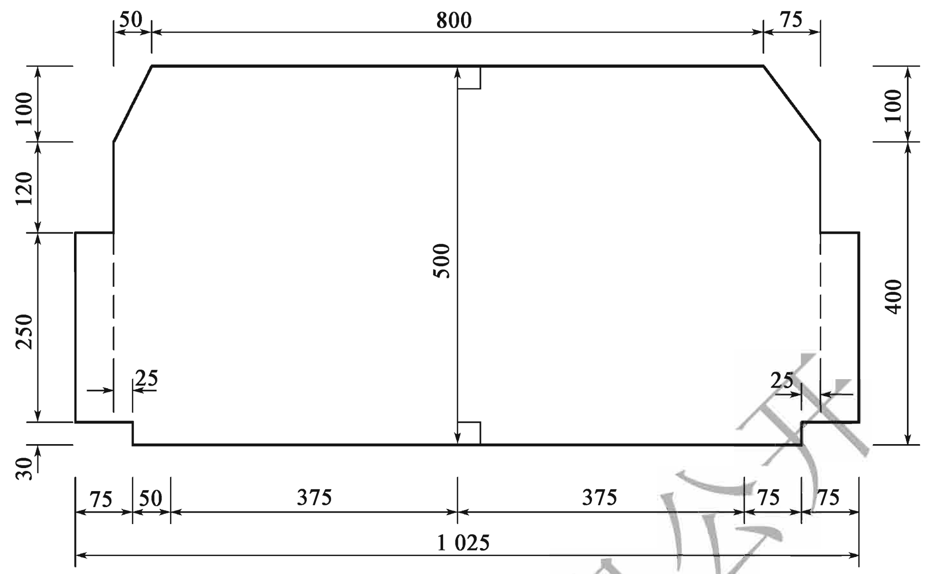


Fig. B.0.7-1 Clearance of two-lane tunnels on expressway or Class-1 highway (80km/h) (dimensions given in cm)

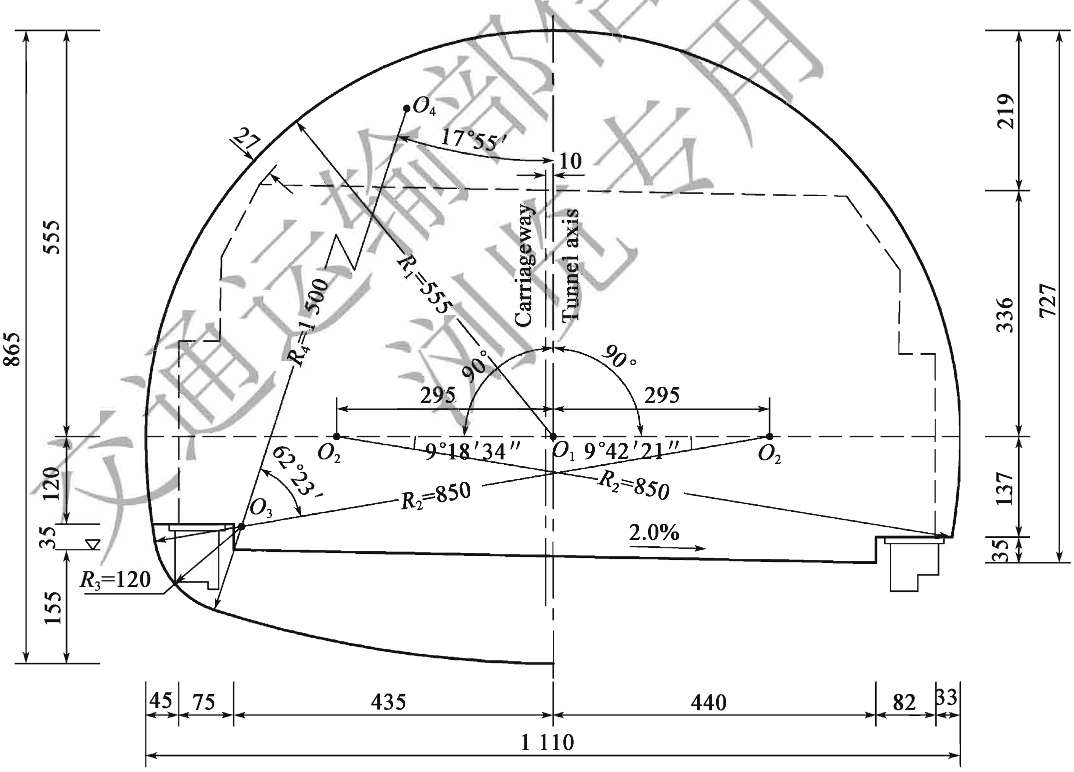


Fig. B.0.7-2 Intrados of two-lane tunnels on expressway or Class-1 highway (80km/h) (dimensions given in cm)

B.0.8 The clearance of two-lane tunnels on expressway or Class-1 highway (100km/h) is illustrated in Fig. B.0.8-1 and B.0.8-2.

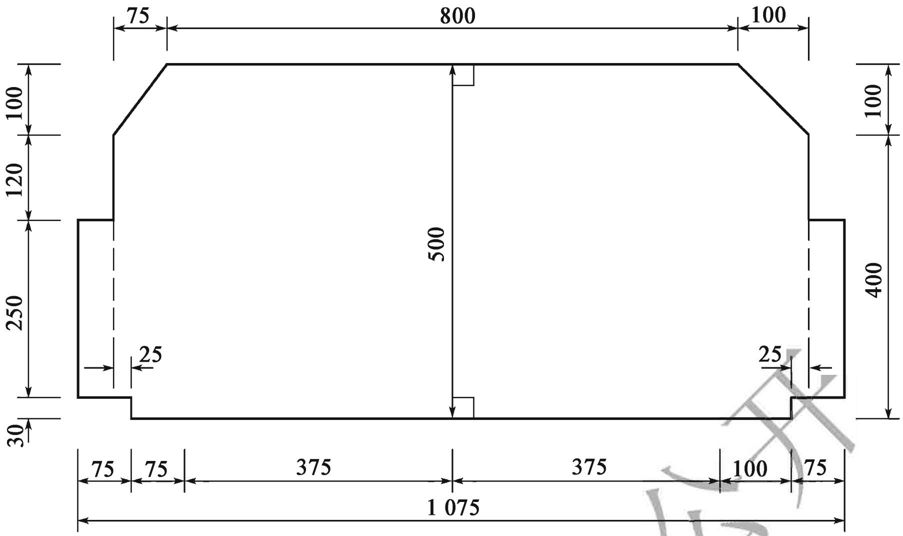


Fig. B.0.8-1 Clearance of two-lane tunnels on expressway or Class-1 highway (100km/h) (dimensions given in cm)

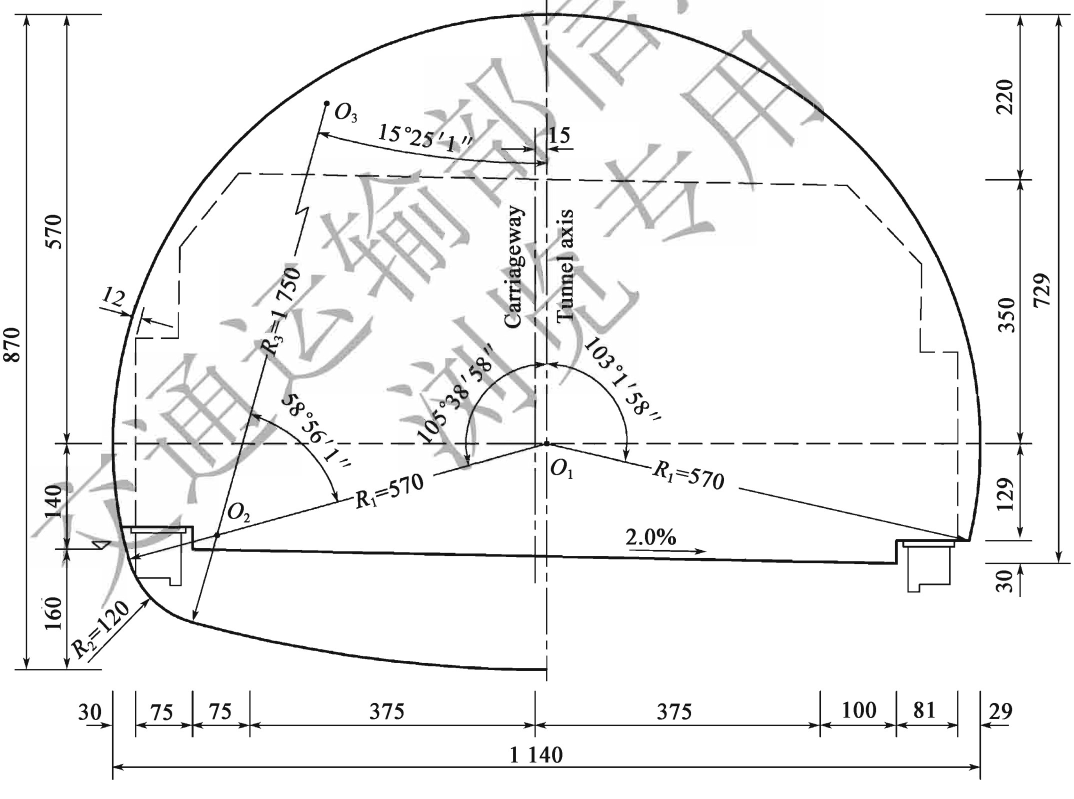


Fig. B.0.8-2 Intrados of two-lane tunnels on expressway or Class-1 highway (100km/h) (dimensions given in cm)

B.0.9 The clearance of two-lane tunnels on expressway or Class-1 highway (120km/h) is illustrated in Fig. B.0.9-1 and B.0.9-2.

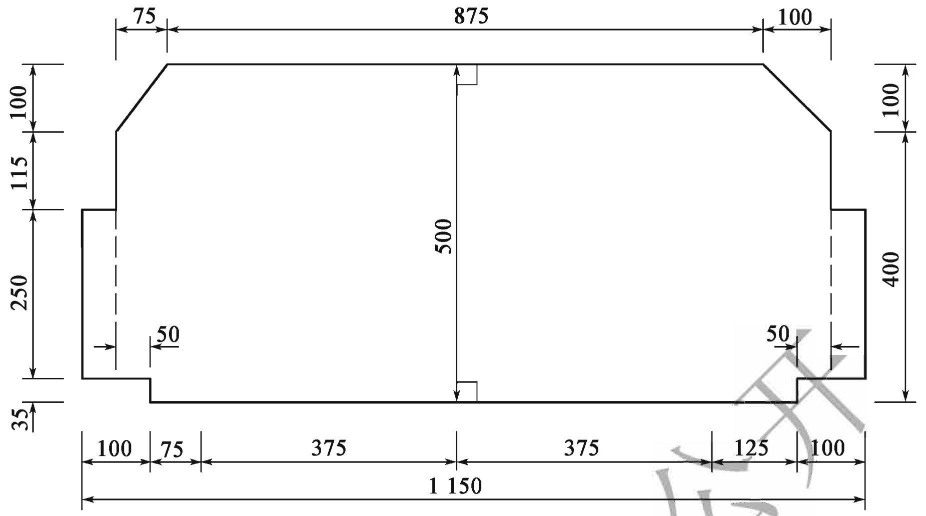


Fig. B.0.9-1 Clearance of expressway two-lane tunnel (120km/h) (dimensions given in cm)

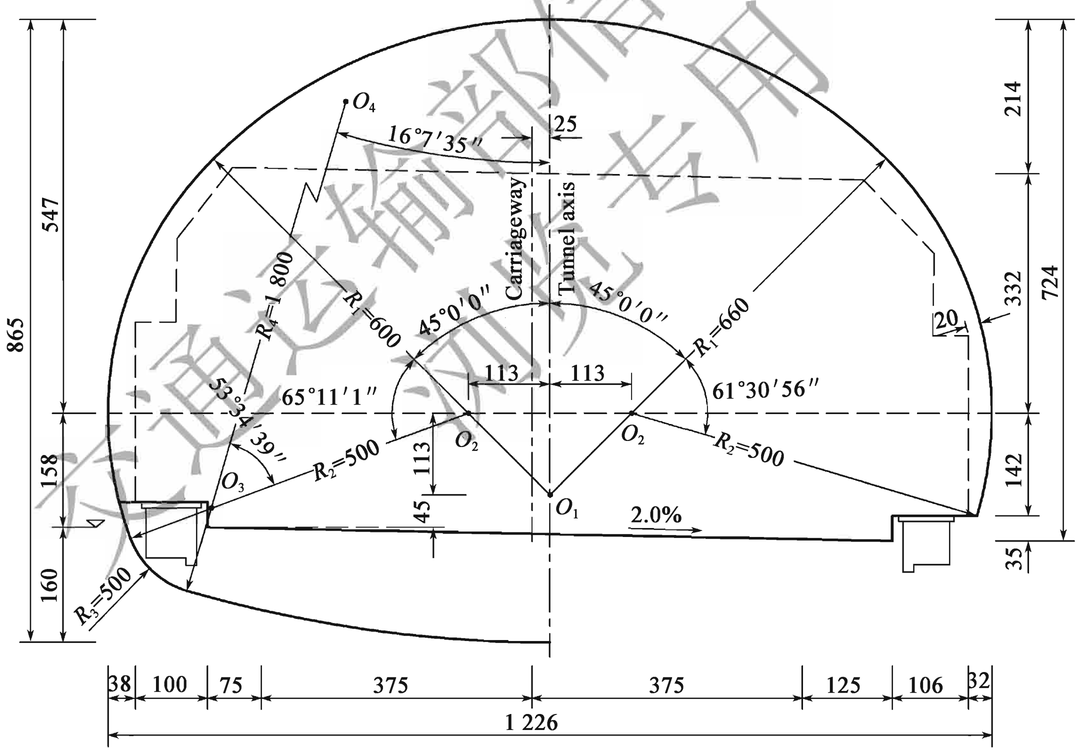


Fig. B.0.9-2 Intrados of expressway two-lane tunnel (120km/h) (dimensions given in cm)

B.0.10 The clearance of a three-lane Class-1 highway tunnel (60km/h) is illustrated in Fig. B.0.10-1 and B.0.10-2.

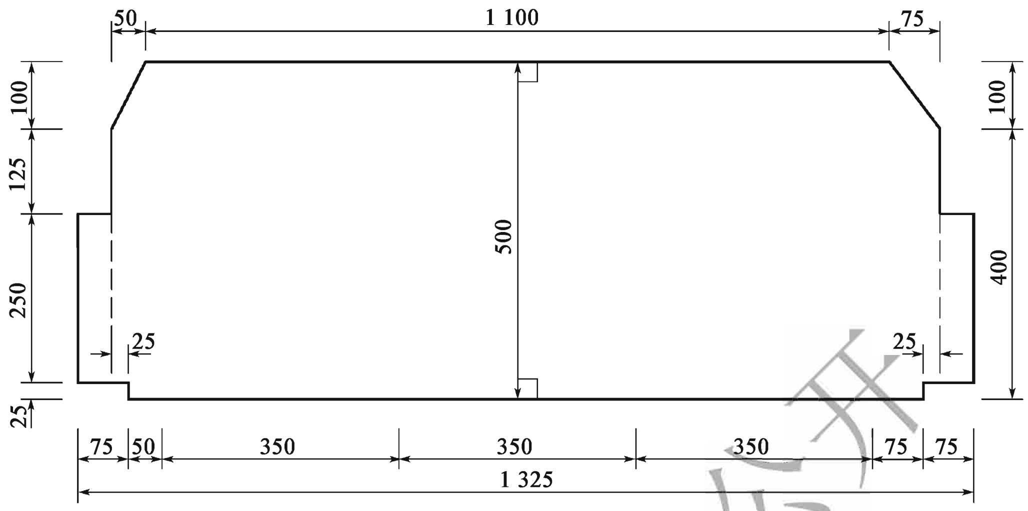


Fig. B.0.10-1 Clearance of three-lane Class-1 highway tunnel (60km/h) (dimensions given in cm)

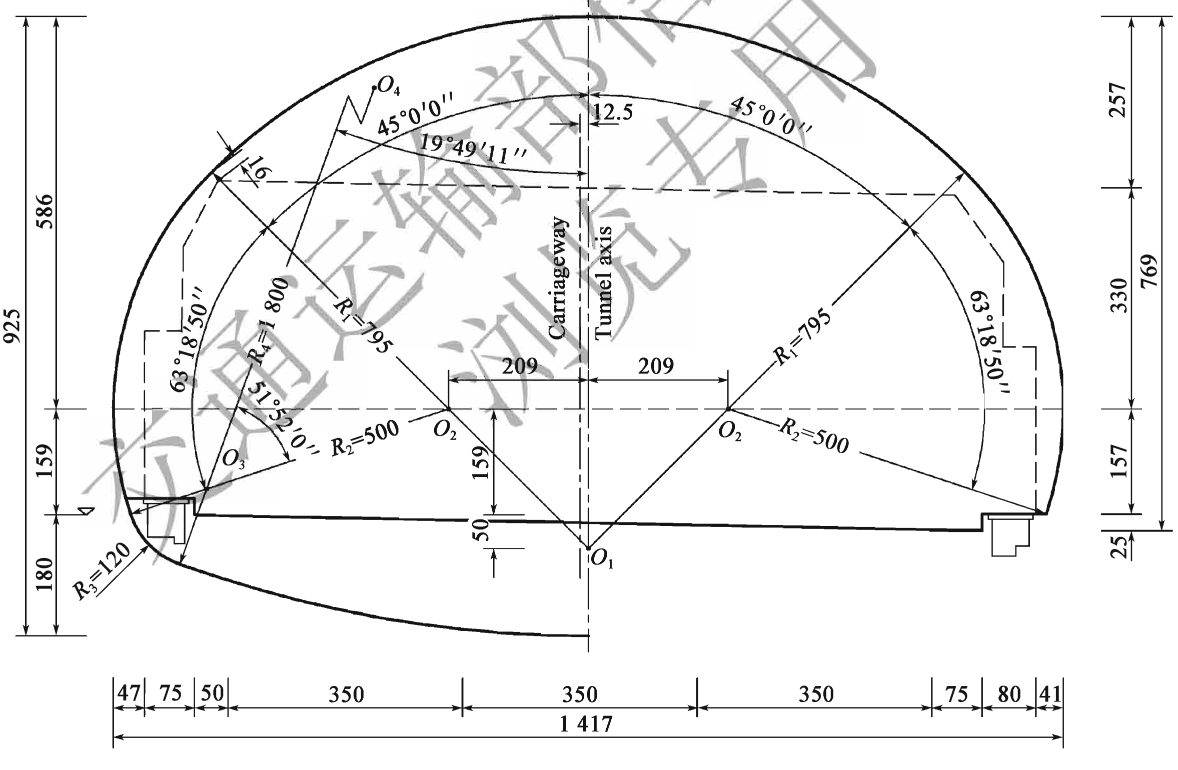


Fig. B.0.10-2 Intrados of three-lane Class-1 highway tunnel (60km/h) (dimensions given in cm)

B.0.11 The clearance of three-lane tunnels on expressway or Class-1 highway (80km/h) is illustrated in Fig. B.0.11-1 and B.0.11-2.

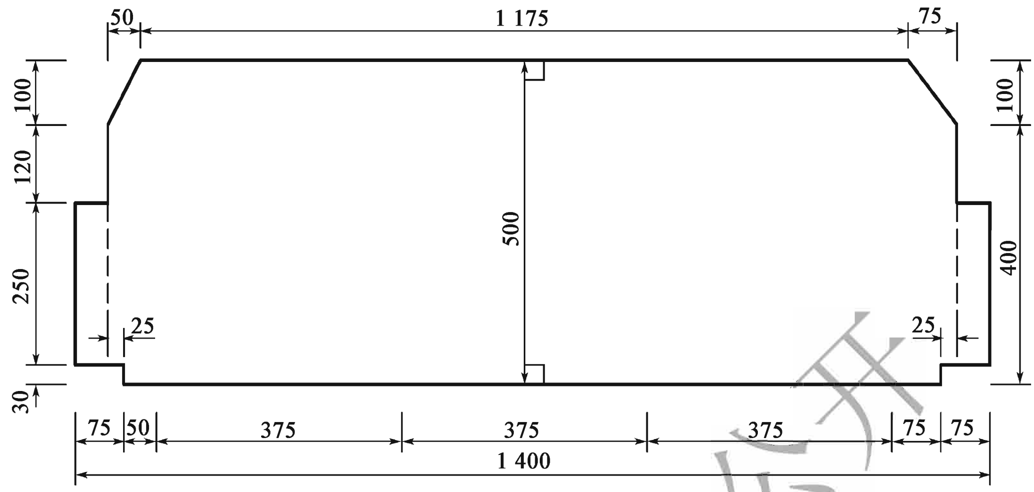


Fig. B.0.11-1 Clearance of three-lane tunnels on expressway or Class-1 highway (80km/h) (dimensions given in cm)

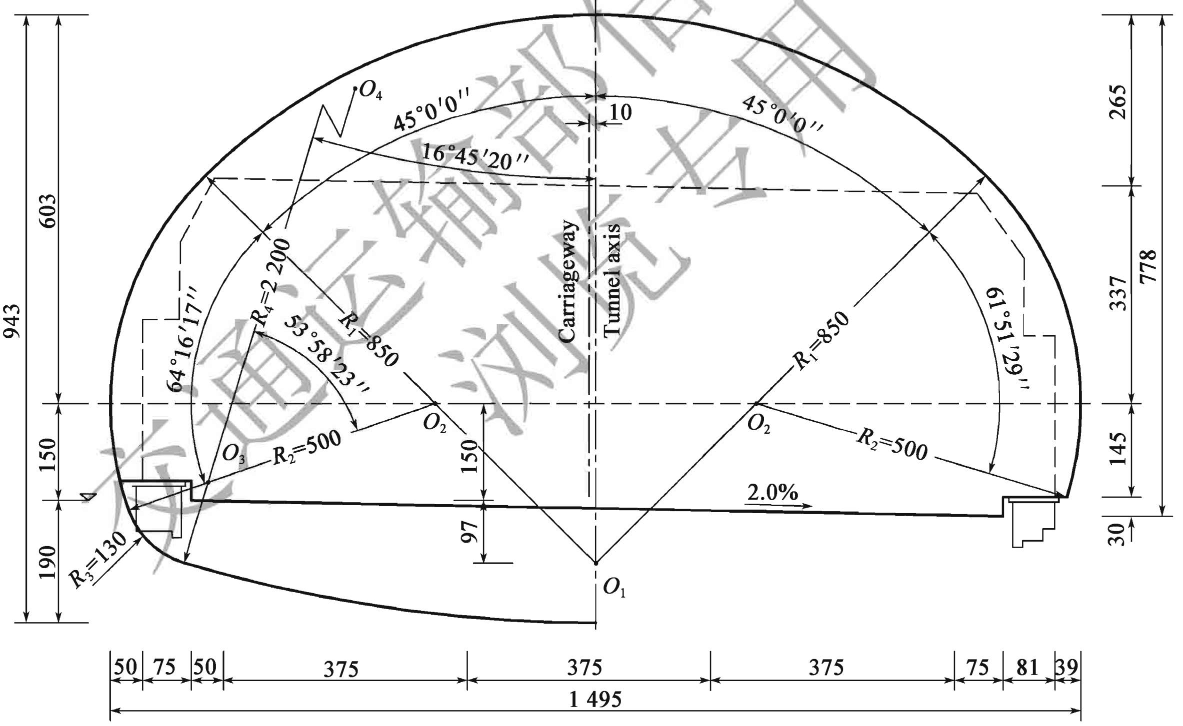


Fig. B.0.11-2 Intrados of three-lane tunnels on expressway or Class-1 highway (80km/h) (dimensions given in cm)

B.0.12 The clearance of three-lane tunnels on expressway or Class-1 highway (100km/h) is illustrated in Fig. B.0.12-1 and B.0.12-2.

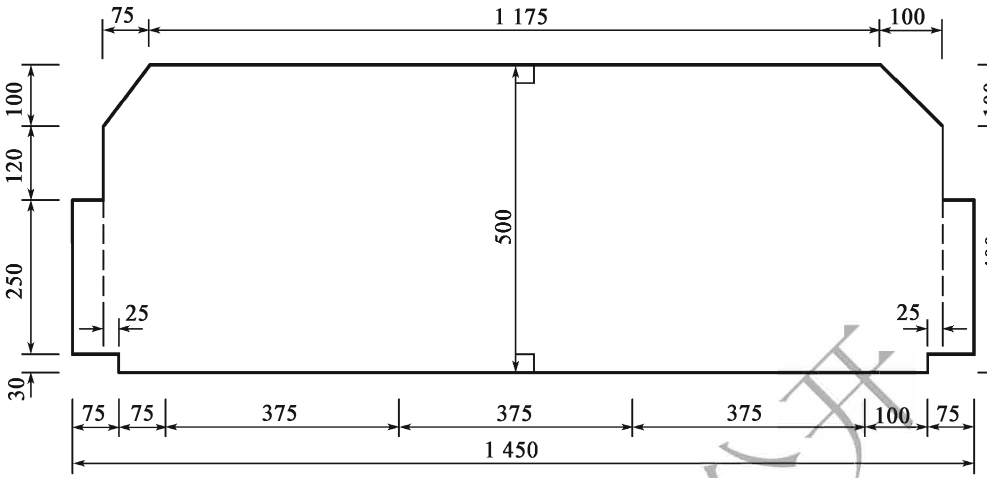


Fig. B. 0. 12-1 Clearance of three-lane tunnels on expressway or Class-1 highway (100km/h) (dimensions given in cm)

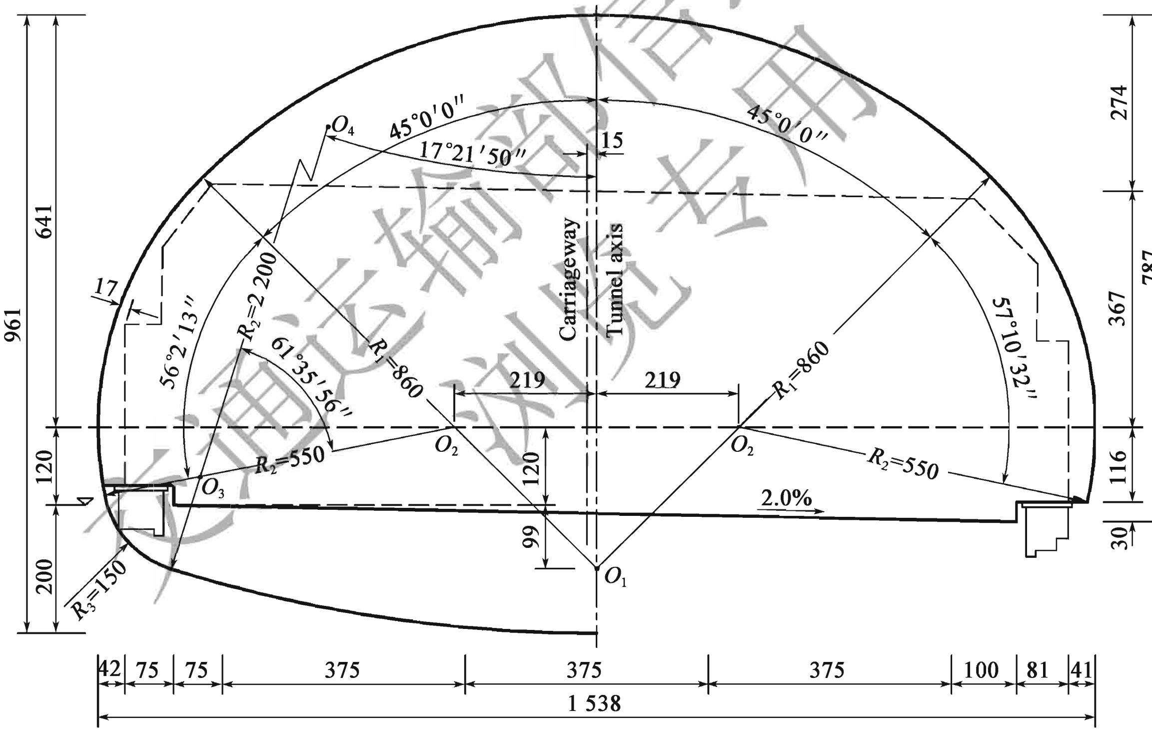


Fig. B. 0. 12-1 Intrados of three-lane tunnels on expressway or Class-1 highway (100km/h) (dimensions given in cm)

B. 0. 13 The clearance of three-lane tunnels on expressway or Class-1 highway (120km/h) is illustrated in Fig. B. 0. 13-1 and B. 0. 13-2.

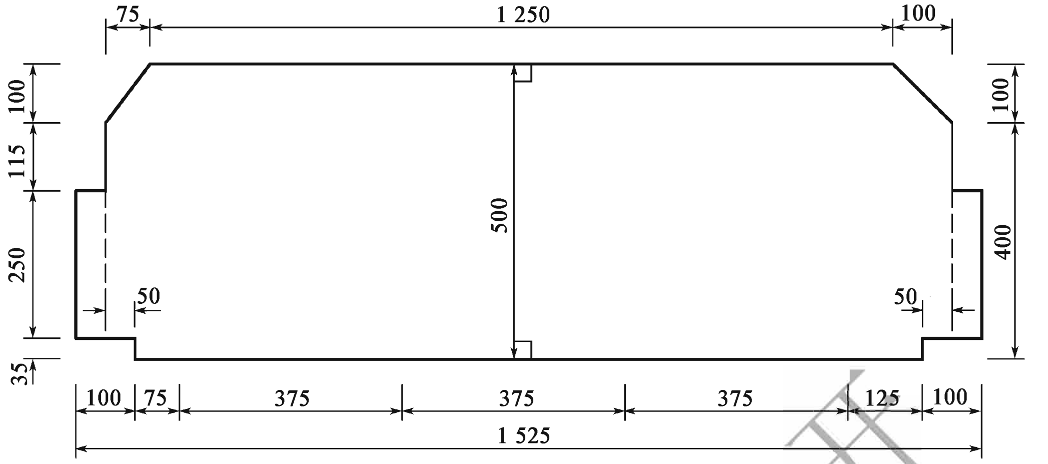


Fig. B. 0. 13-1 Clearance of three-lane tunnels on expressway or Class-1 highway (120km/h) (dimensions given in cm)

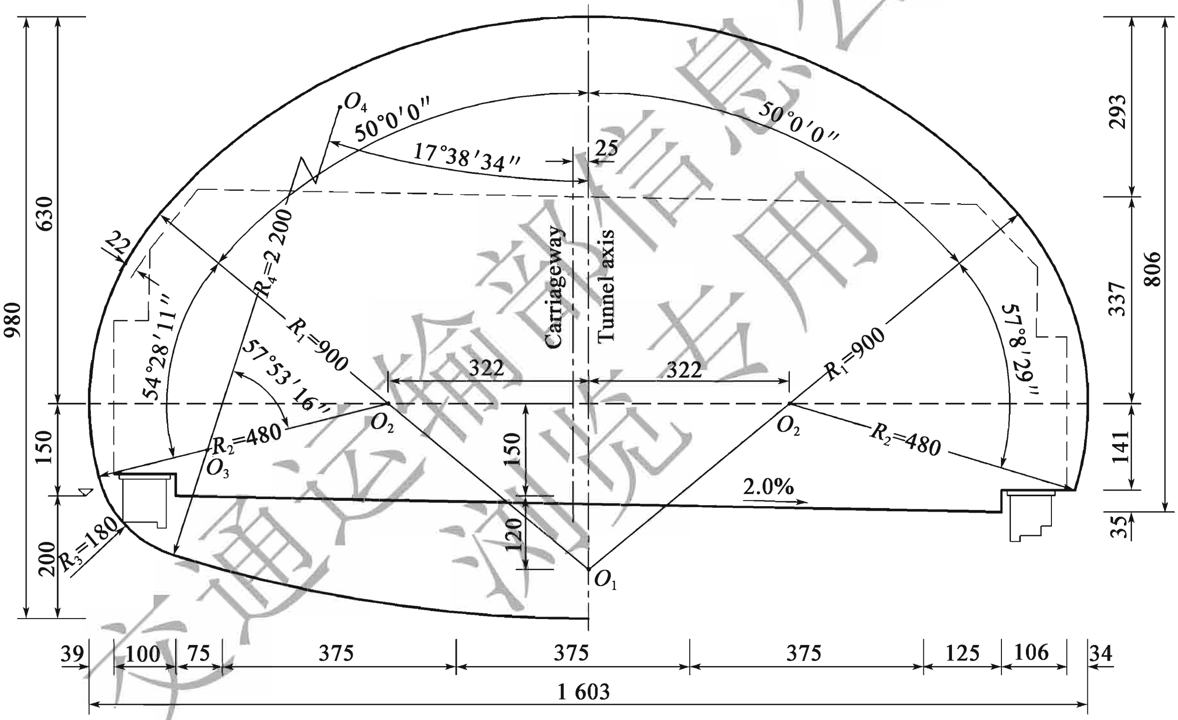


Fig. B. 0. 13-2 Intrados of three-lane tunnels on expressway or Class-1 highway (120km/h) (dimensions given in cm)

Appendix C
Tables of Structural Steel Characteristic Parameters

C.0.1 Sectional characteristic parameters of I-steel may be obtained from Table C.0.1.

Table C.0.1 Sectional characteristic parameters of I-steel

Model	Sectional dimensions (mm)						Sectional area (cm^2)	Theoretical weight (kg/m)	Sectional characteristic parameters						
									X-X				Y-Y		
	h	b	d	t	r	r_1			I_x (cm^4)	W_x (cm^3)	i_x (cm)	I_x, S_X	I_Y (cm^4)	W_Y (cm^3)	i_Y (cm)
I10	100	68	4.5	7.6	6.5	3.3	14.345	11.261	245	49	4.14	8.59	33	9.72	1.52
I12.6	126	74	5.0	8.4	7.0	3.5	18.118	14.223	188	77.5	5.20	10.80	46.9	12.7	1.61
I14	140	80	5.5	9.1	7.5	3.8	21.516	16.890	712	102	5.76	12.00	64.4	16.1	1.73
I16	160	88	6.0	9.9	8.0	4.0	26.131	20.513	1130	141	6.58	13.80	93.1	21.2	1.89
I18	180	94	6.5	10.7	8.5	4.3	30.756	24.143	1660	185	7.36	15.40	122	26.0	2.00
I20a	200	100	7.0	11.4	9.0	4.5	35.578	27.929	2370	237	8.15	17.20	158	31.5	2.12
I20b	200	102	9.0	11.4	9.0	4.5	39.578	31.069	2500	250	7.96	16.90	169	33.1	2.06
I22a	220	110	7.5	12.3	9.5	4.8	42.128	33.070	3400	309	8.99	18.90	225	40.9	2.31
I22b	220	112	9.5	12.3	9.5	4.8	46.528	36.524	3570	325	8.78	18.70	239	42.7	2.27
I25a	250	116	8.0	13.0	10.0	5.0	48.541	38.105	5020	402	10.20	21.60	280	48.3	2.40
I25b	250	118	10.0	13.0	10.0	5.0	53.541	42.030	5280	423	9.94	21.30	309	52.4	2.40
I28a	280	122	8.5	13.7	10.5	5.3	55.404	43.492	7110	508	11.30	24.60	345	56.6	2.50
I32a	320	130	9.5	15.0	11.5	5.8	67.156	52.747	11100	692	12.80	27.50	460	70.8	2.62
I32b	320	132	11.5	15.0	11.5	5.8	73.556	57.741	11600	726	12.60	27.10	502	76.0	2.61
I32c	320	134	13.5	15.0	11.5	5.8	79.956	62.765	12200	760	12.30	26.80	544	81.2	2.61
I36a	360	136	10.0	15.8	12.0	6.0	76.480	60.037	15800	875	14.40	30.70	552	81.2	2.69
I36b	360	138	12.0	15.8	12.0	6.0	83.680	65.689	16500	919	14.10	30.30	582	84.3	2.64

continued

Model	Sectional dimensions (mm)						Sectional area (cm^2)	Theoretical weight (kg/m)	Sectional characteristic parameters						
									X-X				Y-Y		
	h	b	d	t	r	r_1			I_x (cm^4)	W_x (cm^3)	i_x (cm)	I_x, S_X	I_Y (cm^4)	W_Y (cm^3)	i_Y (cm)
I36c	360	140	14.0	15.8	12.0	6.0	90.880	71.341	17300	962	13.80	29.90	612	87.4	2.60
I40a	400	142	10.5	16.5	12.5	6.3	86.112	37.598	21700	1090	15.90	34.10	660	93.2	2.77
I40b	400	144	12.5	16.5	12.5	6.3	94.112	73.878	22800	1140	15.60	33.60	692	96.2	2.71
I40c	400	146	14.5	16.5	12.5	6.3	102.112	80.158	23900	1190	15.20	33.20	727	99.6	2.65
I45a	450	150	11.5	18.0	13.5	6.8	102.446	80.420	32200	1430	17.70	38.60	855	114.0	2.89
I45b	450	152	13.5	18.0	13.5	6.8	111.446	87.485	33800	1500	17.40	38.00	894	118.0	2.84
I45c	450	154	15.5	18.0	13.5	6.8	120.446	94.550	35300	1570	17.10	37.60	938	122.0	2.79
I50a	500	158	12.0	20.0	14.0	7.0	119.304	93.654	46500	1860	19.70	42.80	1120	142.0	3.07
I50b	500	160	14.0	20.0	14.0	7.0	129.304	104.504	48600	1940	19.40	42.40	1170	146.0	3.01
I50c	500	162	16.0	20.0	14.0	7.0	139.304	109.354	50600	2080	19.00	41.80	1220	151.0	2.96
I56a	560	166	12.5	21.0	14.5	7.3	135.435	106.316	65600	2340	22.00	47.70	1370	165.0	3.18
I56b	560	168	14.5	21.0	14.5	7.3	146.635	115.108	68500	2450	21.60	47.20	1490	174.0	3.16
I56c	560	170	16.5	21.0	14.5	7.3	157.835	123.900	71400	2550	21.30	46.70	1560	183.0	3.16
I63a	630	176	13.0	22.0	15.0	7.5	154.658	121.407	93900	2980	24.50	54.20	1700	193.0	3.31
I63b	630	178	15.0	22.0	15.0	7.5	167.258	131.298	98100	3000	24.20	53.50	1810	204.0	3.29
I63c	630	180	17.0	22.0	15.0	7.5	179.858	141.189	102000	3300	23.30	52.90	1920	214.0	3.27

Note: Sectional characteristic parameters in the table: I - moment of inertia; W - section factor; i - radius of inertia; S - net moment of half section.

C.0.2 Sectional characteristic parameters of wide flange H-steel may be obtained from Table C.0.2.

Table C.0.2 Sectional characteristic parameters of wide flange H-steel

Category	Model (height × width)	Sectional dimensions (mm)					Sectional area (cm^2)	Theoretical weight (kg/m)	Sectional characteristic parameters					
									Moment of inertia (cm^4)		Radius of inertia (cm)		Section modulus (cm^3)	
		H	B	T_1	t_2	R			I_x	I_Y	i_x	i_Y	W_x	W_Y
HW	100 × 100	100	100	6	8	8	21.59	16.9	386	134	4.23	2.49	77.1	26.7
	125 × 125	125	125	6.5	9	8	30	23.6	843	293	5.3	3.13	135	46.9
	150 × 150	150	150	7	10	8	39.65	31.1	1620	563	6.39	3.77	216	75.1
	175 × 175	175	175	7.5	11	13	51.43	40.4	2918	983	7.53	4.37	334	112
	200 × 200	200	200	8	12	13	63.53	49.9	4717	1601	8.62	5.02	472	160
		200	204	12	12	13	71.53	56.2	4984	1701	8.35	4.88	498	167

continued

Category	Model (height × width)	Sectional dimensions (mm)					Sectional area (cm^2)	Theoretical weight (kg/m)	Sectional characteristic parameters					
									Moment of inertia (cm^4)		Radius of inertia (cm)		Section modulus (cm^3)	
		H	B	T_1	t_2	R			I_x	I_Y	i_x	i_Y	W_x	W_Y
HW	250 × 250	244	252	11	11	13	81.31	63.8	8573	2937	10.27	6.01	703	233
		250	250	9	14	13	91.43	71.8	10689	3648	10.81	6.32	855	292
		250	255	14	14	13	103.93	81.6	11340	3875	10.45	6.11	907	304
	300 × 300	294	302	12	12	13	106.33	83.5	16384	5513	12.41	7.2	1115	365
		300	300	10	15	13	118.45	93.0	20010	6753	13	7.55	1334	450
		300	305	15	15	13	133.45	104.8	21135	7102	12.58	7.29	1409	466
	350 × 350	338	351	13	13	13	133.27	104.6	27 352	9 376	14.33	8.39	1618	534
		344	348	10	16	13	144.01	113.0	32 545	11242	15.03	8.84	1 892	646
		344	354	16	16	13	164.65	129.3	34 581	11 841	14.49	8.48	2 011	669
		350	350	12	19	13	171.89	134.9	39 637	13 582	15.19	8.89	2 265	776
		350	357	19	19	13	196.39	154.2	42 138	14 427	14.65	8.57	2 408	808
	400 × 400	388	402	15	15	22	178.45	140.1	48 040	16 255	16.41	9.54	2 476	809
		394	398	11	18	22	186.81	146.6	55 597	18 920	17.25	10.06	2 822	951
		394	405	18	18	22	214.39	168.3	59 165	19 951	16.61	9.65	3 003	985
		400	400	13	21	22	218.69	171.7	66 455	22 410	17.43	10.12	3 323	1 120
		400	408	21	21	22	250.69	196.8	70 722	23 804	16.8	9.74	3 536	1 167
		414	405	18	28	22	295.39	231.9	93 518	31022	17.79	10.25	4518	1532
		428	407	20	35	22	360.65	283.1	12 089	39 357	18.31	10.45	5 649	1934
		458	417	30	50	22	528.55	414.9	19 093	60 516	19.01	10.7	8 338	2 902
		498	432	45	70	22	770.05	604.5	30 473	94 346	19.89	11.07	12 238	4 368
	500 × 500	492	465	15	20	22	257.95	202.5	115 559	33 531	21.17	11.4	4 698	1442
		502	465	15	25	22	304.45	239.0	145 012	41910	21.82	11.73	5 777	1 803
		502	470	20	25	22	329.55	258.7	150 283	43 295	21.35	11.46	5 987	1 842
HM	150 × 100	148	100	6	9	8	26.35	20.7	995.3	150.3	6.15	2.39	134.5	30.1
	200 × 150	194	150	6	9	8	38.11	29.9	2 586	506.6	8.24	3.65	266.6	67.6
	250 × 175	244	175	7	11	13	55.49	43.6	5 908	983.5	10.32	4.21	484.3	112.4
	300 × 200	294	200	8	12	13	71.05	55.8	10 858	1602	12.36	4.75	738.6	160.2
	350 × 250	340	250	9	14	13	99.53	78.1	20 867	3 648	14.48	6.05	1227	291.9
	400 × 300	390	300	10	16	13	133.25	104.6	37 363	7 203	16.75	7.35	1916	480.2
	450 × 300	440	300	11	18	13	153.89	120.8	54 067	8 105	18.74	7.26	2 458	540.3
	500 × 300	482	300	11	15	13	141.17	110.8	57 212	6 756	20.13	6.92	2 374	450.4
		488	300	11	18	13	159.17	124.9	67 916	8 106	20.66	7.14	2 783	540.4

continued

Category	Model (height × width)	Sectional dimensions (mm)					Sectional area (cm^2)	Theoretical weight (kg/m)	Sectional characteristic parameters					
									Moment of inertia (cm^4)		Radius of inertia (cm)		Section modulus (cm^3)	
		H	B	T_1	t_2	R			I_x	I_Y	i_x	i_Y	W_x	W_Y
HM	550 × 300	544	300	11	15	13	147. 99	116.2	74 874	6 756	22.49	6.76	2 753	450.4
		550	300	11	18	13	165.99	130.3	88 470	8 106	23.09	6.99	3 217	540.4
	600 × 300	582	300	12	17	13	169. 21	132.8	97 287	7 659	23.98	6.73	3 343	510.6
		588	300	12	20	13	187.21	147.0	112 827	9 009	24. 55	6.94	3 838	600.6
		594	302	14	23	13	217.09	170. 4	132 179	10 572	24.68	6.98	4 450	700.1
HN	100 × 50	100	50	5	7	8	11.85	9.3	191	14.7	4.02	1. 11	38.2	5.9
	25 × 60	125	60	6	8	8	16.69	13. 1	407.7	29. 1	4.94	1.32	65.2	9.7
	150 × 75	150	75	5	7	8	17.85	14.0	645.7	49.4	6.01	1.66	86.1	13.2
	175 × 90	175	90	5	8	8	22.9	18.0	1 174	97.6	7. 16	2.06	134.2	21.6
	200 × 100	198	99	4.5	7	8	22.69	17.8	1 484	113.4	8.09	2.24	149.9	22.9
		200	100	5.5	8	8	26.67	20.9	1753	133.7	8.11	2.24	175.3	26.7
	250 × 125	248	124	5	8	8	31.99	25. 1	3 346	254.5	10.23	2.82	269.8	41. 1
		250	125	6	9	8	36.97	29.0	3 868	293.5	10.23	2.82	309.4	47
	300 × 150	298	149	5.5	8	13	40.8	32.0	5 911	441.7	12.04	3.29	396.7	59.3
		300	150	6.5	9	13	46.78	36.7	6 829	507.2	12.08	3.29	455.3	67.6
	350 × 175	346	174	6	9	13	52.45	41.2	10 456	791. 1	14. 12	3.88	604.4	90.9
		350	175	7	11	13	62.91	49.4	12 980	983.8	14.36	3.95	741.7	112.4
	400 × 150	400	150	8	13	13	70.37	55.2	17 906	733.2	15.95	3.23	895.3	97.8
	400 × 200	396	199	7	11	13	71.41	56.1	19 023	1446	16.32	4.5	960.8	145.3
		400	200	8	13	13	83.37	65.4	22 775	1735	16.53	4.56	1 139	173.5
	450 × 200	446	199	8	12	13	82.97	65.1	27 146	1 578	18.09	4.36	1 217	158.6
		450	200	9	14	13	95.43	74.9	31973	1870	18.3	4.43	1421	187
	500 × 200	496	199	9	14	13	99.29	77.9	39 628	1842	19.98	4.31	1598	185. 1
		500	200	10	16	13	112.25	88. 1	45 685	2 138	20.17	4.36	1 827	213.8
		506	201	11	19	13	129.31	101.5	54 478	2 577	20.53	4.46	2 153	256.4
	550 × 200	546	199	9	14	13	103. 79	81.5	49 245	1842	21.78	4.21	1804	185.2
		550	200	10	16	13	149. 25	117.2	79 515	7 205	23.08	6.95	2 891	480.3
	600 × 200	596	199	10	15	13	117.75	92.4	64 739	1975	23.45	4. 1	2 172	198.5
		600	200	11	17	13	131.71	103.4	73 749	2 273	23.66	4.15	2 458	227.3
		606	201	12	20	13	149.77	117.6	86 656	2 716	24.05	4.26	2 860	270.2
	650 × 300	646	299	10	15	13	152. 75	119.9	107 794	6 688	26.56	6.62	3 337	447.4
		650	300	11	17	13	171.21	134.4	122 739	7 657	26.77	6.69	3 777	510.5

continued

Category	Model (height × width)	Sectional dimensions (mm)					Sectional area (cm^2)	Theoretical weight (kg/m)	Sectional characteristic parameters					
		H	B	T_1	t_2	R			Moment of inertia (cm^4)		Radius of inertia (cm)		Section modulus (cm^3)	
									I_x	I_Y	i_x	i_Y	W_x	W_Y
HN	650 × 300	656	301	12	20	13	195.77	153.7	144 433	9 100	27. 16	6.82	4 403	604.6
	700 × 300	692	300	13	20	18	207.54	162.9	164101	9014	28.12	6.59	4743	600.9
		700	300	13	24	18	231.54	181.8	193622	10814	28.92	6.83	5532	720.9
	750 × 300	734	299	12	16	18	182.7	143.4	155539	7140	29.18	6.25	4238	477.6
		742	300	13	20	18	214.04	168.0	191989	9015	29.95	6.49	5175	601
		750	300	13	24	18	238.04	186.9	225863	10815	30.8	6.74	6023	721
		758	303	16	28	18	284.78	223.6	271350	13008	30.87	6.76	7160	858.6
	800 × 300	792	300	14	22	18	239.5	188.0	242399	9919	31.81	6.44	6121	661.3
		800	300	14	26	18	263.5	206.8	280925	11719	32.65	6.67	7023	781.3
	850 × 300	834	298	14	19	18	227.46	178.6	243858	8400	32.74	6.08	5848	563.8
		842	299	15	23	18	259.72	203.9	291216	10271	33.49	6.29	6917	687
		850	300	16	27	18	292.14	229.3	339670	12179	34.1	6.46	7992	812
		858	301	17	31	18	324.72	254.9	389234	14125	34.62	6.6	9073	938.5
	900 × 300	890	299	15	23	18	266.92	209.5	330588	10273	35.19	6.2	7429	687.1
		900	300	16	28	18	305.82	240.1	397241	12631	36.04	6.43	8828	842.1
		912	302	18	34	18	360.06	282.6	484615	15652	36.69	6.59	10628	1037
	1000 × 300	970	297	16	21	18	276	216.7	382977	9203	37.25	5.77	7896	619.7
		980	298	17	26	18	315.5	247.7	462157	11508	38.27	6.04	9432	772.3
		990	298	17	31	18	345.3	271.1	535201	13713	39.37	6.3	10812	920.3
		1000	300	19	36	18	395.1	310.2	626396	16256	39.82	6.41	12528	1084
		1008	302	21	40	18	439.26	344.8	704572	18437	40.05	6.48	13980	1221
HT	100 × 50	95	48	3.2	4.5	8	7.62	6.0	109.7	8.4	3.79	1.05	23.1	3.5
		97	49	4	5.5	8	9.38	7.4	141.8	10.9	3.89	1.08	29.2	4.4
	100 × 100	96	99	4.5	6	8	16.21	12.7	272.7	97.1	4.1	2.45	56.8	19.6
	125 × 60	118	58	3.2	4.5	8	9.26	7.3	202.4	14.7	4.68	1.26	34.3	5.1
		120	59	4	5.5	8	11.4	8.9	259.7	18.9	4.77	1.29	43.3	6.4
	125 × 125	119	123	4.5	6	8	20.12	15.8	523.6	186.2	5.1	3.04	888	30.3
	150 × 75	145	73	3.2	4.5	8	11.47	9.0	383.2	29.3	5.78	1.6	52.9	8
		147	74	4	5.5	8	14.13	11.1	488	37.3	5.88	1.62	66.4	10.1
	150 × 100	139	97	3.2	4.5	8	13.44	10.5	447.3	68.5	5.77	2.26	64.4	14.1
		142	99	4.5	6	8	18.28	14.3	632.7	97.2	5.88	2.31	89.1	19.6
	150 × 150	144	148	5	7	8	27.77	21.8	1070	378.4	6.21	3.69	148.6	51.1

continued

Category	Model (height × width)	Sectional dimensions (mm)					Sectional area (cm^2)	Theoretical weight (kg/m)	Sectional characteristic parameters					
									Moment of inertia (cm^4)		Radius of inertia (cm)		Section modulus (cm^3)	
		H	B	T_1	t_2	R			I_x	I_Y	i_x	i_Y	W_x	W_Y
HT	150 × 150	147	149	6	8.5	8	33.68	26.4	1338	468.9	6.3	3.73	182.1	62.9
	175 × 90	168	88	3.2	4.5	8	13.56	10.6	619.6	51.2	6.76	1.94	73.8	11.6
		171	89	4	6	8	17.59	13.8	852.1	70.6	6.96	2	99.7	15.9
	175 × 175	167	173	5	7	13	33.32	26.2	1731	604.5	7.21	4.26	207.2	69.9
		172	175	6.5	9.5	13	44.65	35.0	2466	849.2	7.43	4.36	286.8	97.1
	200 × 100	193	98	3.2	4.5	8	15.26	12.0	921	70.7	7.77	2.15	95.4	14.4
		196	99	4	6	8	19.79	15.5	1260	97.2	7.98	2.22	128.6	19.6
	200 × 150	188	149	4.5	6	8	26.35	20.7	1669	331	7.96	3.54	177.6	44.4
	200 × 200	192	198	6	6	13	43.69	34.3	2984	1036	8.26	4.87	310.8	104.6
	250 × 125	244	124	4.5	4.5	8	25.87	20.3	2529	190.9	9.89	2.72	207.3	30.8
	250 × 175	238	173	4.5	4.5	13	39.12	30.7	4045	690.8	10.17	4.2	339.9	79.9
	300 × 150	294	148	4.5	4.5	13	31.9	25.0	4342	324.6	11.67	3.19	295.4	43.9
	300 × 200	286	198	6	6	13	49.33	38.7	7000	1036	11.91	4.58	489.5	104.6
	350 × 175	340	173	4.5	4.5	13	36.97	29.0	6823	518.3	13.58	3.74	401.3	59.9
	400 × 150	390	148	6	6	13	47.57	37.3	10900	433.2	15.14	3.02	559	58.5
	400 × 200	390	198	6	6	13	55.57	43.6	13819	1036	15.77	4.32	708.7	104.6

Note: Sectional characteristic parameters in the table: I - moment of inertia; W - section factor; i - radius of inertia; S - net moment of half section.

C.0.3 Section and notations of I-steel are presented in Fig. C.0.3.

C.0.4 Section and notations of H-steel are presented in Fig. C.0.4.

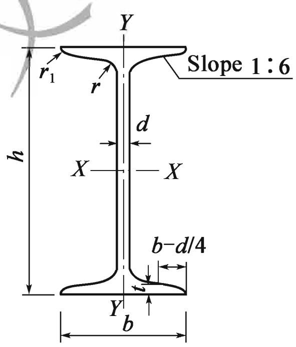


Fig. C.0.3 Section and notations of I-steel

h-height; b-leg height; d-web thickness; t-average leg thickness; r-inner arc radius; r_1- leg end arc radius

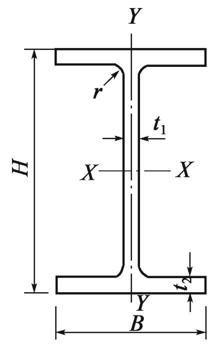


Fig. C.0.4 Section and notations of H-steel

H-height; B-width; t_1-web thickness; t_2-flange thickness; r-process arc radius

C.0.5 Sectional characteristic parameters of U-steel may be obtained from Table C.0.5.

Table C.0.5 Sectional characteristic parameters of U-steel

Specification	Sectional dimensions (mm)																					
	H_1	H_2	H_3	B_1	B_2	B_3	B_4	B_5	B_6	B_7	M	b	c	d	R_1	R_2	R_3	R_4	r_1	r_2	r_3	α	β
18UY	99	18	10	122	84	57	—	—	46.2	—	7.5	—	2	2	—	—	9	9	8	4	2	—	—
25UY	110	26	17	134	92	50.8	45	73.8	45	94.1	6.6	—	—	2.5	400	400	12	10	7	2	—	—	—
25U	120	29	15	135	101.5	40	47	39	29	102.3	6.3	1.3	—	0	450	175	14	12	6	4	10	18	2
29U	124	28.5	16	150.5	116	44	53	42	30	116.6	7.2	3	—	0	450	185	15	16	7	4	—	40	3
36U	138	31.5	17	171	128	50.5	60.5	48.5	35	129.3	7.8	4	—	0	500	200	20	20	9	4	—	40	3
40U	141.9	34.7	20.2	171	128.5	50.5	60.5	48.5	35	129.3	8.5	3.5	—	0	500	200	20	20	9	4	—	40	3

Specification	Sectional area	Theoretical weight	Sectional parameters						
			Moment of inertia		Radius of inertia		Section modulus		static moment
			I_X	I_Y	i_x	i_y	W_x	W_y	S_x
18UY	24.15	18.96	284.26	331.35	3.43	3.70	52.29 57.43	54.32	75.40
25UY	31.54	24.76	451.70	508.70	3.78	4.02	81.68 82.58	75.92	110.90
25U	31.79	24.95	495.81	551.97	3.95	4.17	79.77 85.71	81.77	197.54
29U	37.00	29.00	612.00	771.00	4.07	4.57	106.00 92.00	102.00	212.91
36U	45.69	35.87	928.65	1244.75	4.51	5.22	128.55 141.22	145.59	330.05
40U	51.02	40.05	1064.07	1366.98	4.57	5.18	141.60	159.94	388.37

Note: 25UY: 37.7mm; H5: 46.6mm.

Appendix D
Calculation Method for Rock Load for Shallow Tunnels

D.0.1 The boundary between shallow and deep tunnels may be determined by equivalent height for loads taken into account the geological conditions, construction methods and other factors. In this case Eq. (D.0.1-1) and (D.0.1-2) may be used:

$$H_p = (2 \sim 2.5) h_q \quad \text{(D.0.1-1)}$$

$$h_q = \frac{q}{\gamma} \quad \text{(D.0.1-2)}$$

where

H_p—boundary depth for shallow tunnels (m);

h_q—equivalent height for loads (m);

q—uniformly distributed vertical pressure on deep tunnel obtained from Eq. (6.2.3) (kN/m^2);

γ—unit weight of surrounding rock (kN/m^3).

If the tunnel is constructed by Drill & Blast method or shallow mining method, for Class IV-VI surrounding rock:

$$H_p = 2.5 h_q \quad \text{(D.0.1-3)}$$

For Class I-III surrounding rock:

$$H_p = 2 h_q \quad \text{(D.0.1-4)}$$

D.0.2 The pressure from rock surrounding a shallow tunnel may be calculated under the following two conditions respectively:

1 If the tunnel depth H is less than or equal to equivalent height for loads h_q, vertical pressure is regarded as uniformly distributed:

$$q = \gamma \cdot H \qquad \text{(D. 0. 2-1)}$$

where

q—vertical uniform pressure (kN/m^2);

γ—unit weight of overlying rock above the tunnel (kN/m^3);

H—tunnel depth, i. e. the distance from arc crown to ground surface (m).

When considered as uniformly distributed, lateral pressure e:

$$e = \gamma\left(H + \frac{1}{2}H_t\right)\tan^2\left(45 - \frac{\varphi_c}{2}\right) \qquad \text{(D. 0. 2-2)}$$

where

e—lateral uniform pressure (kN/m^2);

H_t—tunnel height (m);

φ_c—calculated friction angle of surrounding rock (°), to be obtained from Tables A. 0. 7-1 and A. 0. 7-2.

2 If tunnel depth is greater than h_q, and less than or equal to H_p, assume the fracture plane in rock and soil mass is an oblique line at an angle of β with horizontal line, as shown in Fig. D. 0. 2-1. The settlement of EFHG mass results in settlement of triangular rock mass on both sides (such as FDB and ECA in the figure. The settlement of the entire ABDC mass is resisted by undisturbed rock mass. Oblique line AC or BD is the assumed fracture plane. Cohesion c is considered in analysis and calculated friction angle φ_c is used. The other sliding surface FH or EG is not fracture plane. Therefore, the sliding surface resistance shall be less than resistance of fracture planes AC and BD. If the friction angle of this sliding surface is θ, then the value of θ shall be less than the value of φ_c. If no measurement data are available, θ may be obtained from Table D. 0. 2.

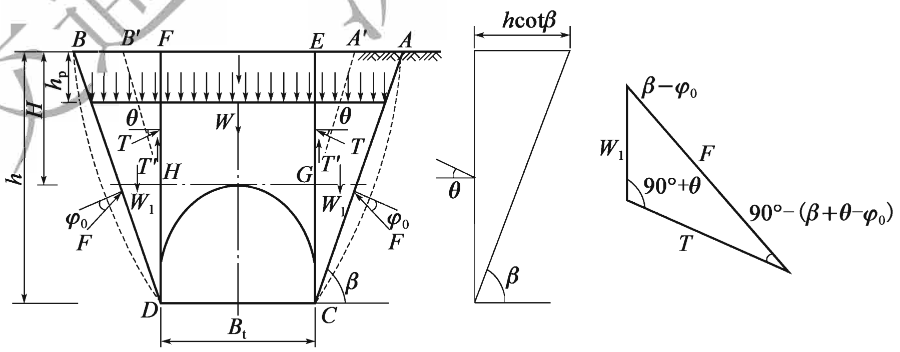


Fig. D. 0. 2-1 Schematic diagram of pressure from rock surrounding shallow tunnel

Table D. 0. 2 Values of θ in various surrounding rocks

Surrounding rock class	Ⅰ, Ⅱ, Ⅲ	Ⅳ	Ⅴ	Ⅵ
Value of θ	$0.9\varphi_c$	$(0.7 \sim 0.9)\varphi_c$	$(0.5 \sim 0.7)\varphi_c$	$(0.3 \sim 0.5)\varphi_c$

As shown in Fig. D.0.2-1, the gravity of overlying rock above the tunnel is W, the gravity of the wedge FDB or ECA on either side is W_1 and the resistance of undisturbed rock mass to the entire sliding body is F. When EFHG settles, resistance to its two sides is T or T'. Total vertical pressure acting upon HG plane, $Q_{shallow}$ is:

$$Q_{shallow} = W - 2T' = W - 2T\sin\theta \quad (D.0.2\text{-}3)$$

Dead weight of the wedge is:

$$W_1 = \frac{1}{2}\gamma h \frac{h}{\tan\beta} \quad (D.0.2\text{-}4)$$

where

h—the distance from tunnel bottom to ground surface (m);

β—angle between fracture plane and horizontal plane (°).

From the figure and The Law of Sines the following is obtained:

$$T = \frac{\sin(\beta - \varphi_c)}{\sin[90° - (\beta - \varphi_c + \theta)]} W_1 \quad (D.0.2\text{-}5)$$

Substitute Eq. (D.0.2-4) into the above equation to obtain:

$$T = \frac{1}{2}\gamma h^2 \frac{\gamma}{\cos\theta} \quad (D.0.2\text{-}6)$$

$$\lambda = \frac{\tan\beta - \tan\varphi_c}{\tan\beta[1 + \tan\beta(\tan\varphi_c - \tan\theta) + \tan\varphi_c \tan\theta]} \quad (D.0.2\text{-}7)$$

$$\tan\beta = \tan\varphi_c + \sqrt{\frac{(\tan^2\varphi_c + 1)\tan\varphi_c}{\tan\varphi_c - \tan\theta}} \quad (D.0.2\text{-}8)$$

where

λ—lateral pressure coefficient.

Meanings of other symbols are the same as above.

At this point the maximum resistance T can be obtained. Substitute the obtained T into Eq. (D.0.2-3) to obtain total vertical pressure acting on HG plane, $Q_{shallow}$:

$$Q_{shallow} = W - 2T\sin\theta = W - \gamma h^2 \lambda \tan\theta \quad (D.0.2\text{-}9)$$

Since GC and HD are often less than EG and EF and the friction angle between the lining and rock/soil masses varies, θ has been used in the previous calculation. When the middle mass slides transferring load from FH and EG planes, a slightly higher pressure considered is safer to the designed structure. Therefore, only the frictional resistance at arc crown is considered whereas that at the tunnel proper is excluded, i.e. replace h with H in calculation, changing Eq. (D.0.2-9) to:

$$Q_{浅} = W - \gamma H^2 \lambda \tan\theta$$

Because $W = B_t H\lambda$:

$$Q_{shallow} = \gamma H(B_t - H\lambda\tan\theta) \tag{D. 0. 2-10}$$

where

B_t——tunnel width (m).

It is converted into uniformly distributed load acting on the support structure, as shown in Fig. D.0.2-2:

$$q_{shallow} = \frac{Q_{shallow}}{B_t} = \gamma H\left(1 - \frac{H}{B_t}\lambda\tan\theta\right) \tag{D. 0. 2-11}$$

where

q_{shallow}—uniformly distributed load acting on support structure (kN/m^2).

Meanings of other symbols are the same as above.

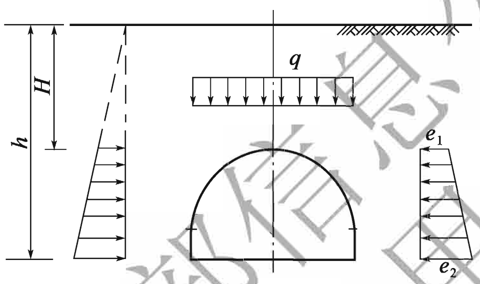


Fig. D.0.2-2 Schematic diagram of uniformly distributed load

Horizontal lateral pressure on both sides of the support structure:

$$\left.\begin{aligned} e_1 &= \gamma H\lambda \\ e_2 &= \gamma h\lambda \end{aligned}\right\} \tag{D. 0. 2-12}$$

When regarded as uniformly distributed, the lateral pressure is:

$$e = \frac{1}{2}(e_1 + e_2) \tag{D. 0. 2-13}$$

Appendix E
Calculation Method for Pressure from Rock Surrounding Shallow Eccentric load Tunnels

E.0.1 Vertical pressure on eccentric load tunnel is calculated from:

$$Q = \frac{\gamma}{2}\left[(h + h')B - (\lambda h^2 + \lambda' h'^2)\tan\theta\right] \tag{E.0.1-1}$$

Assume the eccentric load distribution diagram is consistent with ground slope (Fig. E.0.1).

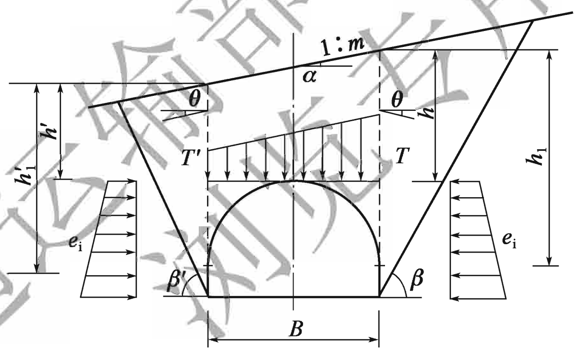


Fig. E.0.1 Eccentric load distribution

where

h, h'—height from inner and outer crown level to ground surface (m);

B—tunnel span (m);

γ—unit weight of surrounding rock (kN/m^3);

θ—friction angle on both sides of roof column (°) which may be obtained from Table D.0.2 if no measurement data are available;

λ, λ'—inner and outer lateral pressure coefficient, calculated from:

$$\lambda = \frac{1}{\tan\beta - \tan\alpha} \times \frac{\tan\beta - \tan\varphi_c}{1 + \tan\beta(\tan\varphi_c - \tan\theta) + \tan\varphi_c \tan\theta} \tag{E.0.1-2}$$

$$\lambda' = \frac{1}{\tan\beta' - \tan\alpha} \times \frac{\tan\beta' - \tan\varphi_c}{1 + \tan\beta'(\tan\varphi_c - \tan\theta) + \tan\varphi_c \cdot \tan\theta} \tag{E.0.1-3}$$

$$\tan\beta = \tan\varphi_c + \sqrt{\frac{(\tan^2\varphi_c + 1)(\tan\varphi_c - \tan\alpha)}{\tan\varphi_c - \tan\theta}} \tag{E. 0. 1-4}$$

$$\tan\beta' = \tan\varphi_c + \sqrt{\frac{(\tan^2\varphi_c + 1)(\tan\varphi_c + \tan\alpha)}{\tan\varphi_c - \tan\theta}} \tag{E. 0. 1-5}$$

where

α—ground slope angle (°);

φ_c—Calculated friction angle of surrounding rock (°);

β,β'—inner and outer fracture angle at maximum thrust (°).

E. 0. 2 Horizontal lateral pressure on pressure on eccentric load tunnel is calculated as follows:

Inner side:

$$e_i = \gamma \cdot h_i\lambda \tag{E. 0. 2-1}$$

Outer side:

$$e_i = \gamma \cdot h'_i\lambda' \tag{E. 0. 2-2}$$

where

h_i, h'_i —the distance from any point i on inner and outer side to ground surface (m).

Appendix F
Calculation Method for Pressure from Rock Surrounding Twin tunnels with narrow pillar

F. 0. 1 Shallow twin tunnels with narrow pillar in any geological conditions shall undergo internal force calculation, analysis and strength check by load - structure method. For deep twin tunnels with narrow pillar, the lining structure in Class IV - VI surrounding rocks requires internal force calculation, analysis and strength check.

F. 0. 2 To determine a twin tunnels with small clearance is shallow or deep, assessment may be made according to equivalent height for loads, geological conditions and construction methods, using Eq. (F. 0. 2-1) and (F. 0. 2-2):

$$H_p = (2 \sim 2.5) h_q \tag{F. 0. 2-1}$$

$$h_q = h_{q1} + h'_{q2} \tag{F. 0. 2-2}$$

where,

H_p—boundary depth for twin tunnels with narrow pillar (m);

h_q—equivalent height for vertical loads on inner side of the arch of deep twin tunnels with narrow pillar (m);

h_{q1}—equivalent height for basic vertical loads on deep twin tunnels with narrow pillar (m), calculated from Eq. (F. 0. 3-4);

h'_{q2}—equivalent height for additional vertical loads on inner side of deep twin tunnels with narrow pillar (m), calculated from Eq. (F. 0. 3-6).

F. 0. 3 Determination of pressure from rock surrounding deep twin tunnels with narrow pillar

1 Vertical pressure

Vertical pressure consists of basic loosening pressure q_1 and additional loosening pressure q_2, q'_2 (Fig. F. 0. 3-1).

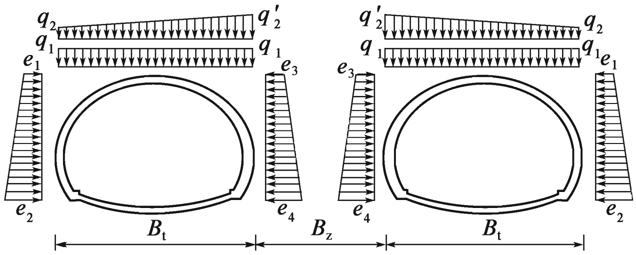


Fig. F.0.3-1 Distribution of load on twin tunnels with narrow pillar

Basic loosening pressure q_1: surrounding pressure from lower stable arch above one tube, assumed to be uniformly distributed (kPa).

Additional loosening pressure q_2、q'_2: loosening pressure of lower surrounding rock of the ultimate balanced arch formed by the left and right tunnels less the basic loosening pressure and the pressure above the central rock pillar and borne by it, assumed to be trapezoid distributed load (kPa).

Vertical pressure on twin tunnels with narrow pillar is calculated from Eq. (F.0.3-1) and (F.0.3.2).

Outer side:

$$q_{outer} = q_1 + q_2 = \gamma(h_{q1} + h_{q2}) \tag{F.0.3-1}$$

Inner side:

$$q_{inner} = q_1 + q'_2 = \gamma(h_{q1} + h'_{q2}) \tag{F.0.3-2}$$

The balance arch above twin tunnels with narrow pillar is typically somewhere between the following two ultimate states:

State 1: Improper excavation method or inappropriate reinforcement measures for the rock pillar results in a lower bearing capacity of the rock pillar and expanding the extent of the balance arch over left and right openings until a common balance arch is formed above them. In this case the role of the rock pillar is neglected and excavation width of the whole twin tunnels with small clearance is taken as collapse arch curve of unsupported tunnel span in the worst case scenario. The height of collapse arch is:

$$h_1^w = 0.45 \times 2^{S-1} \times [1 + i(2B_t + B_{np} - 5)] \tag{F.0.3-3}$$

State 2: The reinforced rock pillar forms a column with a high bearing capacity that prevents settlement of loosened rock/soil masses above and reduces the extent of the balance arch. A stable balance arch is formed above each tunnel and the two balance arches do not affect each other. Take the collapse arch curve for structural calculation of each opening of twin tunnels with small clearance as the most ideal scenario. The height of collapse arch is:

$$h_{q1} = 0.45 \times 2^{S-1}[1 + i(B_t - 5)] \qquad (F.0.3\text{-}4)$$

Vertical rock load on twin tunnels with narrow pillar is given by:

$$q_1 = \gamma h_{q1} = 0.45 \times 2^{S-1}\gamma[1 + i(B_t - 5)] \qquad (F.0.3\text{-}5)$$

$$q'_2 = \gamma h'_{q2} = \gamma\left[\frac{4}{3}(h_1^w - h_{q1}) - \frac{P_z}{\gamma B_m}\right]\frac{B_{wp} + B_t}{B_m} \qquad (F.0.3\text{-}6)$$

$$q_2 = \gamma h_{q2} = \gamma\left[\frac{4}{3}(h_1^w - h_{q1}) - \frac{P_z}{\gamma B_m}\right]\frac{B_{wp}}{B_m} \qquad (F.0.3\text{-}7)$$

Note: If $q_2 < 0$, then take $q_2 = 0$; if $q'_2 < 0$, take $q'_2 = 0$.

where,

i—rock load change rate for every 1m increase/decrease in excavation width, which may be obtained from Table 6.2.2-1; take 0.12 if the width is greater than 14m.

B_{wp}—horizontal projection length of outer side fracture plane (m), which may be calculated from Eq. (F.0.3-8):

$$B_{wp} = (H_t - H_w)\tan\left(45° - \frac{1}{2}\varphi_c\right) \qquad (F.0.3\text{-}8)$$

B_{np}—horizontal projection length of inner side fracture plane (m);

$$B_{np} = \min\left[\frac{1}{2}B_z, (H_t - H_n)\tan\left(45° - \frac{\varphi_c}{2}\right)\right] \qquad (F.0.3\text{-}9)$$

H_t—tunnel excavation height (m).

H_w—the height of the intersection point of outer side fracture plane and excavation line (m);

H_n—the height of start of inner side fracture plane in side wall (m);

γ—unit weight of surrounding rock (kN/m^3);

φ_c—calculated friction angle of rock mass (°);

B_t—excavation width of one tube (m).

B_m—width likely to collapse in one tube of a twin tunnels with small clearance, calculated from Eq. (F.0.3-10):

$$B_m = B_t + B_{wp} + B_{np} \qquad (F.0.3\text{-}10)$$

P_z—support force of the rock pillar to upper rock mass.

The meaning of symbols is presented in Fig. F.0.3-2 and F.0.3-3.

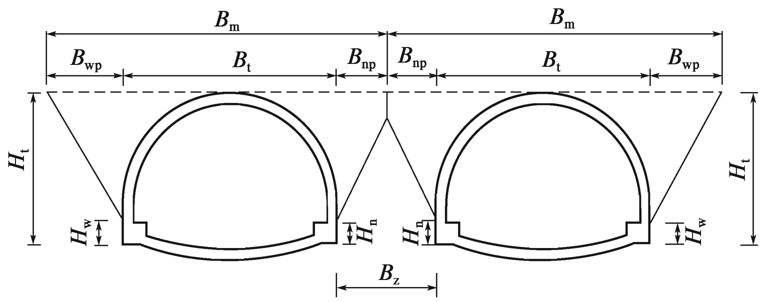


Fig. F. 0. 3-2 Schematic diagram of load calculation for twin tunnels with small clearance ($B_{zp} = 0$)

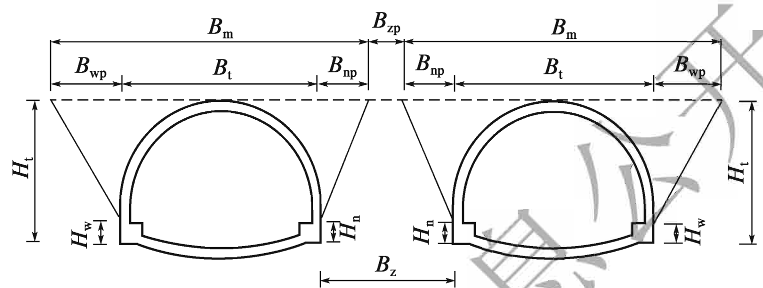


Fig. F. 0. 3-3 Schematic diagram of load calculation for twin tunnels with small clearance ($B_{zp} > 0$)

For the rock pillar of twin tunnels with small clearance, consideration shall be given to increase in rock mass compressive strength due to active support force from tunnel support structures (such as prestressed cross-tie anchor). According to Mohr—Coulomb strength theory, its converted strength may be given by Eq. (F. 0. 3-11).

$$R_S^T = P_i \frac{1 + \sin\varphi}{1 - \sin\varphi} + R_S^b \tag{F. 0. 3-11}$$

where,

R_S^T—converted strength of rock mass in the rock pillar (kPa);

R_S^b—uniaxial compressive design strength of rock mass in the rock pillar (kPa);

P_i—active resisting force of support structure to the rock pillar (kPa);

φ—friction angle in rock mass of the rock pillar (°).

Therefore, the support force of the rock pillar to upper rock may be calculated from Eq. (F. 0. 3-12):

$$P_z = \frac{R_S^T B_{zp}}{K_z} \tag{F. 0. 3-12}$$

where,

K_z—safety factor of support capacity of the rock pillar, generally $K_z = 2$;

B_{zp}—effective bearing width of the rock pillar (m), calculated from Eq. (F. 0. 3-13)

$$B_{zp} = B_z - 2B_{np} \tag{F. 0. 3-13}$$

2 Horizontal lateral pressure

When surrounding rock is Classes I-III:

Outer side:

$$e^{i}_{1\sim2} = \lambda(q_1 + q_2) \tag{F.0.3-14}$$

Inner side:

$$e^{i}_{3\sim4} = \lambda(q_1 + q'_2) \tag{F.0.3-15}$$

When surrounding rock is Classes IV-VI:

Outer side:

$$e^{i}_{1\sim2} = \lambda(q_1 + q_2 + \gamma h_i) \tag{F.0.3-16}$$

Inner side:

$$e^{i}_{3\sim4} = \lambda(q_1 + q'_2 + \gamma h_i) \tag{F.0.3-17}$$

where,

$e^{i}_{1\sim2}$—horizontal pressure on the outside of arch and any point of side walls from surrounding rock (kPa);

$e^{i}_{3\sim4}$—horizontal pressure on the inside of arch and any point of side walls from surrounding rock (kPa);

h_i—the distance from calculation point to crown (m);

λ—lateral pressure coefficient.

F.0.4 The pressure of rock surrounding shallow twin tunnels with narrow pillar shall be determined as follows:

1 When the twin tunnels with small clearance is in one of the following two states, uniformly distributed vertical pressure and lateral rock load acting on the tunnel are calculated by the same method as single- tunnel:

1) Tunnel depth is less than h_q;

2) Tunnel depth is greater than h_q, and less than or equal to H_p, but fracture plane intersection point is at or above ground surface.

2 When the depth of twin tunnels with small clearance (H) is greater than h_q and less than or equal to H_p, ground surface is nearly horizontal and fracture plane intersection point is below ground surface:

1) Vertical pressure

Outer side:

$$q_1 = \gamma H\left(1 - \frac{\lambda_1 H\tan\theta}{B_t}\right) \tag{F.0.4-1}$$

Inner side:

$$q_2 = \gamma H\left(1 - \frac{\lambda_2 H\tan\theta}{B_t}\right) \tag{F.0.4-2}$$

$$\lambda_1 = \frac{\tan\beta - \tan\varphi_c}{\tan\beta[1 + \tan\beta(\tan\varphi_c - \tan\theta) + \tan\varphi_c\tan\theta]} \tag{F.0.4-3}$$

$$\lambda_2 = \frac{B_z}{2}\frac{(2H - 0.5B_z\tan\beta)\sin(\beta - \varphi_c)\cos\theta}{H^2\cos(\theta + \beta - \varphi_c)} \tag{F.0.4-4}$$

$$\tan\beta = \tan\varphi_c + \sqrt{\frac{(\tan^2\varphi_c + 1)\tan\varphi_c}{\tan\varphi_c - \tan\theta}} \tag{F.0.4-5}$$

2) Horizontal pressure

When surrounding rock is Classes I-III:

Outer side:

$$e_{1i} = \lambda_1 q_1 \tag{F.0.4-6}$$

Inner side:

$$e_{2i} = \lambda_2 q_2 \tag{F.0.4-7}$$

When surrounding rock is Classes IV-VI:

Outer side:

$$e_{1i} = \lambda_1(q_1 + \gamma h_i) \tag{F.0.4-8}$$

Inner side:

$$e_{2i} = \lambda_2(q_2 + \gamma h_i) \tag{F.0.4-9}$$

where,

B_t—tunnel excavation width (m);

h_i—vertical distance from calculation point to crown (m);

θ—friction angle on both sides of roof slab soil column (°), which may be obtained from Table E.0.2 if no data are available;

λ_1, λ_2—outside and lateral pressure coefficient, obtained from Eq. (F.0.4-3) and (F.0.4-4);

β—fracture angle at maximum thrust on sides (°);

φ_c—calculated friction angle of surrounding rock (°).

The meaning of symbols is shown in Fig. F.0.4.

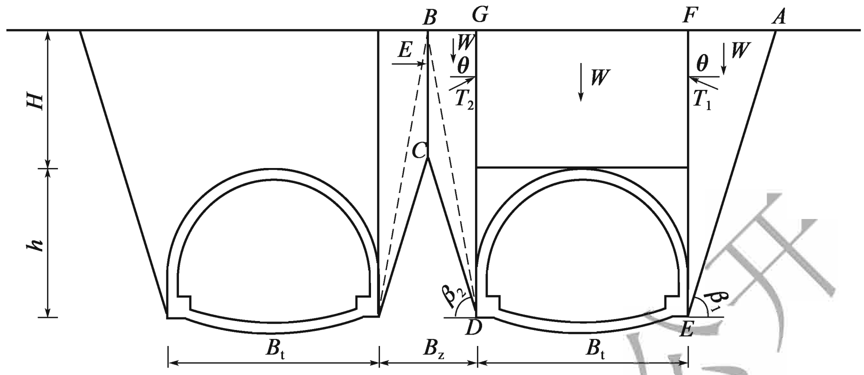


Fig. F.0.4 Schematic diagram of assumed sliding surface

F.0.5 The rock load of eccentric load twin tunnels with narrow pillar shall be determined as follows:

1 If ground transverse slope is skew, tunnel depth is less than h_q or it is greater than h_q and less than or equal to H_p and fracture plane intersection point is at or above ground surface, the vertical pressure and horizontal pressure on both sides are consistent with single-tube tunnel.

2 If ground transverse slope is skew, tunnel depth H is greater than h_q and less than or equal to H_p and fracture plane intersection point is below ground surface:

1) Vertical pressure

$$q_i = \gamma h_i - \frac{\gamma(\lambda_1 h_1{}^2 + \lambda_2 h_2{}^2)\tan\theta}{2B_t} \quad (i = 1,2) \qquad \text{(F.0.5-1)}$$

$$q_i = \gamma h_i - \frac{\gamma(\lambda_3 h_3{}^2 + \lambda_4 h_4{}^2)\tan\theta}{2B_t} \quad (i = 3,4) \qquad \text{(F.0.5-2)}$$

where

$$\lambda_1 = \frac{1}{\tan\beta_1 + \tan\alpha} \times \frac{\tan\beta_1 - \tan\varphi_c}{1 + \tan\beta_1(\tan\varphi_c - \tan\theta) + \tan\varphi_c\tan\theta} \qquad \text{(F.0.5-3)}$$

$$\lambda_2 = \frac{1}{\tan\beta_2 - \tan\alpha} \times \frac{\tan\beta_2 - \tan\varphi_c}{1 + \tan\beta_2(\tan\varphi_c - \tan\theta) + \tan\varphi_c\tan\theta} \qquad \text{(F.0.5-4)}$$

$$\lambda_3 = \frac{1}{\tan\beta_3 + \tan\alpha} \times \frac{\tan\beta_3 - \tan\varphi_c}{1 + \tan\beta_3(\tan\varphi_c - \tan\theta) + \tan\varphi_c\tan\theta} \qquad \text{(F.0.5-5)}$$

$$\lambda_4 = \frac{1}{\tan\beta_4 - \tan\alpha} \times \frac{\tan\beta_4 - \tan\varphi_c}{1 + \tan\beta_4(\tan\varphi_c - \tan\theta) + \tan\varphi_c \tan\theta} \qquad (F.0.5\text{-}6)$$

$$\tan\beta_1 = \tan\varphi_c + \sqrt{\frac{(\tan^2\varphi_c + 1)(\tan\varphi_c + \tan\alpha)}{\tan\varphi_c - \tan\theta}} \qquad (F.0.5\text{-}7)$$

$$\tan\beta_4 = \tan\varphi_c + \sqrt{\frac{(\tan^2\varphi_c + 1)(\tan\varphi_c - \tan\alpha)}{\tan\varphi_c - \tan\theta}} \qquad (F.0.5\text{-}8)$$

$$\tan\beta_2 = \tan\beta_3 = \frac{h_2' + h_3'}{2B_z} \qquad (F.0.5\text{-}9)$$

where,

q_1, q_2, q_3, q_4—vertical pressures on the left side of left tube, on the right side of left tube, on the left side of right tube and on the right side of right tube;

$\lambda_1, \lambda_2, \lambda_3, \lambda_4$—soil pressure coefficients on the left side of left tube, on the right side of left tube, on the left side of right tube and on the right side of right tube;

$\beta_1, \beta_2, \beta_3, \beta_4$—angle between fracture plane and horizontal plane on left side of left tube, on the right side of left tube, on the left side of right tube and on the right side of right tube;

θ—friction angle on both sides of the rock and soil column (°), which may be obtained from Table D.0.2 if no data are available;

α—angle of inclination of ground transverse slope.

The meaning of symbols is shown in Fig. F.0.5.

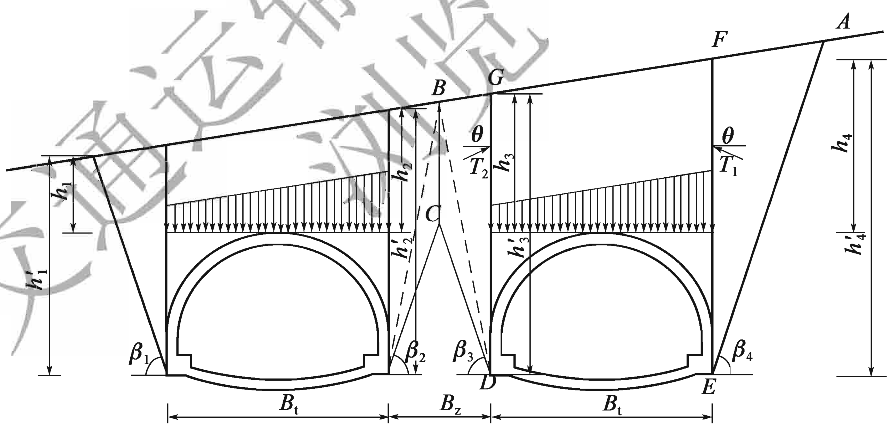


Fig. F.0.5 Sketch for load calculation for eccentric load twin tunnels with small clearance

2) Horizontal pressure

$$e_i = \lambda_i \gamma h_i \qquad (i = 1, 2, 3, 4) \qquad (F.0.5\text{-}10)$$

$$e_i' = \lambda_i \gamma h_i' \qquad (i = 1, 2, 3, 4) \qquad (F.0.5\text{-}11)$$

where,

e_i—horizontal pressure from top rock on the left side of left tube, on the right side of left tube, on the left side of right tube and on the right side of right tube;

e'_i—horizontal pressure from bottom rock on the left side of left tube, on the right side of left tube, on the left side of right tube and on the right side of right tube;

h_i、h'_i—as shown in Fig. F.0.5;

λ_i—see Eq. (F.0.5-3) ~ (F.0.5-6).

F.0.6 The pressure from rock surrounding twin tunnels with small clearance is related to numerous factors including tunnel cross-section shape, dimensions, surrounding rock class, tunnel depth, rock pillar thickness, excavation method, support type and parameter selection. In particular, tunnel excavation method and reinforcing measures and effects for the rock pillar have a big impact on the formation of balance arch and rock load.

Appendix G
Calculation Method for Rock Load for Twin-arch Tunnels

G.0.1 Shallow twin-arch tunnels should undergo internal force calculation, analysis and strength check by load - structure method. For deep twin-arch tunnels, only lining structure in Class III - VI surrounding rocks requires internal force calculation, analysis and strength check; load - structure method should be used. For twin-arch tunnels, simulation, analysis and calculation of construction process may be considered using ground-structure method.

G.0.2 To determine a twin-arch tunnel is shallow or deep, assessment may be made according to equivalent height for loads, geological conditions and construction methods, using Eq. (G.0.2-1) and (G.0.2-2):

$$H_p = (2 \sim 2.5) h_q \tag{G.0.2-1}$$

$$h_q = \frac{q_0}{\gamma} \tag{G.0.2-2}$$

where,

H_p—twin-arch tunnel boundary depth (m); take $H_p = 2.5h_q$ for Class Ⅳ ~ Ⅵ surrounding rock; take $H_p = 2h_q$ for Class I-III surrounding rock.

h_q—equivalent height for loads (m);

γ—unit weight of surrounding rock (kN/m^3).

$q_0 = q + q'$, calculated from Eq. (G.0.3-3) and (G.0.3-4).

G.0.3 The pressure of rock surrounding deep twin-arch tunnels shall be determined as follows:

1 Composition of pressure from rock surrounding deep twin-arch tunnels (Fig. G.0.3-1):

1) Basic vertical pressure from surrounding rock (q): pressure from surrounding rock in lower part of stable balance arch above one tunnel; uniformly distributed load.

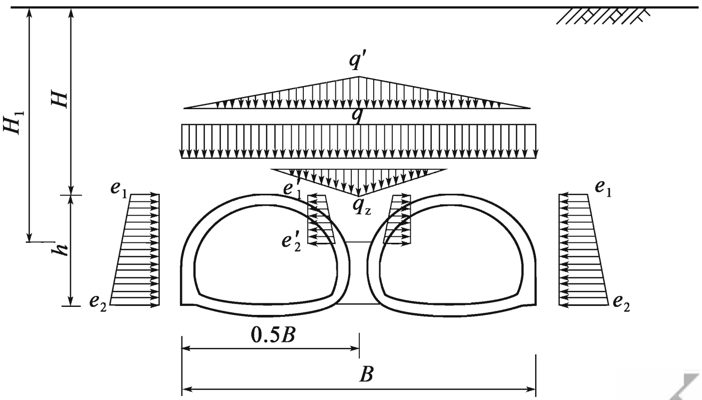


Fig. G.0.3-1 Distribution of load on a deep twin-arch tunnel

2) Additional vertical pressure from surrounding rock (q'): pressure from loosened rock in the lower part of limit balance arch above left and right tunnels less the basic pressure from loosened rock mass, then multiplied by a certain correction coefficient dependent on the timing and degree of compactness of backfill on top of rock pillar.

3) Vertical pressure from rock on top of rock pillar (q_z): distributed load from loosened rock between crowns of both tunnels and the top of rock pillar.

2 Vertical pressure on deep twin-arch tunnels:

Theoretically, if the rock pillar is supported before deformation of top rock occurs and in tight contact with the ground, then half width of the twin-arch tunnel shall be selected for calculation of loosening pressure on the tunnel. Practically, however, deformation of the top rock has occurred when the rock pillar is supported and perfect contact between the rock pillar and surrounding rock is impossible. Therefore, a value between half width of the structure and entire excavation width shall be taken for calculation of the loosening pressure. Fig. G.0.3-2 gives collapse arch curves in various cases.

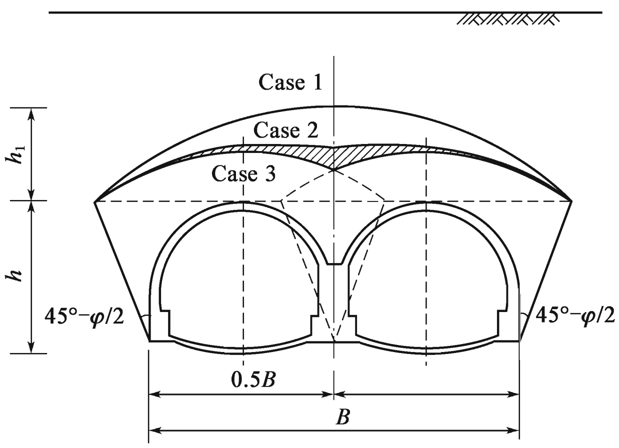


Fig. G.0.3-2 Collapse arch curves for twin-arch tunnel

Case 1: the support role of the rock pillar is neglected and take excavation width of the whole twin-arch tunnel as collapse arch curve of unsupported tunnel span in the worst case scenario. The height of collapse arch is:

$$h_1^w = 0.45 \times 2^{S-1}[1 + i_2(B - 5)] \tag{G. 0. 3-1}$$

Case 2: Take the collapse arch curve for structural calculation of half span of twin-arch tunnel as the most ideal scenario. The height of collapse arch is:

$$h_1^h = 0.45 \times 2^{S-1}[1 + i_1(0.5B - 5)] \tag{G. 0. 3-2}$$

Case 3: The assumed pressure arch curve for twin-arch tunnel is related to the timing and degree of compactness of backfill on top of the rock pillar. Vertical pressure on deep twin-arch tunnels:

$$q = \gamma h_1^h = 0.45 \times 2^{S-1}\gamma\,[1 + i_1(0.5B - 5)] \tag{G. 0. 3-3}$$

$$q' = \xi\gamma(h_1^w - h_1^h) = \xi\gamma \times 0.45 \times 2^{S-1} \times [B(i_2 - 0.5i_1) - 5(i_2 - i_1)] \tag{G. 0. 3-4}$$

$$q_z = \gamma(H_1 - H) \tag{G. 0. 3-5}$$

where,

q—basic vertical uniform pressure on the tunnel, i. e. pressure from rock in lower balance arch formed above one tube (kN/m^2);

q'—additional vertical pressure from surrounding rock (kN/m^2);

q_z—self-weight of triangular block between rock pillar and arch shoulders on both sides (kN/m^2);

γ—unit weight of surrounding rock (kN/m^2);

h_1^h, h_1^w—collapse arch height calculated based on 0.5B and B widths respectively (m);

i_1, i_2—rock pressure change rate calculated based on 0.5B and B widths, taken from Table 6. 2. 3-1 and as 0.12 if width is greater than 14m;

B—overall width of twin-arch tunnel (m);

ξ—additional load correction coefficient; $\xi = 0.2 \sim 0.3$ if backfilling of rock pillar top is timely and top rock is in tight contact with top rock, otherwise $\xi = 0.6 \sim 0.7$; generally $0.3 \sim 0.6$.

3 Calculation of lateral pressure on deep twin-arch tunnels:

Lateral pressures acting on the outside of lined arch and side walls, e_1 and e_2:

$$\left.\begin{aligned} e_1 &= \gamma h_1^h \lambda \\ e_2 &= \gamma(h_1^h + h)\lambda \end{aligned}\right\} \tag{G. 0. 3-6}$$

Horizontal rock pressure acting on linings on both sides of the rock pillar:

$$\left.\begin{aligned} e_1' &= \lambda(q + q') \\ e_2' &= \lambda(q + q' + q_z) \end{aligned}\right\} \qquad (\text{G. 0. 3-7})$$

where,

q, q'—as shown in Fig. G. 0. 3-1;

λ—horizontal lateral pressure coefficient.

G. 0. 4 The pressure of rock surrounding shallow twin-arch tunnels shall be determined as follows:

1 Pressure from rock surrounding extremely shallow twin-arch tunnel:

When its depth H is less than or equal to equivalent height for loads h_q, the twin-arch tunnel is called extremely shallow twin-arch tunnel. Its vertical pressure is calculated from:

$$q = \lambda H \qquad (\text{G. 0. 4-1})$$

$$q_z = \gamma(H_1 - H) \qquad (\text{G. 0. 4-2})$$

where,

q—vertical uniform pressure on the tunnel (kN/m^2);

q_z—maximum load from triangular block between rock pillar and arch shoulders on both sides (kN/m^2);

γ—unit weight of surrounding rock (kN/m^2);

H_1—the distance from rock pillar top to ground surface (m);

H—tunnel depth, i. e. the distance from arc crown to ground surface (m).

Lateral pressure is calculated from:

$$\left.\begin{aligned} e_1 &= \gamma H \lambda \\ e_2 &= \gamma(H + h)\lambda \end{aligned}\right\} \qquad (\text{G. 0. 4-3})$$

where,

e_1, e_2—lateral pressure on arc crown and bottom;

γ—unit weight of surrounding rock (kN/m^2);

H—tunnel depth, i. e. the distance from arc crown to ground surface (m);

h—tunnel excavation height (m).

Horizontal rock pressure acting on linings on both sides of the rock pillar:

$$\left.\begin{aligned} e_1' &= q\lambda \\ e_2' &= (q + q_z)\lambda \end{aligned}\right\} \qquad (\text{G. 0. 4-4})$$

where,

e'_1、e'_2、q、q_z —as shown in Fig. G. 0. 4-1;

λ—horizontal lateral pressure coefficient; refer to Appendix D Calculation Method for Loads on Shallow Tunnels.

The meaning of symbols is shown in Fig. G.0.4-1.

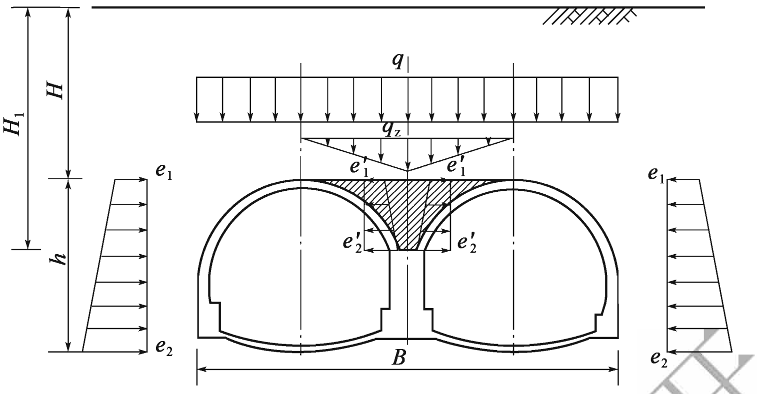


Fig. G.0.4-1 Distribution of loads on extremely shallow twin-arch tunnel

2 Pressure from rock surrounding shallow twin-arch tunnel (Fig. G.0.4-2):

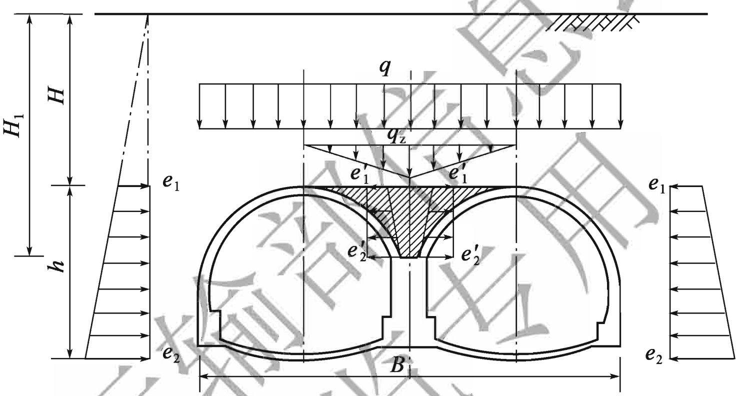


Fig. G.0.4-2 Distribution of loads on general shallow twin-arch tunnel

When its depth is greater than h_q and less than or equal to H_p, it is called general shallow twin-arch tunnel. Its vertical pressure is calculated from:

$$q = \gamma H\left(1 - \frac{H}{B'}\lambda \tan\theta\right) \tag{G.0.4-5}$$

where,

q—vertical pressure on the tunnel (kN/m^2);

γ—unit weight of overlying rock above the tunnel (kN/m^2);

H—tunnel depth, i.e. the distance from arc crown to ground surface (m).

B'—effective width of twin-arch tunnel (m);

θ—friction angle of sliding surface (°), determined per Table D.0.2.

Where arch settlement and convergence are small during center heading excavation, backfilling of rock pillar top is timely and top rock is in tight contact with the rock pillar, B' may be taken as (0.6 ~ 0.7) B if partial-drift excavation is carried out. Where arch settlement and convergence are large, backfilling of rock pillar top is timely and top rock is in loose contact with the rock pillar, B' may be taken as (0.7 ~ 1.0) B if top heading and bench excavation or full face excavation is carried out.

Maximum load from triangular block between rock pillar and arch shoulders on both sides:

$$q_z = \gamma(H_1 - H) \tag{G.0.4-6}$$

Horizontal pressure from rock on both sides of the tunnel:

$$\left.\begin{aligned} e_1 &= q\lambda \\ e_2 &= (q + \gamma h)\lambda \end{aligned}\right\} \tag{G.0.4-7}$$

where,

h—tunnel excavation height (m).

Horizontal pressure on the lining from rock on both sides of the rock pillar:

$$\left.\begin{aligned} e_1' &= \lambda q \\ e_2' &= \lambda(q + q_z) \end{aligned}\right\} \tag{G.0.4-8}$$

where,

λ—lateral pressure coefficient; refer to Appendix D Calculation Method for Loads on Shallow Tunnels.

G.0.5 The pressure of rock surrounding eccentric load twin-arch tunnels shall be determined as follows:

1 Vertical pressure:

Assuming the eccentric load distribution diagram is consistent with ground slope, total vertical pressure applied is:

$$Q = \frac{\gamma}{2}[(h + h')B' - (\lambda h^2 + \lambda' h'^2)\tan\theta] \tag{G.0.5-1}$$

$$q_z = \gamma(H_1 - H) \tag{G.0.5-2}$$

where,

h, h'—height from inner and outer crown level to ground surface (m);

q_z—maximum load from triangular block between rock pillar and arch shoulders on both sides (kN/m^2);

B'—effective width of twin-arch tunnel (m);

γ—unit weight of overlying rock above the tunnel (kN/m^2);

θ—friction angle on both sides of roof slab soil column (°), which may be obtained from Table D.0.2;

λ、λ'—lateral pressure coefficient on inner and outer sides, the same as single-tube eccentric load tunnel.

2 Horizontal lateral pressure on eccentric load tunnel:

Inner side:

$$e_i = \gamma h_i \lambda \qquad \text{(G. 0. 5-3)}$$

Outer side:

$$e'_i = \gamma h'_i \lambda' \qquad \text{(G. 0. 5-4)}$$

where,

h_i、h'_i—the distance from any point i on inner and outer side to ground surface (m);

e_i、e'_i—horizontal lateral pressure on eccentric load tunnel on inner and outer sides (kN/m^2);

γ—unit weight of surrounding rock (kN/m^2);

Horizontal pressure on the lining from rock on both sides of the rock pillar:

$$\left.\begin{aligned} e'_1 &= \lambda\gamma H \\ e'_2 &= \lambda\gamma H_1 \end{aligned}\right\} \qquad \text{(G. 0. 5-5)}$$

$$\left.\begin{aligned} e'_3 &= \lambda'\gamma H \\ e'_4 &= \lambda'\gamma H_1 \end{aligned}\right\} \qquad \text{(G. 0. 5-6)}$$

where,

λ、λ′—lateral pressure coefficient on inner and outer sides, the same as single-tube eccentric load tunnel;

H—the distance from crown horizontal line in the center of rock pillar of twin-arch tunnel to ground surface, as shown in Fig. G. 0. 5;

H_1—the distance from the top of rock pillar of twin-arch tunnel to ground surface, as shown in Fig. G. 0. 5.

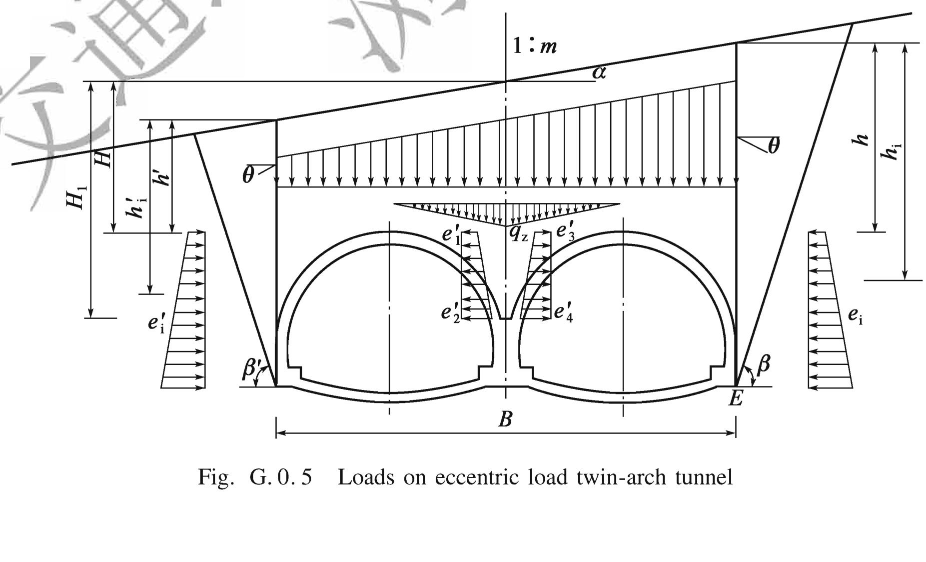


Fig. G. 0. 5 Loads on eccentric load twin-arch tunnel

G. 0. 6 When calculating the rock loads on the support structure of twin-arch tunnel using the above equations, it must be assured that center heading is used, the rock pillar top is backfilled and compacted and rock on top of the rock pillar is stable.

Appendix H
Calculation Method for Backfill Load on Cut-and-cover Tunnel

H.0.1 Vertical pressure from soil backfill on arch ring may be calculated from Eq. (H.0.1):

$$q_i = \gamma_1 h_i \tag{H.0.1-1}$$

where:

q_i—vertical pressure from soil backfill at any point i on cut-and-cover tunnel structure (kN/m^2);

γ_1—unit weight of backfill on extrados (kN/m^3);

h_i—height of soil column at any point i on cut-and-cover tunnel structure (m).

H.0.2 Lateral pressure from backfill on arch ring may be calculated from Eq. (H.0.2-1):

$$e_i = \gamma_1 h_i \lambda \tag{H.0.2-1}$$

where:

e_i—lateral pressure value at any point i (kN/m^2);

γ_1, h_i—the same as previously described;

λ—lateral pressure coefficient.

Lateral pressure coefficient may be calculated in the following two cases:

1) When fill slope inclines upward (Fig. H.0.2-1), it is calculated per unconfined soil mass:

$$\lambda = \cos\alpha \frac{\cos\alpha - \sqrt{\cos^2\alpha - \cos^2\varphi_1}}{\cos\alpha + \sqrt{\cos^2\alpha - \cos^2\varphi_1}} \tag{H. 0.2-2}$$

where:

α—design fill slope angle (°);

φ_1—calculated friction angle of backfill on extrados (°).

2) When fill slope inclines downward (Fig. H.0.2-2), it is calculated per confined soil

mass:

$$\lambda = \frac{1-\mu n}{(\mu+n)\cos\rho + (1-\mu n)\sin\rho} \cdot \frac{mn}{m-n} \qquad (\mathrm{H.0.2\text{-}3})$$

where:

ρ—angle between lateral pressure action direction and horizontal line (°);

n—Ratio of side slopes excavated;

m—Ratio of backfilled slope surface;

μ—frictional coefficient between backfill material and side slopes excavated surface.

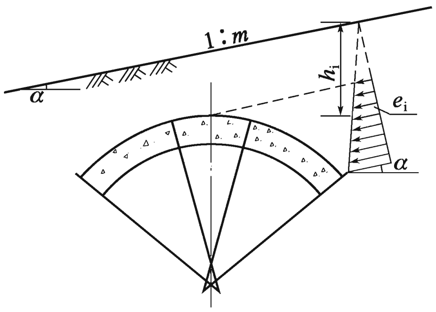


Fig. H.0.2-1 Fill slope inclining upward

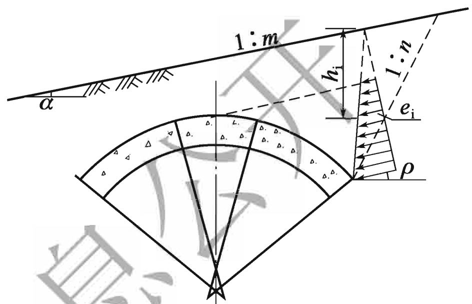


Fig. H.0.2-2 Fill slope inclining downward

H.0.3 Lateral pressure from backfill behind side walls may be calculated from Eq. (H.0.3-1):

$$e_i = \gamma_2 h_i' \lambda \qquad (\mathrm{H.0.3\text{-}1})$$

where:

γ_2—unit weight of backfill behind wall (kN/m^3);

h_i'—converted height of calculation point on side wall, $h_i' = h_i'' + \frac{\gamma_1}{\gamma_2} h_1$

h_i''—height from wall top to calculation point (m);

h_1—vertical height from fill slope surface to wall top (m);

λ—lateral pressure coefficient.

Lateral pressure coefficient may be calculated in the following three cases:

1) When fill slope inclines upward (Fig. H.0.3-1):

$$\lambda = \frac{\cos^2\varphi_2}{\left[1 + \sqrt{\frac{\sin\varphi_2 \sin(\varphi_2 - \alpha')}{\cos\alpha'}}\right]^2} \qquad (\mathrm{H.0.3\text{-}2})$$

2) When fill slope inclines downward (Fig. H.0.3-2):

$$\lambda = \frac{\tan\theta_0}{\tan(\theta_0 + \varphi_2)(1 + \tan\alpha' \tan\theta_0)} \qquad (\mathrm{H.0.3\text{-}3})$$

where:

φ_2—calculated friction angle of backfill material behind wall (°).

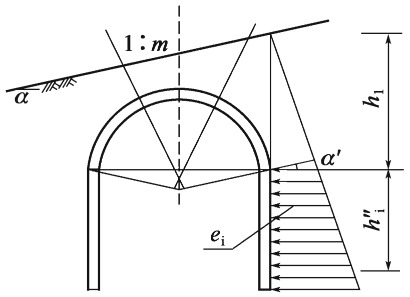


Fig. H.0.3-1 Fill slope inclining upward

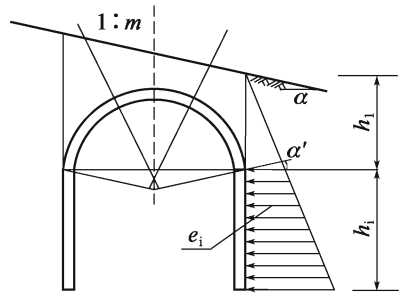


Fig. H.0.3-2 Fill slope inclining downward

$$\alpha' = \arctan\left(\frac{\gamma_1}{\gamma_2}\tan\alpha\right) \tag{H.0.3-4}$$

$$\tan\theta_0 = \frac{-\tan\varphi_2 + \sqrt{(1+\tan^2\varphi_2)(1+\tan\alpha'/\tan\varphi_2)}}{1+(1+\tan^2\varphi_2)\tan\alpha'/\tan\varphi_2} \tag{H.0.3-5}$$

3) When fill slope is horizontal:

$$\lambda = \tan^2\left(\frac{\pi}{4} - \frac{\varphi_2}{2}\right) \tag{H.0.3-6}$$

Appendix J
Calculation Method for Soil pressure on Portal Wall

J.0.1 The pressure on end wall, wing wall and retaining wall of tunnel portals may be calculated as follows:

1 The angle between the most dangerous fracture plane and vertical plane:

$$\tan\omega = \frac{\tan^2\varphi_c + \tan\alpha\tan\varepsilon - \sqrt{(1+\tan^2\varphi_c)(\tan\varphi_c - \tan\varepsilon)(\tan\varphi_c + \tan\alpha)(1-\tan\alpha\tan\varepsilon)}}{\tan\varepsilon(1+\tan^2\varphi_c) - \tan\varphi_c(1-\tan\alpha\tan\varepsilon)}$$

(J.0.1-1)

where:

φ_c—Calculated friction angle of surrounding rock (°);

ε, α—ground slope angle and wall face dip angle (°), as shown in Fig. J.0.1.

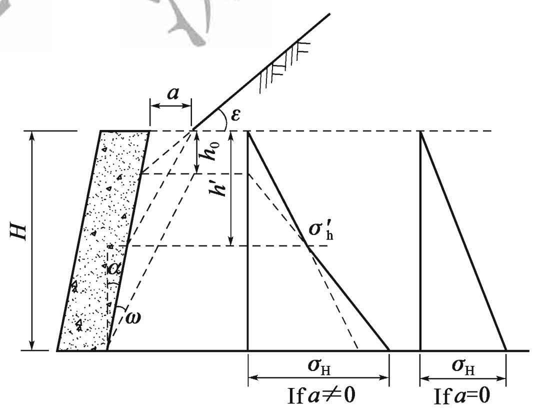


Fig. J.0.1 Schematic diagram of ground slope angle and wall face dip angle

2 Soil pressure:

$$E = \frac{1}{2}\gamma\lambda \cdot [H^2 + h_0(h' - h_0)] \cdot b \cdot \xi \qquad \text{(J.0.1-2)}$$

$$\lambda = \frac{(\mathrm{tg}\omega - \mathrm{tg}\alpha)(1 - \mathrm{tg}\alpha \cdot \mathrm{tg}\varepsilon)}{\mathrm{tg}(\omega + \varphi_c)(1 - \mathrm{tg}\omega \cdot \mathrm{tg}\varepsilon)} \tag{J. 0. 1-3}$$

$$h' = \frac{\alpha}{\mathrm{tg}\omega - \mathrm{tg}\alpha} \tag{J. 0. 1-4}$$

where:

E—soil pressure (kN);

γ —unit weight of strata (kN/m^3);

λ—lateral pressure coefficient;

ω —fracture angle of soil behind wall (°);

b—calculated strip width of portal wall (m);

ξ—uncertainty coefficient for soil pressure calculation mode, which may be taken as $\xi = 0.6$.

Appendix K
Calculation Method of Seismic Load

K.0.1 When using static method for calculation, seismic loads applied on the structure consist mainly of earthquake inertia from lining self-weight, earthquake inertia from overlying soil column and increment in earthquake-induced lateral soil pressure. Tunnel seismic response in cross-section direction may be calculated based on the sketch given in Fig. K.0.1.

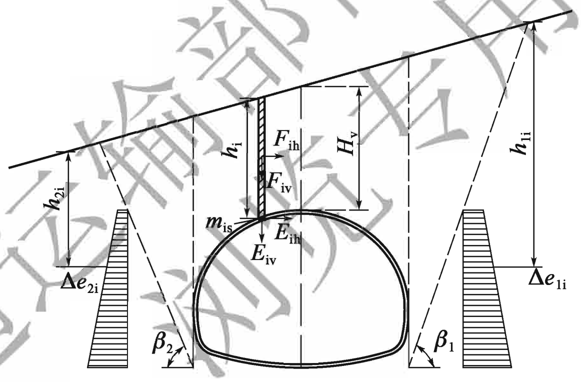


Fig. K.0.1 Sketch for calculation by static method

K.0.2 The earthquake inertia from lining self-weight is the inertia of the structure itself caused by earthquake acceleration. Its earthquake effects in horizontal and vertical directions are calculated respectively from Eq. (K.0.2-1) and (K.0.2-2):

$$E_{ih} = A_h m_{is} = C_i C_s A m_{is} \tag{K.0.2-1}$$

$$E_{iv} = K_v A_h m_{is} = K_v C_i C_s A m_{is} \tag{K.0.2-2}$$

where:

C_i—seismic importance coefficient, obtained from Table 16.2.3;

C_s—site adjustment coefficient, which may be obtained from Table 5.2.2 of Specification of Seismic Design for Highway Engineering (JTG B02—2013); take 1.0 if the seismic ground motion parameter adopted is applicable to the tunnel site;

A—horizontal basic seismic peak ground acceleration, obtained from Table 16.2.2 in the

Specifications;

m_{is}—weight at calculation point of tunnel lining (kg);

K_v—ratio of vertical seismic peak ground acceleration to horizontal seismic peak ground acceleration, which may be obtained by conversion from Table 3.3.2 of Specification of Seismic Design for Highway Engineering (JTG B02—2013).

K.0.3 In calculation, assume the earthquake inertia of overlying soil column which acts on the soil unit centroid (Fig. K.0.1), its value is calculated from Eq. (K.0.3-1) and (K.0.3-2). In calculating structural internal force, simplify this earthquake force using force shift theorem into nodal forces and bending moments on the upper lining.

Horizontal earthquake effect of overlying soil column:

$$F_{ih} = A_h Q_i / g \tag{K.0.3-1}$$

Vertical earthquake effect of overlying soil column:

$$F_{iv} = K_v A_h Q_i / g \tag{K.0.3-2}$$

Vertical soil pressure of overlying soil column:

$$Q_i = \frac{\gamma}{2}[2h_i B_i - (\lambda_1 h_i^2 + \lambda_2 h_i^2)\tan\theta_0] \tag{K.0.3-3}$$

where:

A_h—horizontal design seismic peak ground acceleration, calculated from Eq. (K.0.2-1);

g—gravitational acceleration;

γ—unit weight of surrounding rock (kN/m);

h_i—height of overlying soil column; taken as the value of H_v (m) when the height of overlying soil column is greater than maximum calculated height of overlying soil column H_v (see Table K.0.5 in the appendix);

B_i—width of overlying soil column (m);

θ_0—friction angle on both sides of soil column (°);

λ_1, λ_2—Inner and outer lateral pressure coefficient in the event of earthquake, calculated from the following equation:

$$\lambda_1 = \frac{(\tan\beta_1 - \tan\varphi_1)(1 - \tan\theta_1 \tan\theta)}{(\tan\beta_1 - \tan\alpha)[1 + \tan\beta_1(\tan\varphi_1 - \tan\theta_1) + \tan\varphi_1 \tan\theta_1]} \tag{K.0.3-4}$$

$$\lambda_2 = \frac{(\tan\beta_2 - \tan\varphi_2)(1 - \tan\theta_2 \tan\theta)}{(\tan\beta_2 + \tan\alpha)[1 + \tan\beta_2(\tan\varphi_2 - \mathit{tan}\theta_2) + \tan\varphi_2 \tan\theta_2]} \tag{K.0.3-5}$$

$$\tan\beta_1 = \tan\varphi_1 + \sqrt{\frac{(\tan^2\varphi_1 + 1)(\tan\varphi_1 - \tan\alpha)}{(\tan\varphi_1 - \tan\theta_1)}} \tag{K.0.3-6}$$

$$\tan\beta_2 = \tan\varphi_2 + \sqrt{\frac{(\tan^2\varphi_2 + 1)(\tan\varphi_2 + \tan\alpha)}{(\tan\varphi_2 - \tan\theta_2)}} \tag{K.0.3-7}$$

$$\varphi_1 = \varphi_g - \theta \tag{K.0.3-8}$$

$$\varphi_2 = \varphi_g + \theta \tag{K.0.3-9}$$

$$\theta_1 = \theta_0 - \theta \tag{K.0.3-10}$$

$$\theta_1 = \theta_0 + \theta \tag{K.0.3-11}$$

where:

φ_g—Calculated friction angle of surrounding rock (°);

θ—earthquake angle, obtained from Table K.0.3; take $\theta = 0$ if there is no earthquake;

α—ground slope angle (°); take $\alpha = 0$ if the ground surface is level grade;

β_1, β_2—inner and outer fracture angle at maximum thrust (°).

Meanings of other symbols are the same as above.

Table K.0.3 Correspondence between horizontal basic seismic peak ground acceleration and earthquake angle

Level of fortification (degree)	7		8		9
Horizontal basic seismic peak ground acceleration A (g)	0.10	0.15	0.2	0.3	0.4
Earthquakeangle θ	1°30′		3°	4°30′	6°

K.0.4 Increment in lateral soil pressure in the event of earthquake shall be calculated as follows:

1 Inner side soil pressure increment:

$$\Delta e_{1i} = C_i C_s \gamma h_{1i} (\lambda_1 - \lambda) \tag{K.0.4-1}$$

2 Outer side soil pressure increment:

$$\Delta e_{2i} = C_i C_s \gamma h_{2i} (\lambda_2 - \lambda') \tag{K.0.4-2}$$

where:

λ, λ'—normal lateral pressure coefficient on inner and outer sides;

h_{1i}, h_{2i}—distance from any point i on inner and outer sides of the lining to ground surface (m);

Meanings of other symbols are the same as above.

K.0.5 Research shows in seismic calculation by static method, the internal force in lining structure is dependent on surrounding rock conditions, tunnel span, seismic peak ground acceleration and other factors while maximum calculated height of overlying soil column is only dependent on surrounding rock conditions and tunnel span, where surrounding rock class is determined based on the criteria presented in the Specifications and tunnel span is divided into two-lane and three-lane tunnels according to commonly used number of lanes. Obtain maximum calculated height of overlying soil column H_v from Table K.0.5 according to the class of rock surrounding the tunnel and tunnel span. If the height of overlying soil column is less than the corresponding value in Table K.0.5, actual height shall be used in calculation; if the height of overlying soil column is greater than the corresponding value in the table, then the corresponding value in Table K.0.5 shall be used in calculation.

Table K.0.5 Maximum calculated height value (H_v) of overlying soil column at crown by static method

Condition	Depth for seismic calculation	
Span	Two-lane tunnel	Three-lane tunnel
Class I-II surrounding rock	0.5B	0.5B
Class III surrounding rock	1.5B	1.0B
Class IV surrounding rock	2.5B	2.0B
Class V surrounding rock	3.0B	2.5B

K.0.6 Seismic effects on cut-and-cover tunnel and shed tunnel may be calculated as follows; the calculation sketch is shown in Fig. K.0.6.

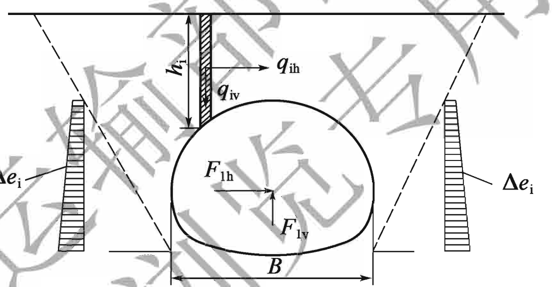


Fig. K.0.6 Sketch for calculation of seismic effects on cut-and-cover tunnel and shed tunnel

1 Seismic force from structural self-weight may be calculated according to K.0.2 herein;

2 Horizontal seismic earth load from backfill at arc crown:

$$q_{ih} = A_h h_i \gamma / g \quad \text{(K.0.6-1)}$$

where:

h_i—backfill thickness at calculation point (m);

γ—unit weight of backfill (kN/m^3);

Meanings of other symbols are the same as above.

3 Vertical seismic earth load from backfill at arc crown:

$$q_{iv} = k_v A_h h_i \gamma / g \quad \text{(K.0.6-2)}$$

4 Increment in lateral pressure from backfill at the sides is calculated from Eq. (K.0.4-1) and (K.0.4-2).

K.0.7 Horizontal seismic additional load from the self weight of portal wall and retaining wall may be calculated from Eq. (K.0.7-1) and (K.0.7-2):

$$E_{i\mathrm{hw}} = C_i A \Psi_{i\mathrm{w}} m_{i\mathrm{w}} \tag{K.0.7-1}$$

$$E_{i\mathrm{vw}} = C_i K_{\mathrm{v}} A \Psi_{i\mathrm{w}} m_{i\mathrm{w}} \tag{K.0.7-2}$$

where:

$E_{i\mathrm{hw}}$—horizontal seismic load at wall gravity center above the i^{th} section (kN);

$E_{i\mathrm{vw}}$—vertical seismic load at wall gravity center above the i^{th} section (kN);

$m_{i\mathrm{w}}$—wall weight above the i^{th} section (kg);

$\psi_{i\mathrm{w}}$—distribution factor of horizontal seismic load along wall height, which may be taken according to Eq. (7.2.3-2) in the Specification of Seismic Design for Highway Engineering (JTG B02—2013).

Meanings of other symbols are the same as above.

K.0.8 Seismic active soil pressure from clayed fill soil may be calculated from Eq. (K.0.8-1):

$$E_{\mathrm{ep}} = \left(\frac{1}{2}\gamma H^2 + qH\frac{\cos\alpha}{\cos(\alpha-\beta)}\right)K_{\mathrm{psp}} + 2cHK_{\mathrm{cp}} \tag{K.0.8-1}$$

where: γ—gravity and density of clayed fill soil (kN/m³);

H—portal wall or retaining wall height (m);

q—uniformly distributed load on sliding wedge (kN/m);

α—the angle between portal wall or retaining wall back and vertical line (°);

β—the angle between fill surface and horizontal plane (°);

c—cohesion factor of clayed fill soil;

K_{a}—coefficient of seismic active soil pressure;

K_{ca}—coefficient, calculated from Eq. (K.0.8-3).

The angles are shown in Fig. K.0.8.

K_{a} in Eq. (K.0.8-1) is calculated from Eq. (K.0.8-2):

$$K_{\mathrm{a}} = \frac{\cos^2(\varphi-\alpha-\theta)}{\cos\theta\cos^2\alpha\cos(\alpha+\delta+\theta)\left[1+\sqrt{\dfrac{\sin(\varphi+\delta)\sin(\varphi-\beta-\theta)}{\cos(\alpha-\beta)\cos(\alpha+\delta+\theta)}}\right]^2} \tag{K.0.8-2}$$

where:

φ—internal friction angle of fill (°);

δ—friction angle between fill and portal wall or retaining wall back (°).

$$K_{\mathrm{ca}} = \frac{1-\sin\varphi}{\cos\varphi} \tag{K.0.8-3}$$

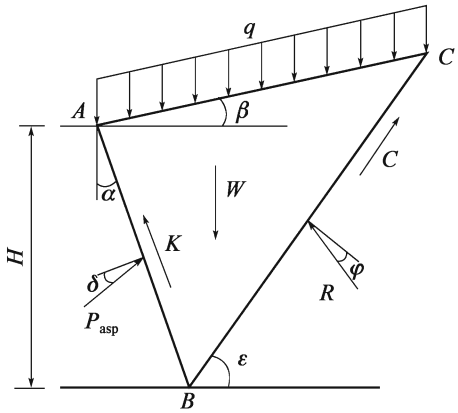


Fig. K.0.8 Schematic diagram for calculation of seismic soil pressure

Earthquake angle θ is obtained from Table K.0.8.

Table K.0.8 Values of earthquake angle

Design earthquake intensity		7	8	9
Earthquake angle θ (°)	Above-water	1.5	3.0	6.0
	Underwater	2.5	5.0	10.0

K.0.9 Seismic passive soil pressure of clayed fill may be calculated from Eq. (K.0.9-1) and (K.0.9-2):

$$E_{ep}=\left(\frac{1}{2}\gamma H^2+qH\frac{\cos\alpha}{\cos(\alpha-\beta)}\right)K_{psp}+2cHK_{cp} \quad (K.0.9\text{-}1)$$

$$K_{cp}=\frac{\sin(\varphi-\theta)+\cos\theta}{\cos\theta\cos\varphi} \quad (K.0.9\text{-}2)$$

where:

K_{psp}—coefficient of seismic passive soil pressure;

K_{cp}—coefficient of passive soil pressure arising from the cohesiveness of soil.

Meanings of other symbols in the equation are the same as above.

K.0.10 When seismic soil pressure acts at $q=0$, the location $H/3$ from wall base may be taken; when $q\neq0$, fill height converted from q shall be added to H.

K.0.11 If the fill behind portal wall or retaining wall is cohesionless, seismic active soil pressure acting on the wall back may also be calculated from the following simplified equation:

$$E_{ea}=\frac{1}{2}\gamma H^2K_A\left(1+\frac{3C_iA}{g}\tan\varphi\right) \quad (K.0.11\text{-}1)$$

$$K_A=\frac{\cos^2\varphi}{(1+\sin\varphi)^2} \quad (K.0.11\text{-}2)$$

where:

E_{ea}—active soil pressure acting on each liner meter length of wall back when earthquake occurs (kN/m), at a distance of 0.4H from wall base;

γ—gravity and density of earth (kN/m^3);

H—wall height (m);

K_A—coefficient of active soil pressure acting on wall back under non-earthquake conditions;

φ—internal friction angle of earth behind wall (°);

C_i—seismic importance coefficient.

If presence of liquefiable soil layer or soft soil layer below ground surface at the wall is established, the active soil pressure acting on wall back shall be calculated from Eq. (K.0.11-3):

$$E_{ea} = \frac{1}{2}\gamma H^2 (K_A + 2C_i A/g) \qquad (K.0.11\text{-}3)$$

Meanings of symbols in the equation are the same as above.

Appendix L
Load Structure Method

L.0.1 Design principle:

The load structure method is based on the design principle that ground action following tunnel excavation is mainly to apply load on lining structure which shall be able to safely and reliably withstand the loading from rock load. First determine rock load by stratum classification method or practical formula; then calculate internal force in the lining by the calculation method for structures on elastic ground and carry out structural section design.

L.0.2 Calculation principle:

1 Basic unknown quantity and basic equation

Take the displacement of lining structural nodes as basic unknowns. Based on the principle of minimum potential energy or variational principle, the equilibrium equation for system overall solution is derived as:

$$[K]\{\delta\} = \{P\} \tag{L.0.2-1}$$

where:

$\{\delta\}$—vector composed of joint displacement of lining structure, i.e. $\{\delta\} = [\delta_1 \delta_2 \cdots \delta_m]^T$;

$\{P\}$—vector composed of load at lining structure nodes, i.e. $\{P\} = [P_1, P_2, \cdots, P_m]^T$;

$[K]$—overall stiffness matrix of the lining structure, as *$m \times m$ square matrix where m is total number of system degree of freedoms of nodes.*

Matrixes $\{P\}$, $[K]$ and $\{\delta\}$ may be assembled of element load matrix $\{P\}^e$, element stiffness matrix $[k]^e$ and element displacement vector matrix $\{\delta\}^e$. Therefore, in analysis by finite element method elements need to be divided to establish element stiffness matrix $[k]^e$ and element load matrix $\{P\}^e$.

If the axis of load-carrying structure of the tunnel is in an arc shape, the arc needs to be simulated by broken line elements. When dividing elements, only the length of the element needs to be determined. The member thickness d is the thickness of the bearing structure; member width is taken as 1 (m). The corresponding cross – sectional area of the member is $A = d \times 1$ (m^2); bending moment of inertia is $I = \frac{1}{12} \times 1 \times d^3$ (m^4); its elastic modulus E ($\mathrm{kN/m^2}$) is taken as the elastic modulus of concrete.

2 Element stiffness matrix calculation

Set node displacement of beam element under local coordinate system as $\{\bar{\delta}\} = [\overline{u_i}, \overline{v_i}, \overline{\theta_i}, \overline{u_j}, \overline{v_j}, \overline{\theta_j}]^T$ and the corresponding node force as $\{\bar{f}\} = [\overline{X_i}, \overline{Y_i}, \overline{M_i}, \overline{X_j}, \overline{Y_j}, \overline{M_j}]^T$, then:

$$\{\bar{f}\} = \{\bar{k}\}^e \{\bar{\delta}\} \tag{L. 0. 2-2}$$

where:

$[\bar{k}]^e$ is stiffness matrix of beam element under local coordinate system, and:

$$[\bar{k}]^e = \begin{bmatrix} \frac{EA}{l} & 0 & 0 & -\frac{EA}{l} & 0 & 0 \\ 0 & \frac{12EI}{l^3} & \frac{6EI}{l^2} & 0 & -\frac{12EI}{l^3} & \frac{6EI}{l^2} \\ 0 & \frac{6EI}{l^2} & \frac{4EI}{l} & 0 & -\frac{6EI}{l^2} & \frac{2EI}{l} \\ -\frac{EA}{l} & 0 & 0 & \frac{EA}{l} & 0 & 0 \\ 0 & -\frac{12EI}{l^3} & -\frac{6EI}{l^2} & 0 & \frac{12EI}{l^3} & -\frac{6EI}{l^2} \\ 0 & \frac{6EI}{l^2} & \frac{2EI}{l} & 0 & -\frac{6EI}{l^2} & \frac{4EI}{l} \end{bmatrix} \tag{L. 0. 2-3}$$

where:

l—beam element length;

A—sectional area of beam;

I—moment of inertia of beam;

E—elastic modulus of beam.

For the overall structure, the local coordinate system used for each element varies. Therefore, when establishing overall matrix, the element stiffness matrix $[\bar{k}]^e$ established under local coordinate system needs to be converted by Eq. (L. 0. 2-4) into element stiffness matrix $[k]^e$ under global coordinate system.

$$[k]^e = [T]^T [\bar{k}]^e [T] \tag{L. 0. 2-4}$$

where:

$[T]$—transposed matrix, expressed as:

$$[T]=\begin{bmatrix} \cos\beta & \sin\beta & 0 & 0 & 0 & 0 \\ -\sin\beta & \cos\beta & 0 & 0 & 0 & 0 \\ 0 & 0 & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & \cos\beta & \sin\beta & 0 \\ 0 & 0 & 0 & -\sin\beta & \cos\beta & 0 \\ 0 & 0 & 0 & 0 & 0 & 1 \end{bmatrix} \tag{L. 0. 2-5}$$

where:

β—the angle between local and global coordinate systems (°).

3 Ground reaction mode

Elastic resisting force of ground is given by:

$$F_{\mathrm{n}}=K_{\mathrm{n}}\cdot U_{\mathrm{n}} \tag{L. 0. 2-6}$$

$$F_{\mathrm{s}}=K_{\mathrm{s}}\cdot U_{\mathrm{s}} \tag{L. 0. 2-7}$$

where

$$K_{\mathrm{n}}=\begin{cases} K_{\mathrm{n}}^{+} & U_{\mathrm{n}}\geqslant 0 \\ K_{\mathrm{n}}^{-} & U_{\mathrm{n}}<0 \end{cases} \tag{L. 0. 2-8}$$

$$K_{\mathrm{s}}=\begin{cases} K_{\mathrm{s}}^{+} & U_{s}\geqslant 0 \\ K_{\mathrm{s}}^{-} & U_{s}<0 \end{cases} \tag{L. 0. 2-9}$$

where:

F_{n}, F_{s}—normal and tangential elastic resisting force.

K_{n} and K_{s} are the corresponding elastic resisting force coefficient of surrounding rock, and K^{+} and K^{-} are resisting force coefficients in compressive and tensile zones respectively; commonly, set $K_{\mathrm{n}}^{-}=K_{\mathrm{s}}^{-}=0$.

After member element is determined, ground spring element can be determined; it is only arranged at the nodes of member elements. Ground spring elements may be arranged along the entire section or at some nodes. Where ground spring elements are arranged along the entire section, deformation control analysis by iteration method is required during calculation to identify the exact location of the zone of resistance.

Appendix M
Ground-Structure Method

M.0.1 Design principle:

The ground-structure method is based on the design principle that treats the lining and ground as an integral system carrying loads and calculates internal forces in the lining and ground respectively while meeting deformation compatibility conditions to check ground stability and develop structural section design.

The current predominant calculation method is finite element method that applies to design of linings in soft rock or stable ground.

M.0.2 Calculation of initial In-Situ Stress:

1 Initial self-weight stress

Initial self-weight stress is frequently calculated by finite element method or by given horizontal lateral pressure coefficient.

1) Finite element method

Initial self-weight stress is calculated by finite element method and converted to equivalent nodal load.

2) Given horizontal lateral pressure coefficient method

With given horizontal lateral pressure coefficient K_0, initial self-weight In-Situ Stress is calculated from the following:

$$\sigma_z^g = \sum \gamma_i H_i \tag{M.0.2-1}$$

$$\sigma_x^g = K_0 \cdot (\sigma_z - P_w) + P_w \tag{M.0.2-2}$$

where:

σ_z^g, σ_x^g—initial self-weight In-Situ Stress in vertical and horizontal directions;

γ_i—unit weight of rock in the ith layer above calculation point;

H_i—thickness of rock in the ith layer above calculation point;

P_w—pore water pressure at calculation point. When groundwater head variations are not considered, P_w is determined by hydrostatic pressure at calculation point, i. e. $P_w = \gamma_w \cdot H_w$ (γ_w is groundwater unit weight and H_w is groundwater table difference).

2 Tectonic stress

Tectonic stress may be assumed to be uniformly distributed or linearly distributed stress. Assuming the direction of principal stress action remains constant, the universal expression of two-dimensional plane strain is:

$$\begin{cases} \sigma_x^s = a_1 + a_4 z \\ \sigma_z^s = a_2 + a_5 z \\ \tau_{xz}^s = a_3 \end{cases} \tag{M. 0. 2-3}$$

where:

$a_1 \sim a_5$—constant coefficient;

z—vertical coordinate.

3 Initial In-Situ Stress

Add initial self-weight stress to tectonic stress to derive initial In-Situ Stress.

M. 0. 3 Constitutive model:

1 Rock mass element

1) Elastic model

For a plain strain problem, stress increment of isotropic elastomers may be expressed as:

$$\{\Delta\sigma\} = \begin{Bmatrix} \Delta\sigma_x \\ \Delta\sigma_z \\ \Delta\tau_{zx} \end{Bmatrix} = [D]\{\Delta\varepsilon\} = \begin{bmatrix} \dfrac{E_0 E_v - \mu_{uh}^2 E_h^2}{E_0} & \dfrac{E_h E_v \mu_{vh}(1+\mu_{hh})}{E_0} & 0 \\ \dfrac{E_h E_v \mu_{vh}(1+\mu_{hh})}{E_0} & \dfrac{E_v^2(1-\mu_{hh}^2)}{E_0} & 0 \\ 0 & 0 & G_{hv} \end{bmatrix} \begin{Bmatrix} \Delta\varepsilon_x \\ \Delta\varepsilon_z \\ \Delta\gamma_{zx} \end{Bmatrix} \tag{M. 0. 3-1}$$

where: E_v—elastic modulus in vertical direction (z);

E_h—elastic modulus in horizontal direction (x, y);

μ_{vh}—Poisson' s ratio of vertical strain to horizontal strain (Poisson' s ratio within vertical plane);

μ_{hh}—Poisson's ratio within horizontal plane;

G_{hv}—shear modulus within vertical plane.

Stress increment of isotropic elastomers may be expressed as:

$$\{\Delta\sigma\} = \begin{Bmatrix} \Delta\sigma_{x} \\ \Delta\sigma_{z} \\ \Delta\tau_{zx} \end{Bmatrix} = [D]\{\Delta\varepsilon\} = \frac{E(1-\mu)}{(1+\mu)(1-2\mu)} \begin{bmatrix} 1 & \dfrac{\mu}{1-\mu} & 0 \\ \dfrac{\mu}{1-\mu} & 1 & 0 \\ 0 & 0 & \dfrac{1-2\mu}{2(1-\mu)} \end{bmatrix} \begin{Bmatrix} \Delta\varepsilon_{x} \\ \Delta\varepsilon_{z} \\ \Delta\gamma_{zx} \end{Bmatrix} \tag{M.0.3-2}$$

2) Non-linear elastic model

Using Duncan-Chang model assumptionsand assuming the stress-strain relationship may be approximated by hyperbola relationship, when principal stress σ3 remains constant:

$$\sigma_1 - \sigma_3 = \frac{\varepsilon_1}{a + b\varepsilon_1} \tag{M.0.3-3}$$

Assume hyperbola relationship also exists between axial strain ε_1 and lateral strain ε_3, then:

$$\varepsilon_1 = \frac{\varepsilon_3}{f + d\varepsilon_3} \tag{M.0.3-4}$$

where:

a, b, f, d—parameters determined by test.

In different stress states, elastic modulus is expressed as:

$$E_i = \left[1 - \frac{R_f(1-\sin\phi)(\sigma_1-\sigma_3)}{2c\cos\phi + 2\sigma_3\sin\phi}\right]^2 K \cdot P_0 \cdot \left(\frac{\sigma_3}{P_0}\right)^n \tag{M.0.3-5}$$

where: R_f—failure ratio, less than 1 (typically 0.75 ~ 1.0);

c, φ—soil cohesion and internal friction angle;

P_0—atmospheric pressure, generally 100kPa;

K, n—parameters determined by test.

In different stress states, Poisson's ratio is expressed as:

$$\mu_i = \frac{G - F\lg\left(\dfrac{\sigma_3}{P_0}\right)}{(1-A)^2} \tag{M.0.3-6}$$

$$A = \frac{(\sigma_1 - \sigma_3)d}{Kp_0\left(\dfrac{\sigma_3}{P_0}\right)^n\left[1 - \dfrac{R_f(1-\sin\varphi)(\sigma_1-\sigma_3)}{2c\cos\varphi + 2\sigma_3\sin\varphi}\right]} \tag{M.0.3-7}$$

where:

G, F, d—parameters determined by test. Elastic matrix [D] can be determined from E_i and μ_i in this stress state.

3) Elastic-plastic model

①Yield criterion

To determine whether the material is in plastic state, Drucker-Prager or Mohr—Coulomb yield criterion is applied. The Drucker-Prager yield criterion is expressed as:

$$f=\alpha \cdot I_1+\sqrt{J_2}-k=0 \tag{M.0.3-8}$$

where:

I_1—the first invariant of stress tensor;

J_2—the second invariant of stress deviator;

$$\alpha=\frac{\sin\phi}{\sqrt{3}\sqrt{3+\sin^2\phi}},k=\frac{\sqrt{3}C\cos\phi}{\sqrt{3+\sin^2\phi}} \tag{M.0.3-9}$$

*Mohr—Coulomb*yield criterion is expressed as:

$$f=\frac{1}{3}I_1\sin\phi-\left(\cos\theta+\frac{1}{\sqrt{3}}\sin\theta\sin\phi\right)\sqrt{J_2}+C\cos\phi=0 \tag{M.0.3-10}$$

where $\theta=\frac{1}{3}\sin^{-1}\left(\frac{-3\sqrt{3}}{2}\frac{J_3}{(J_2)^{\frac{3}{2}}}\right)$, $-\frac{\pi}{6}\leqslant\theta\leqslant\frac{\pi}{6}$;

J_3—the third invariant of stress deviator.

②Elasto-plastic matrix

After the material becomes plastic, its elasto-plastic stress-strain relationship increment is expressed as:

$$\{\mathrm{d}\sigma\}=\left([D]-\frac{[D]\left\{\frac{\partial g}{\partial\sigma}\right\}\left\{\frac{\partial f}{\partial\sigma}\right\}^T[D]}{A+\left\{\frac{\partial f}{\partial\sigma}\right\}^T[D]\left\{\frac{\partial g}{\partial\sigma}\right\}}\right)\{\mathrm{d}\varepsilon\}=([D]-[D_{\mathrm{p}}])\{\mathrm{d}\varepsilon\}=[D_{\mathrm{ep}}]\{\mathrm{d}\varepsilon\} \tag{M.0.3-11}$$

where:

$[D]$, $[D_{\mathrm{p}}]$, $[D_{\mathrm{ep}}]$—elastic matrix, plastic matrix and elasto-plastic matrix of the material respectively;

A—parameter related to material hardening; $A=0$ in ideal elasto-plastic condition;

f—function of yielding surface;

g—plastic potential surface function; $g=f$ when associated flow rule is applied.

③Calculation process for elasto-plastic analysis

During incremental time step loading, after some rock and soil become plastic, excessive plastic strain induced by material yield is transferred in the form of initial strain and shared by all elements of the entire system. In each time step, each element and the initial strain corresponding to excessive plastic strain both act in the form of equivalent nodal force and are treated as additional nodal load at subsequent calculation for iterative operation until the final calculation time of time step, while meeting the given accuracy requirements.

4) Visco-elastic model

Three-element generalized Kelvin model consists of elastic elements and Kelvin model in series, as shown in Fig. M.0.3.

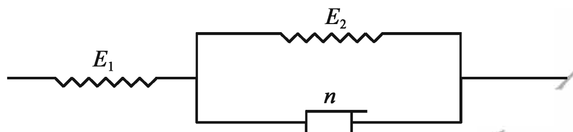


Fig. M.0.3 Generalized *Kelvin model*

Its stress-strain relation is expressed as:

$$\frac{\eta}{E_1+E_2}\dot{\sigma}+\sigma=\frac{\eta E_1}{E_1+E_2}\dot{\varepsilon}+\frac{E_1E_2}{E_1+E_2}\varepsilon \qquad \text{(M.0.3-12)}$$

Creep equation after application of linings is:

$$\varepsilon(t)=\left[\frac{1}{E_1}+\frac{1}{E_2}\left(1-e^{-\frac{E_2}{\eta}t}\right)\right]\sigma_0=\sigma_0 J(t) \qquad \text{(M.0.3-13)}$$

where:

$J(t)$—creep compliance;

σ_0—constant stress.

2 Beam element

See "Element stiffness matrix calculation" in Appendix I.

3 Bar element

Set node displacement of bar element under local coordinate system as $\{\bar{\delta}\}=[\bar{u}_i,\bar{v}_i,\bar{u}_j,\bar{v}_j]^T$ and the corresponding joint force as $\{\bar{f}\}=[\bar{X}_i,\bar{Y}_i,\bar{X}_j,\bar{Y}_j]^T$, then:

$$\{\bar{f}\}=\{\bar{k}\}\{\bar{\delta}\} \qquad \text{(M.0.3-14)}$$

where $[\bar{k}]$ is stiffness matrix of bar element under local coordinate system, and:

$$[\bar{k}]=\begin{bmatrix}\frac{EA}{l} & 0 & -\frac{EA}{l} & 0\\ 0 & 0 & 0 & 0\\ -\frac{EA}{l} & 0 & \frac{EA}{l} & 0\\ 0 & 0 & 0 & 0\end{bmatrix} \qquad \text{(M.0.3-15)}$$ where:

l—pole length;

A—sectional area of pole;

E—elastic modulus of pole.

4 Contact surface element

The contact surface is simulated by no-thickness joint elements. When normal and tangential couplings are not considered, the increment is expressed as:

$$\begin{Bmatrix} \Delta\tau_s \\ \Delta\sigma_n \end{Bmatrix} = \begin{bmatrix} K_s & 0 \\ 0 & K_n \end{bmatrix} \begin{Bmatrix} \Delta u_s \\ \Delta u_n \end{Bmatrix} = [K^e] \begin{Bmatrix} \Delta u_s \\ \Delta u_n \end{Bmatrix} \tag{M. 0. 3-16}$$

where:

K_s—tangential stiffness of contact surface;

K_n—normal stiffness of contact surface.

The stress-strain relation of contact surface material is generally non linear and it is often in plastic stress state. Where Mohr-Coulomb yield condition is used, joint material is assumed to be ideal elasto-plastic material and associated flow rule is applied, for a plain strain problem, the shear sliding plastic matrix of contact surface element may be derived as:

$$[D_p] = \frac{1}{S_0} \begin{bmatrix} K_s^2 & K_s S_1 \\ K_s S_1 & S_1^2 \end{bmatrix} \tag{M. 0. 3-17}$$

where $S_0 = K_s + K_n \tan^2\varphi, S_1 = K_n \tan\varphi$

φ—internal friction angle of contact surface (°).

For non-linear contact surface elements, the relation between stress and relative displacement is expressed as:

$$\tau_s = K_s \cdot \Delta u_s \qquad \sigma_n = K_n v_m \frac{\Delta u_n}{v_m - \Delta u_n} \qquad (\Delta u_n < v_m) \tag{M. 0. 3-18}$$

where:

v_m—normal maximum permissible amount of embedment of contact surface element.

M. 0. 4 Element model:

1 One-dimensional element

For two-node one-dimensional linear element, set nodal displacement as $\{\delta\} = \{u_i, v_i, u_j, v_j\}$ and the displacement of any point on the element is:

$$u = \sum N_i u_i \tag{M. 0. 4-1}$$

where N *is interpolation function and*:

$$N_1 = \frac{1-\xi}{2}, N_2 = \frac{1+\xi}{2} \tag{M. 0. 4-2}$$

2 Triangular element

For a three-node triangular element, assuming node coordinate as $\{x_i, y_i, x_j, y_j, x_m, y_m\}$, nodal displacement as $\{\delta\} = \{u_i, v_i, u_j, v_j, u_m, v_m\}$ and the corresponding nodal force as $\{F\} = \{X_i, Y_i, X_j, Y_j, X_m, Y_m\}$, when linear displacement mode is taken, the displacement of any point in the element is:

$$\begin{pmatrix} u \\ v \end{pmatrix} = [N]\{\delta\} \tag{M.0.4-3}$$

where $[N]$ is shape function matrix, i.e.:

$$[N] = \begin{bmatrix} N_i & 0 & N_j & 0 & N_m & 0 \\ 0 & N_i & 0 & N_j & 0 & N_m \end{bmatrix} \tag{M.0.4-4}$$

where $N_i = \dfrac{1}{2\Delta}(a_i + b_i x + c_i y)$

Δ—element area.

$$\begin{cases} a_i = x_i y_{\mathrm{m}} - x_{\mathrm{m}} y_i \\ b_i = y_i - y_{\mathrm{m}} \\ c_i = x_{\mathrm{m}} - x_i \end{cases}$$

3 Quadrilateral element

For a four-node isoparametric element, assuming nodal displacement as $\{\delta\} = [u_1, v_1, u_2, v_2, u_3, v_3, u_4, v_4]^T$, the displacement mode can be given by bilinear interpolation function:

$$\begin{aligned} u &= N_1 u_1 + N_2 u_2 + N_3 u_3 + N_4 u_4 \\ v &= N_1 v_1 + N_2 v_2 + N_3 v_3 + N_4 v_4 \end{aligned} \tag{M.0.4-5}$$

where N is interpolation function, i.e.:

$$\begin{cases} N_1 = \dfrac{1}{4}(1-\xi)(1-\eta) \\ N_2 = \dfrac{1}{4}(1+\xi)(1-\eta) \\ N_3 = \dfrac{1}{4}(1+\xi)(1+\eta) \\ N_4 = \dfrac{1}{4}(1-\xi)(1+\eta) \end{cases} \tag{M.0.4-6}$$

M.0.5 Construction process simulation:

1 General expression

Simulation of excavation process is generally achieved by applying relief load on excavation boundary. A relatively complete construction stage is called a construction step and assume each construction step includes several increment steps. Then the excavation relief load corresponding to this construction step may be released gradually in the increment steps included to accurately simulate the construction process. In specific calculation, the amount of load relief in each increment step may be controlled by release coefficient.

For the state of each construction stage, the expression for finite element analysis is:

$$[K]_i\{\Delta\delta\}_i = \{\Delta F_r\}_i + \{\Delta F_g\}_i + \{\Delta F_p\}_i \qquad (i = 1, L) \tag{M.0.5-1}$$

$$[K]_i = [K]_0 + \sum_{\lambda=1}^{i}[\Delta K]_\lambda \qquad (i \geqslant 1) \tag{M.0.5-2}$$

where:

L—total number of construction steps;

$[K]_i$—total stiffness matrix of rock/soil masses and structure in the i^{th} construction step;

$[K]_0$—total initial stiffness matrix of rock/soil masses and structure (present prior to the start of construction);

$[\Delta K]_\lambda$—increase or decrease in rock/soil masses and structure stiffness in the λ^{th} construction step during construction to reflect excavation, fill of rock/soil masses element and construction or removal of structural element;

$\{(\Delta F_r)\}_i$—equivalent joint force of relief load at excavation boundary in the i^{th} construction step;

$\{\Delta F_g\}_i$—equivalent joint force of added self-weight in the i^{th} construction step;

$\{\Delta F_p\}_i$—equivalent joint force of increased load in the i^{th} construction step;

$\{\Delta\delta\}_i$—joint displacement increment in the i^{th} construction step.

For each construction step, the expression for finite element analysis of increment loading process is:

$$[K]_{ij}\{\Delta\delta\}_{ij} = \{\Delta F_r\}_i \cdot \alpha_{ij} + \{\Delta F_g\}_{ij} + \{\Delta F_p\}_{ij} \qquad (i = 1, L; j = 1, M) \tag{M.0.5-3}$$

$$[K]_{ij} = [K]_{i-1} + \sum_{\xi=1}^{j}[\Delta K]_{i\xi} \tag{M.0.5-4}$$

where:

M—the number of incremental loadings in each construction step;

$[K]_{ij}$—stiffness matrix when applying the j^{th} load increment in the i^{th} construction step;

α_{ij}—excavation boundary relief load coefficient corresponding to the j^{th} load increment in the i^{th} construction step; $\sum_{j=1}^{M}\alpha_{ij} = 1$ when load at excavation boundary is completely released;

$\{\Delta F_g\}_{ij}$—equivalent joint force of added element self-weight in the j^{th} increment in the i^{th} construction step;

$\{\Delta\delta\}_{ij}$—joint displacement increment in the j^{th} increment step in the i^{th} construction step;

$\{\Delta F_p\}_{ij}$—equivalent joint force of incremental load in the j^{th} increment step in the i^{th} construction step.

2 Excavation process simulation

Excavation effect may be simulated by arranging relief load at excavation boundary and translating it into equivalent nodal force. The expression is:

$$[K - \Delta K]\{\Delta\delta\} = \{\Delta P\} \tag{M.0.5-5}$$

where:

$[K]$—the system's stiffness matrix prior to excavation;

$[\Delta K]$—stiffness of excavated part during excavation;

$\{\Delta P\}$—equivalent joint force of excavation relief load.

The excavation relief load may be calculated by element stress method or Mana method. See Appendix D to the Specifications for details.

3 Simulation of Backfilling Process

The fill effect shall include two parts: change in overall stiffness and increase in added element self-weight load. Its calculation expression is:

$$[K+\Delta K]\{\Delta\delta\} = \{\Delta F_g\} \tag{M.0.5-6}$$

where:

K—the system's stiffness matrix prior to fill;

ΔK—stiffness of added solid element;

$\{\Delta F_g\}$—equivalent joint load of added solid element self-weight.

4 Structure construction and removal

The effect of structure construction shall be reflected as increase in overall stiffness and the impact of added structure self-weight on the system. Its expression is:

$$[K+\Delta K]\{\Delta\delta\} = \{\Delta F_g^s\} \tag{M.0.5-7}$$

where:

K—the system's stiffness matrix prior to structure construction;

ΔK—stiffness of added structure;

$\{\Delta F_g^s\}$—equivalent joint load of constructed structure self-weight.

The effect of structure removal shall include decrease in overall stiffness and the impact of relief of support internal force. The relief of support internal force may be realized by applying a reverse internal force. Its expression is:

$$[K-\Delta K]\{\Delta\delta\} = -\{\Delta F\} \tag{M.0.5-8}$$

where:

K—the system's stiffness matrix prior to structure construction;

ΔK—stiffness of removed structure;

$\{\Delta F\}$—equivalent joint force of internal force in removed structure.

5 Application of incremental load

External load applied during construction may be expressed as incremental load in the increment step. Its expression is:

$$[K]\{\Delta\delta\} = \{\Delta F\} \tag{M.0.5-9}$$

where:

K—the system's stiffness matrix prior to application of incremental load;

$\{\Delta F\}$—equivalent joint force of incremental load applied.

Appendix N
Calculation Method for Quantity of Reinforcement for Reinforced Concrete Flexural and Compression Members

N. 0. 1 The sectional strength of reinforced concrete flexural members shall be calculated from the following formula (Fig. N. 0. 1):

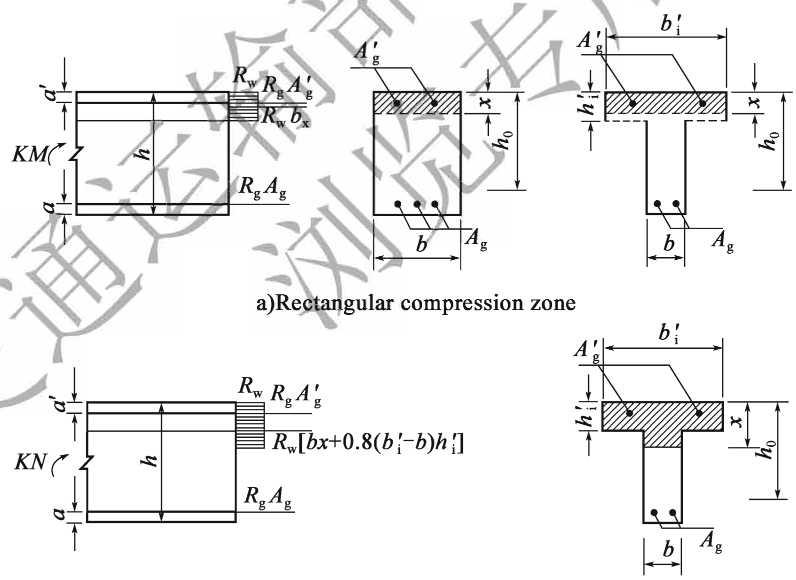


Fig. N. 0. 1 Calculation of sectional strength of reinforced concrete flexural members

1 In the case of rectangular compression zone:

$$KM \leqslant R_w bx(h_0 - x/2) + R_g A'_g(h_0 - a') \quad (N.0.1\text{-}1)$$

The location of neutral axis is determined by:

$$R_g(A_g - A'_g) = R_w bx \quad (N.0.1\text{-}2)$$

2 In the case of T-shaped compression zone

$$KM \leqslant R_{\mathrm{w}}[bx(h_0 - x/2) + 0.8(b_i' - b)h_i'(h_0 - h_i'/2)] + R_{\mathrm{g}}A_{\mathrm{g}}'(h_0 - a') \quad \text{(N.0.1-3)}$$

The location of neutral axis is determined by:

$$R_{\mathrm{g}}(A_{\mathrm{g}} - A_g') = R_{\mathrm{w}}[bx + 0.8(b_i' - b)h_i'] \quad \text{(N.0.1-4)}$$

When calculation for flexural members is based on the above formulae, the height of concrete compression zone shall comply with Eq. (N.0.1-5) and (N.0.1-6) and its sectional strength shall comply with Eq. (N.0.1-7). However, if the structure contains no compression bar or the compression bar is not considered in calculation, only Eq. (N.0.1-5) needs to be complied with.

$$x \leqslant 0.55h_0 \quad \text{(N.0.1-5)}$$

$$x \geqslant 2a' \quad \text{(N.0.1-6)}$$

$$KM \leqslant 0.5R_{\mathrm{w}}bh_0^2 \quad \text{(N.0.1-7)}$$

where:

K—Safety factor, to be obtained from Table 9.2.4-2 herein;

M—bending moment (MN · m);

R_{w}—ultimate bending compressive strength of concrete, $R_{\mathrm{w}} = 1.25R_{\mathrm{a}}$, obtained from Table 5.2.4 in the Specifications;

R_{g}—standard value of tensile or compressive strength of rebar, obtained from Table 5.2.11 in the Specifications;

A_{g}, A_{g}'—sectional area of rebar in tension and compression zones (m^2);

a, a'—the distance from gravity center of bar A_{g} or A_{g}' to nearest edge of cross-section (m);

h—section height (m);

h_0—effective height of section, $h_0 = h - a$;

x—height of concrete compression zone (m);

b—width of rectangular section or rib width of T-shaped section (m);

b_i'—calculated width of flange in T-shaped section compression zone (m), taken as minimum value from Table N.0.1;

h_i'—height of flange in T-shaped section compression zone (m).

Table N.0.1 Width of flange in T-shaped section compression zone

S. N.	Consideration	Rib beam	Independent beam
1	Span K	$l/3$	$l/3$
2	Clear distance of beam rib	$b + s$	—
3	Height of flange h_i' ($h_i'/h_0 \geqslant 0.1$)	—	$B + 12h_i'$

N.0.2 For a flexural member with rectangular or T-shaped cross-section, its cross-section shall meet the requirements of formula (N.0.2):

$$KQ \leqslant 0.3R_{\mathrm{a}}bh_0 \quad \text{(N.0.2)}$$

where:

K—safety factor, obtained from Table 9.2.4-2;

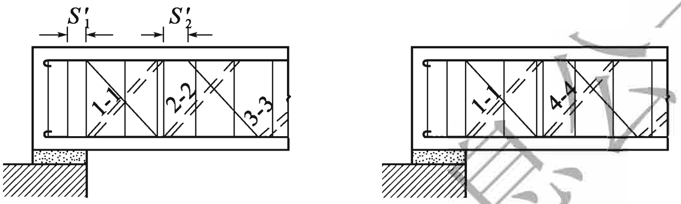
Q—shear force (MN);

b—width of rectangular section or rib width of T-shaped section (m).

Meanings of other symbols are the same as above.

N.0.3 Calculation of oblique-section shear strength shall be based on the location as specified below:

1 Section at bearing edge (Section 1-1 in Fig. N.0.3);



a)Bent-up bar b)Longitudinal bar and stirrup

1-1——Oblique section at bearing edge; 2-2 and 3-3——Oblique section of bent-up bar in tension zone at bent-up point; 4-4——Oblique section at changes in stirrup quantity and spacing

Fig. N.0.3 Calculation location for oblique-section shear strength

2 Section of bent-up bar in tension zone at bent-up point (Sections 2-2 and 3-3 in Fig. N.0.3);

3 Section in tension zone at changes in stirrup quantity and spacing (Section 4-4 in Fig. N.0.3).

N.0.4 For a flexural member of rectangular or T section reinforced only with stirrups, its oblique-section shear strength shall be calculated from:

$$KQ \leqslant Q_{\mathrm{kh}} \quad (\mathrm{N.0.4\text{-}1})$$

$$Q_{\mathrm{kh}} = 0.07R_{\mathrm{a}}bh_0 + \alpha_{\mathrm{kh}}R_{\mathrm{g}}\frac{A_{\mathrm{K}}}{S}h_0 \quad (\mathrm{N.0.4\text{-}2})$$

$$A_{\mathrm{k}} = na_{\mathrm{k}} \quad (\mathrm{N.0.4\text{-}3})$$

where:

Q—maximum oblique-section shear force (MN);

Q_{kh}—shear strength of compression zone concrete and stirrup on oblique section (MPa);

α_{kh}—shear strength influence factor, as specified below:

If $KQ/(bh_0) \leqslant 0.2R_{\mathrm{a}}$, then $\alpha_{\mathrm{kh}} = 2.0$;

If $KQ/(bh_0) = 0.3R_{\mathrm{a}}$, then $\alpha_{\mathrm{kh}} = 1.5$;

If $KQ/(bh_0)$ is an intermediate value, then the value of α_{kh} is obtained by linear interpolation method;

A_{k}—Total sectional area of stirrups placed within the same section (m^2);

n—number of stirrups in the same section;

a_k—sectional area of single-bar stirrup (m^2);

S—spacing of stirrups along the length of member (m);

R_g—standard tensile strength of stirrups, obtained from Table 5.2.12 in the Specifications.

N.0.5 For a rectangular or T-section flexural member reinforced with stirrups and bent-up bar, its oblique-section shear strength shall be calculated from formula (N.0.5):

$$KQ \leqslant Q_{kh} + 0.8 R_g A_w \sin\theta \tag{N.0.5}$$

where:

Q—shear force at the bent-up bar (MN), obtained according to N.0.6 of the Specifications;

A_w—sectional area of bent-up bars placed in the same bent plane (m^2);

θ—the angle between bent-up bar and longitudinal axis of the member (°).

N.0.6 In calculation for bent-up bars, the shear force Q may be taken as specified below according to Fig. N.0.3a):

1 In calculation for the first row (relative to the bearing) of bent-up bars, take share force value on the edge of the bearing.

2 In calculation for each subsequent row of bent-up bars, take the shear force value at the start of bent-up bars in the previous row (relative to the bearing).

N.0.7 For a rectangular or T-section flexural member that meets the requirement of formula (N.0.7), oblique-section shear strength calculation is not required and stirrups shall be configured per structural requirements.

$$KQ \leqslant 0.07 R_a b h_0 \tag{N.0.7}$$

N.0.8 For a reinforced concrete compression member of rectangular section with large eccentricity ($x \leqslant 0.55 h_0$), its sectional strength shall be calculated from the following formula (Fig. N.0.8):

$$KN \leqslant R_w bx + R_g (A'_g - A_g) \tag{N.0.8-1}$$

or

$$KNe \leqslant R_w bx (h_0 - x/2) + R_g A'_g (h_0 - a') \tag{N.0.8-2}$$

In this case, the location of neutral axis is determined by:

$$R_g (A_g e \mp A'_g e') = R_w bx (e - h_0 + x/2) \tag{N.0.8-3}$$

When axial force N acts on the middle between gravity centers of bar A_g and bar A'_g, the second term in Eq. (N.0.8-3) from the left shall take +; when N acts on a location outside

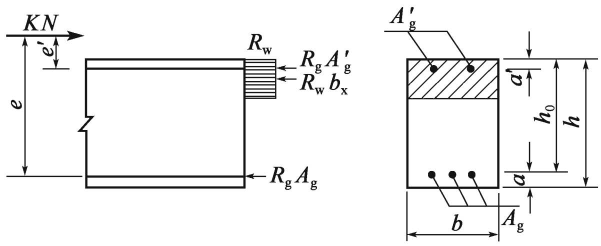


Fig. N. 0. 8 Sectional strength calculation for reinforced concrete compression member with large eccentricity

gravity centers of A_g and A'_g, " − " shall be taken.

If calculation takes into account compression bars, then the height of concrete compression zone shall comply with Eq. (N. 0. 1-6); if this is not satisfied, formula (N. 0. 8-4) shall be used:

$$KNe' \leqslant R_g A_g (h_0 - a') \qquad (N.0.8\text{-}4)$$

where:

N—axial force (MN);

e, e'—the distance from the gravity centers of rebars A_g and A'_g to axial force action point (m);

Meanings of other symbols are the same as above.

If the sectional strength of the member obtained using formula (N. 0. 8-4) is less than when compression rebar is not considered, then compression rebar shall not be considered in calculation.

N. 0. 9 For a reinforced concrete compression member of rectangular section with small eccentricity ($x > 0.55h_0$), its sectional strength shall be calculated from the following formula (Fig. N. 0. 9):

$$KNe \leqslant 0.5R_w bh_0^2 + R_g A'_g (h_0 - a') \qquad (N.0.9\text{-}1)$$

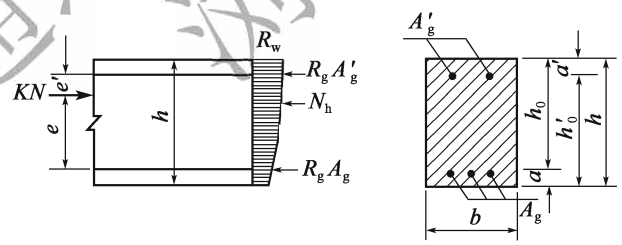


Fig. N. 0. 9 Sectional strength calculation for reinforced concrete compression member with small eccentricity

When axial force N acts on a location between the gravity centers of rebars A_g and A'_g, the following requirements shall be met:

$$KNe' \leqslant 0.5R_w bh_0^2 + R_g A_g (h'_0 - a) \qquad (N.0.9\text{-}2)$$

Meanings of symbols in the equation are the same as above.

N. 0. 10 Calculation of a reinforced concrete eccentrically compressed member of rectangular section shall take into account the effect of deflection of the member within bending moment action plane on the increase in axial force eccentricity. In this case, coefficient of eccentricity amplification η shall be applied to axial force eccentricity e_0. The value of η is given by:

$$\eta = \frac{1}{1 - \frac{KN}{10\alpha E_c I_0} H^2} \qquad (N.0.10\text{-}1)$$

where:

K—Safety factor, to be obtained from Table 9.2.4-2 herein;

E_c—compressive elasticity modulus of concrete, as specified under 5.2.5 of the Specifications;

I_0—transformed section moment of inertia in full section (including rebar) of concrete (m^4);

H—member height (m);

α—coefficient related to eccentricity, given by:

$$\alpha = \frac{0.12}{0.3 + \frac{e_0}{h}} + 0.17 \qquad (N.0.10\text{-}2)$$

If $e_0/h \geqslant 1$, then take $\alpha = 0.26$.

For tunnel lining, cut-and-cover tunnel arch ring and cut-and-covertunnel side walls with tightly backfilled back and when the ratio of member height to sectional side length within bending moment action plain $H/h \leqslant 8$, $\eta = 1$ may be taken.

For an eccentrically compressed member, in addition to calculating the strength within bending moment action plane, this strength shall be checked per axial compression member. In this case, the effect of bending moment is not considered, but factor of longitudinal bending shall be considered per Table N.0.10.

Table N.0.10 Factor of longitudinal bending of reinforced concrete member

H/b	≤8	10	12	14	16	18	20	22	24	26	28	30
Factor of longitudinal bending ϕ	1.00	0.98	0.95	0.92	0.87	0.81	0.75	0.70	0.65	0.60	0.56	0.52

Notes:

1. H is the calculated length of member; $H = 0.5l$ when both ends are rigidly fixed; $H = 0.7l$ if one end is rigidly fixed and the other end is immovable hinge; $H = l$ when both ends are immovable hinges; $H = 2l$ if one end is rigidly fixed and the other is free.
2. l is full length of the member; b is short side dimension of rectangular section member.

N.0.11 For a reinforced concrete tension, flexural and eccentrically compressed member, its maximum crack width w_{max} may be calculated from Eq. (N.0.11). If $e_0 \leqslant 0.55h_0$ checking of crack width is not required.

$$w_{max} = \alpha\psi\gamma(2.7C_s + 0.1d/\rho_{te})\sigma_s/E_s \qquad (N.0.11)$$

where:

α—member stress characteristic coefficient; take $\alpha = 2.7$ for axial tension member; take $\alpha = 2.1$ for flexural and eccentrically compressed members; take $\alpha = 2.4$ for eccentric tension member;

ψ—strain nonuniformity coefficient of longitudinal tension rebars between cracks $\psi = 1.1 - 0.65 f_{ctk} / (\rho_{te} \sigma_s)$ where ρ_{te} longitudinal tension reinforcement ratio calculated based on effective tension concrete area: $\rho_{te} = A_s / A_{ce}$; if $\rho_{te} < 0.01$, then consider $\rho_{te} = 0.01$;

A_s—area of longitudinal bars in tension zone;

A_{ce}—effective tension sectional area of concrete (Fig. N.0.11): for a tension member, A_{ce} is taken as sectional area of the member; for a flexural, eccentrically compressed or eccentric tension member, take $A_{ce} = 0.5bh + (b_f - b) h_1$; for a rectangular section member, take $A_{ce} = 0.5bh$ (b and h are the width and height of concrete section respectively); if $\psi < 0.4$, then consider $\psi = 0.4$; if $\psi > 1.0$, then consider $\psi = 1.0$; for a member directly carrying repeated load, take $\psi = 1.0$;

γ—surface characteristic coefficient of longitudinal tension bar; take 0.7 for deformed bar and 1.0 for plain bar;

C_s—the distance from the outer edge of the outermost longitudinal tension bar to the bottom of tension zone (mm); if $C_s < 20$, then consider $C_s = 20$;

d—rebar diameter (mm); when rebars of different diameters are adopted, $d = 4A_s / u$ *where u is the sum of sectional perimeters of longitudinal tension bars*;

σ_s—stress in longitudinal tension bars (MPa), calculated according to N.0.12 herein;

E_s—elastic modulus of rebars, as specified in 5.2.10 of the Specifications.

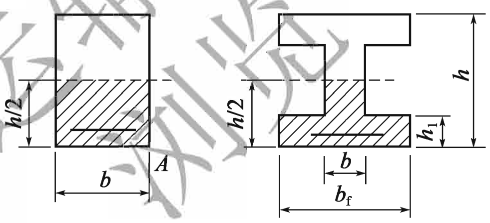


Fig. N.0.11 Effective tension sectional area of concrete

N.0.12 In checking crack width, stress in longitudinal tension bars in the member may be calculated from the following formulae:

1 Flexural members:

$$\sigma_s = M_s / (0.87 h_0 A_s) \quad \text{(N.0.12-1)}$$

2 Eccentrically compressed members:

$$\sigma_s = N_s (e - z) / (A_s z) \quad \text{(N.0.12-2)}$$

3 Axial tension members:

$$\sigma_s = N_s / A_s \quad \text{(N.0.12-3)}$$

4 Eccentric tension members:

$$\sigma_s = N_s e' / [A_s (h_0 - a_s')] \quad (N.0.12\text{-}4)$$

where:

M_s, N_s—values of bending moment (MN · m) and axial force (MN) calculated based on load combinations;

A_s—sectional area of longitudinal rebars in tension zone (m^2);

e—the distance from axial pressure point to resultant stress point in longitudinal tension rebars (m), given by $e = \eta e_i + y_{sp}$, where e_i is initial eccentricity (m); η is coefficient of eccentricity amplification taking into account deflection effect, obtained from Table N.0.10; y_{sp} is the distance from sectional gravity center to A_s resultant stress point.

z—the distance from resultant stress point in longitudinal tension bars to resultant force in compression zone (m), $z = [0.87 - 0.12 (h_0/e)^2] h_0$ and $z < 0.87 h_0$;

a_s'—the distance from resultant stress point in longitudinal non-prestressed compression bars to sectional near-edge (m);

e'—the distance from axial force action point to resultant stress point in longitudinal compression bars (m);

h_0—effective section height (m).

Appendix P
Tables of Tunnel Support Parameters

P.0.1 Design parameters of composite linings for a two-lane tunnel may be obtained from Table P.0.1.

Table P.0.1 Design parameters of composite linings for a two-lane tunnel

Surrounding rock class	Primary support								Secondary lining thickness (cm)	
	Shotcrete thickness (cm)		Rockbolt (m)			Spacing of reinforcement meshes (cm)	Steel rib support		Arch and wall concrete	Arch invert concrete
	Arch and wall	Arch invert	Location	Length	Spacing		Spacing (m)	Section height (cm)		
Ⅰ	5	—	Local	2.0 ~ 3.0	—	—	—	—	30 ~ 35	—
Ⅱ	5 ~ 8	—	Local	2.0 ~ 3.0	—	—	—	—	30 ~ 35	—
Ⅲ	8 ~ 12	—	Arch and wall	2.0 ~ 3.0	1.0 ~ 1.2	Local spacing @25 × 25	—	—	30 ~ 35	—
Ⅳ	12 ~ 20	—	Arch and wall	2.5 ~ 3.0	0.8 ~ 1.2	@25 × 25 for arch and wall	0.8 ~ 1.2 for arch and wall	0 or 14 ~ 16	35 ~ 40	0 or 35 ~ 40
Ⅴ	18 ~ 28	—	Arch and wall	3.0 ~ 3.5	0.6 ~ 1.0	@20 × 20 for arch and wall	0.6 ~ 1.0 for arch, wall andarch invert	14 ~ 22	35 ~ 50 reinforced concrete	0 or 35 ~ 50 reinforced concrete
Ⅵ	Determined by test or calculation									

Notes:

1. Higher values may be taken if groundwater is present and lower values may be taken if groundwater is absent.
2. Steel rib support when used should be steel lattice girder.
3. Steel rib support is not required when shotcrete thickness is less than 18cm.
4. "0 or ..." means placement is not required; placement when required shall meet minimum thickness requirement.

P. 0. 2 Design parameters of composite linings for a three-lane tunnel may be obtained from Table P. 0. 2.

Table P. 0. 2 Design parameters of composite linings for a three-lane tunnel

Surrounding rock class	Primary support								Secondary lining thickness (cm)	
	Shotcrete thickness (cm)		Rockbolt (m)			Spacing of reinforcement meshes (cm)	Steel rib support		Arch and wall concrete	Arch invert concrete
	Arch and wall	Arch invert	Location	Length	Spacing		Spacing (m)	Section height (cm)		
Ⅰ	5 ~ 8	—	Local	2.0 ~ 3.5	—	—	—	—	35 ~ 40	—
Ⅱ	8 ~ 12	—	Local	2.0 ~ 3.5			—	—	35 ~ 40	—
Ⅲ	12 ~ 20	—	Arch and wall	2.5 ~ 3.5	1.0 ~ 1.2	@ 25 × 25 for arch and wall	1.0 ~ 1.2 for arch and wall	0 or 14 ~ 16	35 ~ 45	—
Ⅳ	16 ~ 24	—	Arch and wall	3.0 ~ 3.5	0.8 ~ 1.2	@ 20 × 20 for arch and wall	0.8 ~ 1.2 for arch and wall	16 ~ 20	40 ~ 50 ■	0 or 40 ~ 50
Ⅴ	20 ~ 30	—	Arch and wall	3.5 ~ 4.0	0.5 ~ 1.0	@ 20 × 20 for arch and wall	0.5 ~ 1.0 for arch, wall andarch invert	18 ~ 22	50 ~ 60 reinforced concrete	0 or 50 ~ 60 reinforced concrete
Ⅵ	Determined by test or calculation									

Notes:

1. Higher values may be taken if groundwater is present and lower values may be taken if groundwater is absent.
2. Steel rib support when used should be steel lattice girder.
3. Steel rib support is not required when shotcrete thickness is less than 18cm.
4. "0 or . . . " means placement is not required; placement when required shall meet minimum thickness requirement.
5. "■" means reinforced concrete may be adopted.

P. 0. 3 Where shotcrete-and-rockbolt is installed as permanent support, design parameters may be obtained from Table P. 0. 3.

Table P. 0. 3 Design parameters of shotcrete-and-rockbolt permanent support

Surrounding rock class	Ⅰ	Ⅱ	Ⅲ
Pedestrian cross passage	Shotcrete 5cm	Shotcrete 5cm	①Shotcrete 6 ~ 8cm ②Rockbolt Φ22, 1.0 ~ 2.0m long
Vehicle cross passage	Shotcrete 5cm	①Shotcrete 5cm ②Rockbolt Φ22, 1.5 ~ 2.0m long ③At 1.0 × 1.0m spacing	①Shotcrete 8 ~ 10cm ②Rockbolt Φ22, 2.0 ~ 2.5m long ③Rockbolt spacing @ 1.0 × 1.0m

continued

Surrounding rock class	Ⅰ	Ⅱ	Ⅲ
Two-lane tunnel	Shotcrete 5cm	①Shotcrete 5 ~ 8cm ②Rockbolt Φ22, 2.0 ~ 2.5m long ③Rockbolt spacing 1.2 × 1.2m	①Concrete 8 ~ 15cm ②Rockbolt Φ22, 2.0 ~ 3.5m long ③Rockbolt spacing @1.0 × 1.0m ④Reinforcement mesh Φ6.5, @25 × 25cm

Note: In surrounding rock of Classes IV-VI with weak and fractured ground and abundant groundwater, composite linings should be adopted.

P.0.4 Design parameters of composite linings for a two-tube four-lane twin tunnels with small clearance may be obtained from Table P.0.4.

Table P.0.4 Recommended reinforcement of support measures for two-tube four-lane twin tunnels with small clearance

<table>
<tr><td colspan="3">Two-tube impact</td><td colspan="2">Serious impact</td><td colspan="2">Moderate impact</td><td colspan="2">Slight impact</td></tr>
<tr><td rowspan="2">Surrounding rock class</td><td>Class III</td><td rowspan="2">Clear distance between tubes</td><td colspan="2">≤0.375B</td><td colspan="2">(0.375 ~ 0.75)B</td><td colspan="2">(0.75 ~ 2.0)B</td></tr>
<tr><td>Class IV</td><td colspan="2">≤0.5B</td><td colspan="2">(0.5 ~ 1.0)B</td><td colspan="2">(1.0 ~ 2.5)B</td></tr>
<tr><td></td><td>Class V</td><td></td><td colspan="2">≤0.75B</td><td colspan="2">(0.75 ~ 1.5)B</td><td colspan="2">(1.5 ~ 3.5)B</td></tr>
<tr><td rowspan="9">Support reinforcement principle</td><td rowspan="8">Primary support</td><td rowspan="2">Shotcrete</td><td>Class III</td><td rowspan="2">Thickness increase by 3 ~ 8cm</td><td>Class III</td><td>Thickness increase by 2 ~ 5cm</td><td colspan="2" rowspan="3">Thickness increase by 2 ~ 5cm</td></tr>
<tr><td>Class IV</td><td>Class IV</td><td>Thickness increase by 3 ~ 8cm</td></tr>
<tr><td></td><td>Class V</td><td></td><td>Class V</td><td>Thickness increase by 3 ~ 8cm</td></tr>
<tr><td>Systematic rock bolting</td><td>Class III</td><td>Length increase by 50cm
Spacing decrease by 10 ~ 20cm</td><td>Class III</td><td>Not reinforced</td><td>Class III</td><td rowspan="3">Not reinforced</td></tr>
<tr><td></td><td>Class IV</td><td></td><td>Class IV</td><td rowspan="2">Length increase by 50cm</td><td>Class IV</td></tr>
<tr><td></td><td>Class V</td><td></td><td>Class V</td><td>Class V</td></tr>
<tr><td>Steel rib support</td><td>Class III</td><td>Adding steel lattice frame</td><td>Class III</td><td>Adding steel lattice frame locally in soft rock zones</td><td colspan="2">Not reinforced</td></tr>
<tr><td></td><td>Class IV</td><td rowspan="2">Spacing decrease by 10 ~ 20cm</td><td>Class IV</td><td>Spacing decrease by 10 ~ 20cm</td><td colspan="2"></td></tr>
<tr><td></td><td></td><td>Class V</td><td>Class V</td><td></td><td colspan="2"></td></tr>
</table>

continued

Two-tube impact		Serious impact		Moderate impact		Slight impact
Support reinforcement principle	Secondary lining	Class III	Thickness increase by 0 ~ 5cm	Class III	Not reinforced	Not reinforced
		Class IV	0 ~ 5cm increase in reinforced concrete thickness	Class IV	0 ~ 5cm increase in reinforced concrete thickness	
		Class V	5 ~ 10cm increase in reinforced concrete thickness	Class V	5 ~ 10cm increase in reinforced concrete thickness	
	Reinforcing of rock pillar	Class III	Cross (prestressed) rockbolt	Class III	Cross anchor if < 6m; longer systematic rock bolting if > 6m	Not reinforced
		Class IV	Tremie grouting assisted by prestressed cross anchors	Class IV	Longer prestressed systematic rock bolting	
		Class V	Tremie grouting assisted by longer prestressed systematic rock bolting (prestressed cross anchors)	Class V	Longer prestressed systematic rock bolting	

Notes:

1. Parameters listed in this table are values added to support parameters for ordinary two-lane tunnels.
2. For surrounding rock of Classes I and II, reference may be made to this table to specify support reinforcement measures.
3. Due to a lack of available data on Class VI surrounding rock and three-lane twin tunnels with narrow pillar, decision shall be made based on calculation and analysis considering site conditions.
4. *B* is tunnel excavation width.

P.0.5 Design parameters of composite linings for a two-lane composite twin-arch tunnel may be obtained from Table P.0.5.

P.0.6 Design parameters of composite linings for a three-lane composite twin-arch tunnel may be obtained from Table P.0.6.

Table P.0.5 Design parameters of composite linings for a two-lane composite twin-arch tunnel

Surrounding rock class	Primary support						Secondary lining		
	Shotcrete thickness (cm)		Rockbolt (m)		Spacing of steel rib supports (m)		Reinforced concrete rock pillar (m)	Arch and wall (cm)	Arch invert (cm)
	Arch and wall	Arch invert	Length	Longitudinal spacing	Spacing	Section height			
Ⅰ	5 ~ 8	—	2.0 ~ 3.0	Local	—	—	1.4 ~ 2.5	35 reinforced concrete	–
Ⅱ	8 ~ 12	—	2.5 ~ 3.0	Local	—	—	1.4 ~ 2.5	35 reinforced concrete	–
Ⅲ	10 ~ 18	—	2.5 ~ 3.5	1.0 ~ 1.2	—	—	1.4 ~ 2.5	40 reinforced concrete	–
Ⅳ	16 ~ 22	0 or 16 ~ 22	3.0 ~ 3.5	0.8 ~ 1.0	0.8 ~ 1.2	0 or 16 ~ 18	1.4 ~ 2.5	40 ~ 50 reinforced concrete	0 or 40 ~ 50 reinforced concrete
Ⅴ	20 ~ 28	20 ~ 28	3.0 ~ 4.0	0.5 ~ 0.8	0.5 ~ 1.0	16 ~ 22	1.4 ~ 2.5	45 ~ 60 reinforced concrete	45 ~ 60 reinforced concrete

Notes:

1. Take upper limit values for lining support parameters for monolithic twin-arch tunnel.
2. Reinforcement mesh may be the same as for composite linings for a two-lane tunnel.
3. "0 or . . . " means placement is not required; placement when required shall meet minimum thickness requirement.

Table P.0.6 Design parameters of composite linings for a three-lane composite twin-arch tunnel

Surrounding rock class	Primary support						Secondary lining		
	Shotcrete thickness (cm)		Rockbolt (m)		Steel rib support		Reinforced concrete rock pillar (m)	Arch and wall (cm)	Arch invert (cm)
	Arch and wall	Arch invert	Length	Longitudinal spacing	Spacing (m)	Section height (cm)			
I	8 ~ 10	—	2.5 ~ 3.0	Local	—	—	2.0 ~ 3.5	40 reinforced concrete	–
II	10 ~ 12	—	2.5 ~ 3.5	Local	—	—	2.0 ~ 3.5	40 reinforced concrete	–
III	12 ~ 22	—	3.0 ~ 3.5	1.0 ~ 1.2	1.0 ~ 1.2	0 or 16 ~ 18	2.0 ~ 3.5	40 ~ 45 reinforced concrete	0 or 40 ~ 45

continued

Surrounding rock class	Primary support						Secondary lining		
	Shotcrete thickness (cm)		Rockbolt (m)		Steel rib support		Reinforced concrete rock pillar (m)	Arch and wall (cm)	Arch invert (cm)
	Arch and wall	Arch invert	Length	Longitudinal spacing	Spacing (m)	Section height(cm)			
IV	20 ~ 26	0 or 15 ~ 20	3.0 ~ 4.0	0.8 ~ 1.0	0.8 ~ 1.0	16 ~ 20	2.0 ~ 3.5	45 ~ 60 reinforced concrete	0 or 45 ~ 60 reinforced concrete
V	24 ~ 30	24 ~ 30	3.5 ~ 4.5	0.5 ~ 1.0	0.5 ~ 1.0	20 ~ 26	2.0 ~ 3.5	50 ~ 70 reinforced concrete	50 ~ 70 reinforced concrete

Notes:

1. Take upper limit values for lining support parameters for monolithic twin-arch tunnel.
2. Reinforcement mesh may be the same as for composite linings for a three-lane tunnel.
3. "0 or ..." means placement is not required; placement when required shall meet minimum thickness requirement.

P.0.7 Design parameters of vertical shaft lining support may be obtained from Table P.0.7.

Table P.0.7 Vertical shaft lining support parameters

Surrounding rock class	Shotcrete and rockbolt lining		Monolithic lining	Composite lining		
				Primary support		Secondary lining
	$D < 5$m	5m$\leqslant D \leqslant$7m		$D < 5$m	5m$\leqslant D \leqslant$7m	
I	Shotcrete 10cm thick	Shotcrete 10 ~ 15cm thick, spot rockbolt installed when necessary	cast-in-situ or reinforced concrete 30cm thick or masonry 40cm thick	—	—	—
II	Shotcrete 10 ~ 15cm thick, rockbolts 1.5 ~ 2m long at 1 ~ 1.5m intervals	Shotcrete 15 ~ 20cm thick, rockbolts 2 ~ 2.5m long at 1m intervals, complete with reinforcement mesh, steel ring beam added if necessary	cast-in-situ or reinforced concrete 30cm thick or masonry 50cm thick	—	—	—
III	Shotcrete 15 ~ 20cm thick, rockbolts 2 ~ 2.5m long at 1m intervals, complete with reinforcement mesh, steel reinforced ring beam added if necessary	Shotcrete 20cm thick, rockbolts 2.5 ~ 3m long at 1m intervals, complete with reinforcement mesh, steel ring beam added	Concrete or reinforced concrete 40cm thick or masonry 60cm thick	Shotcrete 5 ~ 10cm thick, rockbolts 1.5 ~ 2m long at 1m intervals, reinforcement mesh installed when necessary	Shotcrete 10 ~ 15cm thick, rockbolts 2 ~ 2.5m long at 1m intervals, reinforcement mesh installed locally if necessary	30cm

continued

Surrounding rock class	Shotcrete and rockbolt lining		Monolithic lining	Composite lining		
				Primary support		Secondary lining
	$D<5$m	5m$\leq D \leq$7m		$D<5$m	5m$\leq D \leq$7m	
IV			Concrete or reinforced concrete 50cm thick or masonry 70cm thick	Shotcrete 10 ~ 15cm thick, rockbolts 2 ~ 2.5m long at 1m intervals, reinforcement mesh installed when necessary	Shotcrete 15 ~ 20cm thick, rockbolts 2.5 ~ 3m long at 0.75 ~ 1m intervals, complete with reinforcement mesh	40cm
V			Concrete or reinforced concrete 60cm thick or masonry 80cm thick	Shotcrete 15 ~ 20cm thick, rockbolts 2.5 ~ 3m long at 0.75 ~ 1m intervals, complete with reinforcement mesh, steel ring beam installed if necessary	Shotcrete 20 ~ 25cm thick, rockbolts 3 ~ 3.5m long at 0.5 ~ 0.7m intervals, with reinforcement mesh, steel arch support installed if necessary	50cm

Notes:

1. In Class VI surrounding rock, special support measures shall be put in place.
2. *D* is vertical shaft diameter; for a vertical shaft larger than 7m in diameter, special design shall be developed.

P.0.8 Design parameters of inclined shaft, parallel heading, transverse gallery and duct linings may be obtained from Table P.0.8.

Table P.0.8 Inclined shaft, parallel heading, transverse gallery and duct lining parameters

Surrounding rock class	Shotcrete and rockbolt lining	Cast-in-situ concrete lining	Composite lining	
			Primary support	Secondary lining
I	5cm	20cm	Not supported; local shotcrete or cement mortar applied for surface protection	20cm
II	5cm	20cm	Local shotcrete, 5cm thick	20cm
III	10cm, local rockbolt 2 ~ 2.5m long	25 ~ 30cm	Shotcrete 5 ~ 8cm thick, spot rockbolt installed, 2m long	20cm
IV	—	35 ~ 40cm	Shotcrete 8 ~ 10cm thick, rockbolts installed at the arch, 2 ~ 2.5m long at 1 ~ 1.2m intervals, reinforcement mesh installed at the arch if necessary	25 ~ 30cm

continued

Surrounding rock class	Shotcrete and rockbolt lining	Cast-in-situ concrete lining	Composite lining	
			Primary support	Secondary lining
V	—	45 ~ 50cm, arch invert placed as necessary	Shotcrete 10 ~ 15cm thick, with systematic rock bolting 2.5 ~ 3m long at 1m intervals, complete with reinforcement mesh	35 ~ 40cm, arch invert placed as necessary

Notes:

1. In Class VI surrounding rock, special design shall be developed.
2. Shotcrete and rockbolt linings are suitable only for Class I-III surrounding rock where groundwater is undeveloped and non-corrosive and the effect of smooth blasting can be guaranteed.
3. This table is for channel width not greater than 5m. When channel width is greater than 5m, separate design shall be developed.

Wording Explanation for the *Specifications*

1 The strictness in execution of the *Specifications* is expressed by using the wording as follows:

1) MUST—A very restrict requirement in any circumstances.

2) SHALL—A mandatory requirement in normal circumstances.

3) SHOULD—An advisory requirement.

4) MAY—A permissive condition. No requirement is intended.

2 Expressions used for reference to standards are explained as follows:

The standards for which a year is added to the standard number shall be the specific verions to be used. Otherwise they shall be the latest available verions.

Background to Provisions

1 General

1.0.1 China is a mountainous country, and about 70% of the land is of mountainous region or hilling terrain. The tunnel scheme should be taken seriously in the construction of mountain roads, in order to shorten the mileage, protect the environment and conserve the land. Since the opening of China 40 years ago, China's highway traffic construction has developed rapidly, and the scale of highway tunnel construction is getting larger. Especially in the decade after 2000, the average annual mileage increase of highway tunnel construction in China was nearly 1,000km. The length of the tunnel has grown from 3km in the early 1990s to more than 10km today. In terms of the type of tunnel, it has developed from the traditional single-tube two-lane tunnel to the current three-lane tunnel and four-lane tunnel. The structural forms of the tunnel include twin-arch tunnel, twin tunnels with small clearance, branching-out tunnel as well as "underground interchange structure", "bridge-tunnel combinations", "ramp entrance", etc. The completion of these tunnels has shortened the traveling distance, improved the highway transportation efficiency, and played an important role in reducing the traffic accident rate and protecting the environment, achieving good social and economic benefits. In recent years, a series of scientific researches have been carried out on practical issues of tunnel and underground engineering in China, and the theoretical and technical levels have been further improved, achieving new accumulation of experiences and techniques of tunnel construction, laying a foundation for the revision of the *Specifications for Design of Highway Tunnel* (JTG D70—2004).

1.0.2 The methods of constructing highway tunnels in mountain and rock / soil mainly include drill-and-blast (D&B) method and tunnel boring machine (TBM) method. The Specifications apply to new, reconstructed or expanded highway tunnels excavated primarily by D&B.

1.0.3 The basic purpose of constructing highway tunnel is to shorten highway mileage and to improve traffic convenience. Under the premise of meeting the basic functions of highway, considerations for tunnel design shall be given to ensure structural safety and operational safety; and to achieve the benefit of energy conservation and environmental protection during construction and usage, and at the same time to save construction cost.

1.0.4 The division according to the tunnel length is mainly to give people a broad concept; and this division based on length is called Tunnel Classification. The International Tunneling Association divides tunnels into extra-long, long, medium and short tunnels according to their length. The division based on the two indices of tunnel length and traffic volume is called tunnel grades. For example, the tunnels in UK, Norway, Japan, France and Sweden are graded in accordance with these two indices. Other countries, such as Switzerland, only distinguish the length distribution range of the tunnel, but not the length. Germany and Australia only regulate the safety facilities that should be installed according to the length of the tunnel.

In China, tunnels are classified into four categories according to their length. This classification method is appropriate judged from the usage in recent years. The starting position of tunnel length is shown in Fig. 1-1.

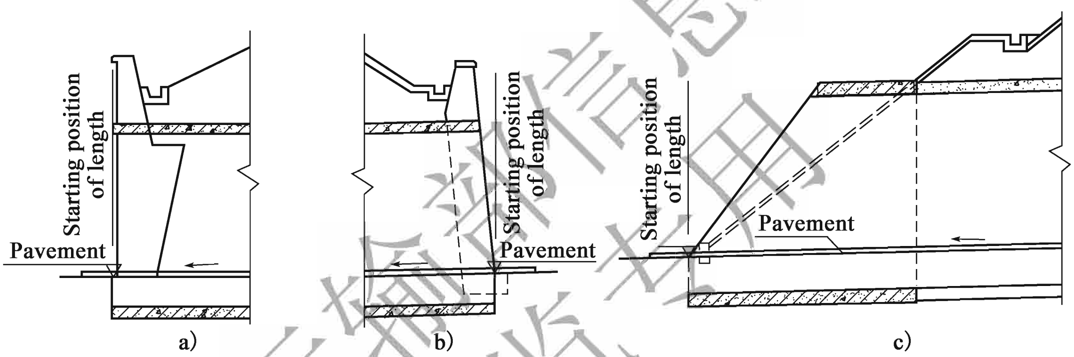


Fig. 1-1 Schematic Diagram of Starting Position of Tunnel Length

1.0.5 As mentioned here, the main tunnel structures are civil engineering structures, including tunnel portal, support lining structures of tunnel main cavern and each auxiliary cavern (such as air duct, inclined shaft, vertical shaft, tunnel ventilation fan (TVF) room, power distribution room, cross passage, refuge cavern, etc.), pavement structure, arch invert and the backfilling and waterproofing and drainage facilities. The design on the basis of permanent structures reflects the design idea of "a hundred year project", and the design service life should meet the requirements of the *Technical Standard of Highway Engineering* (JTG B01). Routine maintenance and repairing of the tunnel is an important guarantee to enable the tunnel to meet the needs of long-term operations, so it is necessary to consider the convenience of maintenance and repair in operation.

1.0.6 Highway tunnel design is composed of support lining structure, waterproofing and drainage, pavement, as well as ventilation, lighting, power supply and distribution, disaster prevention and reduction, traffic monitoring and other operational facilities, which is an assembly of multiple disciplines. Therefore, it requires close cooperation and comprehensive analysis to enable the design to achieve the most optimal, comprehensive, economic and technological effects.

1.0.7 Different from ordinary structures, tunnels have very complex physical and mechanical properties in surrounding rock, and the tunnel design is greatly influenced by the topographic and geologic conditions and construction methods of the mountain being passed through. The surrounding rock is not only the load acting on the tunnel support structure, but also a part of the tunnel structure. Therefore, the geologic conditions form the basic premise for its correct design. However, it is difficult to obtain highly accurate geologic information before tunnel excavation under the current technical level and conditions. Therefore, on the one hand, it is required to use high level of technology in collaboration with experience judgement of the geologic conditions in the pre-design phase. On the other hand, during the construction and excavation phases, it is required to grasp the stress state of tunnel surrounding rock and support structure through constant field observation and measurement, and timely adjust support parameters and construction methods, thus apply observational design approach and information-based construction and make the design more reasonable.

1.0.8 China is densely populated with not much cultivated land and fragile ecology. Tunnels passing through mountains may change the storage conditions of groundwater, causing the loss of groundwater and surface water above the tunnels. For highway tunnel construction, it is required to utilize wasteland as much as possible to avoid occupying fertile farmland, pay attention to protect water conservancy facilities, protect the original vegetation as far as possible, and reduce groundwater loss. The waste cut generated by tunnel construction should be properly treated, and the waste water should be discharged after precipitation and purification as required.

2 Terminology and Symbols

2.1 Terminology

2.1.1 Generally, there are proper ventilation, lighting, traffic engineering and fire protection facilities in highway tunnels.

2.1.2 A mountain tunnel, as the name suggests, is a tunnel passing through a mountain, including tunnels passing through rock and soil.

2.1.9 The cross passage (transverse tunnel), by its form is divided into the cross passage connecting the two parallel tunnels, cross passage connecting the tunnel with the fan cavern and other caverns, and the cross passage connecting the tunnel with the ground. By its function it is divided into the vehicle cross passage, pedestrian cross passage, horizontal ventilation adit, construction cross passage, etc. The cross passage is generally approximately horizontal and the vertical slope is not large.

2.1.10 Vertical shaft is a vertical channel structure in the rock, which is generally used for ventilation during tunnel operation and seldom only used for tunnel construction operation.

2.1.11 Inclined shaft is an inclined channel structure in the rock, which is mainly used for ventilation during tunnel operation and temporary tunnel for tunnel construction operation. Inclined shaft generally has a slope of more than 12%. Inclined shaft with a slope of less than 12% is usually considered as a channel.

2.1.19 Loosening pressure refers to the load formed in this way, that is, the surrounding rocks above the tunnel become loose, and the rocks fall and slip under the action of gravity, so that their dead load directly acts on the tunnel lining structure.

2.1.22 Eccentric load in English is also called uneven pressure. It refers to the unsymmetrical load formed when the overburden of the upper tunnel is thin and inclined, or the stratum is inclined relative to the tunnel cross section, or the lithology is uneven, or the swelling stratum is subject to strong compression from one side.

2.1.31 Pilot tunnel in English is also called the drift. It refers to the smaller adit driven within the large tunnel face and is ahead of the tunnel face due to the large cross section of the main tunnel or poor surrounding rock conditions. According to the excavation location of the drift, there are crown drift, middle drift, invert drift, sidewall drift, etc.

3 Tunnel Survey and Classification of Surrounding Rock

3.1 General requirements

3.1.1 The survey data are the basis for the whole tunnel design for location selection, project layout, structural design, construction method, planned duration and project investment, etc. Therefore, the survey data should be complete and accurate.

3.1.2 The survey should be carried out in phases. The pre-construction phase includes three sub phases: feasibility site investigation, Preliminary site investigation and detailed site investigation. The survey in construction should throughout the whole process of construction and excavation, including geologic description, advance probing, etc.

3.1.4 The Specifications follow the provisions in the *Standard for Engineering Classification of Rock Masses* (GB/T 50218—2014), and the surrounding rock shall be classified using a comprehensive grading method combining qualitative analysis and quantitative calculated indices for surrounding rock, which is applied by most surrounding rock classification methods at home and abroad at present. Qualitative analysis and quantitative calculation can cross check and verify each other to improve the reliability of classification.

3.3 Topographical and geological survey

3.3.1 Different phases of tunnel survey have different objectives. The contents of the survey at all phases are basically the same. The scope of the survey is from the large extent of the feasibility study phase to the surrounding area along the alignment that affects the tunnel during the detailed site investigation phase. The depth of the survey includes the depth obtained during the feasibility study phase mainly via ground site reconnaissance, as well as the depth obtained during the detailed site investigation phase to meet the design, construction, preliminary design cost estimate and budget requirements by necessary investigation and exploration means and rock-soil physical and mechanical tests.

3.3.2 The maps and drawings provided at each phase shall meet the accuracy and tunnel design requirements stipulated in the *Code for Highway Engineering Geological Investigation* (JTG C20—2011) and the *Specifications for Highway Reconnaissance* (JTG C10—2007). For example: topographic maps and topographic cross-section maps with scales of 1∶5,000 ~ 1∶10,000 shall be collected and drawn during the feasibility study phase, and geological plans or engineering geological plans with scales of 1∶50,000 ~ 1∶200,000 shall be prepared based on survey data, collected regional geological maps and geological data. The following maps shall be drawn based on survey and exploration data in the preliminary site investigation and detailed site investigation phases: tunnel engineering geological plans with scales of 1∶500 ~ 1∶2,000; tunnel geological vertical section map with scales of 1∶500 ~ 1∶2,000 and the identical horizontal and vertical scales; vertical and transverse sections of portals with scales of 1∶500 ~ 1∶2,000 and the identical horizontal and vertical scales.

3.3.3 Due to the different complexity, scale, nature and natural geographical conditions of various geological problems, it is difficult to classify the basic contents of preliminary site investigation and detailed site investigation, which are often overlapped with each other in practice. Only the contents of the survey are put forward in the articles, and the key points of the survey and analysis should be arranged according to the actual situations.

3.3.4 Special survey should be made into some special geological environmental problems and precautions should be put forward. This is a special requirement for the key contents of the survey.

3.3.5 This Article specifies the contents of the geological survey during construction.

1 In the case of geological survey during construction, methods such as direct observation, geological sketch, photography and measurement of excavation tunnel face are generally adopted to verify the conditions of the exposed surrounding rock .

2 Additional ground survey, analysis of investigation data and survey on exposed surrounding rock should be used to analyze possible geologic conditions of surrounding rock ahead of the excavation. For tunnels with complex geologic conditions, seismic reflection, acoustic reflection, ground penetrating radar and other geophysical exploration means should be adopted, or probe holes, parallel heading and trial tunnel should be used for advance detection, so as to forecast the possible unfavorable geological phenomena.

For the advance geological forecast, the method of combining geological survey and geophysical exploration should be adopted. Meanwhile, the combination of medium and long distance forecast and short distance forecast, as well as physical exploration and geological drilling should be emphasized.

3.4 Meteorological survey

3.4.1 Extreme values are the maximum and minimum values of temperature, wind speed, rainfall and snowfall. Tunnel location should avoid areas with poor weather conditions.

3.4.2 For the tunnels in remote mountainous areas, due to the lack of meteorological data of the tunnel site area, meteorological observation points (stations) should be set up at tunnel portals and at inclined (vertical) shafts of extra-long tunnels to continuously collect local meteorological data, grasp the meteorological conditions of the tunnel site area, and provide basic data for rational use of natural wind in the operation period. At the same time, basic data is provided to prevent the damage that extreme weather may cause to the tunnel.

3.6 Surrounding rock classification

3.6.1 The quantitative classification of surrounding rock in the Specifications follows the class categories and methods stipulated in the *Standard for Engineering Classification of Rock Masses* (GB/T 50218—2014). The *Standard for Engineering Classification of Rock Masses* (GB/T 50218—2014) partially revised the *Standard for Engineering Classification of Rock Masses* (GB 50218—94).

3.6.2 There are hundreds of methods for quantitative classification of rock mass quality according to the quantitative indices of classification factors, which can be roughly summarized into the following three methods:

(1) Single parameter method, such as RQD method;

(2) Multiple parameters method. For example, the method of dynamic classification by computer program based on the four parameters of R_c, longitudinal elastic wave velocity of rock mass (v_{pm}), average joint spacing (d_p) and displacement stability time of surrounding rock, developed by Northeastern University (China);

(3) Comprehensive index method consisting of multiple parameters. Such as the surrounding rock classification of tunnel engineering adopted by the 4th Engineering Design and Research Academy, General Staff, PLA, the classification index is composed of four parameters, namely R_c, K_v, groundwater state and occurrence of rocks.

The Specifications use the multiple parameters method and takes the quantitative indices R_c and K_v of two classification factors as parameters to calculate the Rock Quality Designation BQ, as the quantitative basis of classification.

In Appendix A, the quantitative determination methods of R_c and K_v values are given in A.0.1 and A.0.2.

3.6.3 Hardness and integrity of rock are the basic properties of rock mass, which are the common characteristics of various types of rock engineering and reflect the basic features of rock mass quality, but they are not all factors affecting the stability of rock mass. When groundwater, high initial stress and unfavorable weak structure plane exist in the surrounding rock of tunnel, the stability will be reduced, which can be used as correction coefficients for surrounding rock classification.

Eq. (3.6.3) is the calculation equation for modified Rock Quality Designation, in which the determination of three correction coefficients K_1, K_2 and K_3 should be selected according to Appendix A.0.3. If a case is not listed in the table, the correction coefficient is zero.

3.6.4 The classification method of surrounding rock basically should be implemented under the methods and intentions in the *Standard for Engineering Classification of Rock Masses* (GB/T 50218—2014), which is based on the following considerations:

(1) The standard is formulated by China's water conservancy and hydropower departments together with railway, metallurgy, urban and rural construction and other relevant departments, as one of the national basic standards.

(2) This standard is the classification standard applicable to all industries in China, and it is beneficial to unify the classification methods and standards of engineering rock mass (or surrounding rock) in China.

(3) The surrounding rock classification standard uses the methods combining qualitative analysis and quantitative calculation, taking hardness and integrity of rock as two basic factors, and taking groundwater, structure plane occurrence and initial high in-situ stress state as correction coefficients. These classification methods and provisions are a

summary of most of the surrounding rock classification methods and provisions in China and have been accepted by a majority of the peers.

(4) The errors caused by qualitative classification can be reduced.

Table 3.6.4 is proposed after several amendments to Table 4.1.1 in the *Standard for Engineering Classification of Rock Masses* (GB/T 50218—2014). The structural state and soil characteristics are added in the column of "Main qualitative characteristics of surrounding rock or soil".

The classification of underground engineering rock mass in the *Standard for Engineering Classification of Rock Masses* (GB/T 50218—2014) is proposed for rock tunnel and other underground engineering, excluding the surrounding soil classification. In order to adapt to the actual situation and needs of highway tunnel, surrounding soil classification is introduced into Table 3.6.4. There is no unified standard for surrounding soil classification. The surrounding soil classification in Table 3.6.4 herein refers to the relevant contents in Table 3.2.7 in the Code for Design on Tunnel of Railway (TB 10003—2005). In the future practice, it is necessary to carry out specific research into the surrounding soil in order to put forward the surrounding soil classification that combines qualitative analysis and quantitative calculation.

Surrounding rock is classified into six classes, from good to poor are Class I, Class II, Class III, Class IV, Class V and Class VI respectively.

During the use of the former specifications, it was found that the classes of surrounding rocks determined by the qualitative and quantitative indices of [BQ] values were not consistent, and the quantitative classification by the [BQ] values was generally higher than the qualitative classification by "half a class". The cause of this phenomenon is mainly due to the calculation equation of Rock Quality Designation BQ being determined on the basis of the existing sampled population, and it has not yet or is impossible to completely cover all surrounding rock types for the highway tunnel; especially the index errors of Class IV and Class V surrounding rocks are larger. With the accumulation of experience and data during usage, the factors in the equation may be adjusted to some extent, but its mathematical pattern and classification can remain unchanged.

It is normal that the qualitative classification does not coincide with the quantitative classification. If necessary, the qualitative analysis shall be re-evaluated and the quantitative indices shall be rechecked. On this basis, the rock mass classification shall be redetermined through comprehensive analysis.

According to the current practical situation in China, in the feasibility study and preliminary site investigation (preliminary design) phase of tunnel engineering, the rock physical and mechanical test and elastic wave (acoustic wave) measurement are limited for medium and short tunnels or tunnel engineering below Class III, and the parameters for evaluating the surrounding Rock Quality Designation*BQ* are insufficient. Therefore, it is stipulated in this Article that, when the above situation occurs, the determination of surrounding rock class can be qualitatively classified as the main basis, or be classified by the engineering analogy methods.

A.0.5 and A.0.6 herein give the qualitative classification method of hardness and integrity of rock, respectively.

4 Overall Design

4.1 General requirements

4.1.1 Guided by the overall highway design principle, tunnel design shall meet overall functional requirements for the highway, control tunnel size, reasonably utilize land resource and protect the eco-environment.

4.1.3

1 This revision has deleted the specification that "in poor geological conditions, the location of extra-long tunnel shall dictate route alignment" in the previous edition. The current technical standard for highway engineering emphasizes comprehensive design of route, bridge and tunnel rather than which dictates which. Expressway and Class-1 highway in recent years have generally met overall route requirements on the basis of comprehensive design.

2 In selecting tunnel solution, complex topographic, geological and weather conditions, cultural conditions, environmental conditions, social and economic development and traffic conditions along the route will be encountered. On the basis of investigation and survey data, a recommended solution shall be proposed after comprehensive comparison of multiple options.

3 The internal profile of clear section shall take into account structure stress in addition to satisfying traffic operation clearance requirement and space required for various facilities in the tunnel.

5 The ventilation method and the size of lighting, traffic monitoring, disaster prevention and rescue and other facilities in a highway tunnel shall be considered in overall design.

Specific design for disaster prevention shall be developed for extra-long tunnel when necessary.

6 Tunnel excavation may change groundwater storage conditions and groundwater path; tunnel muck and sewage may cause pollution to the environment. Measures shall be taken to prevent and minimize damage and pollution to the environment. Therefore, this provision requires comprehensive consideration of the above issues in overall design.

8 To reduce consumption of energy and carbon emissions is the goal of tunnel energy conservation design. Tunnel maintenance and repair are regular and long-term work during tunnel operation; and shall be designed so that this work is easy to be done.

4.2 Tunnel siting

4.2.1 Geological conditions of highway tunnel construction is one of key factors directly influencing project cost and duration. Good geological conditions lead to less project investment and faster construction whereas poor geological conditions result in more investment and slower construction, with an impact on the construction of the whole highway. Therefore, based on the design guideline of selecting route according to geological conditions, the tunnel shall be located in a stable stratum to avoid or reduce the length in unfavorable geological zones.

When passing through a massive mountain, the tunnel will encounter relatively good geological conditions. When passing directly through a low-lying area between two mountains (pass) into the mountain, the tunnel is short but will typically encounter poor geological conditions and high groundwater inflow. In general, passing through a pass into the mountain shall be avoided. In China there are many cases of a tunnel passing through a pass into the mountain that have encountered poor geological conditions and high volumes of groundwater, cave-in during construction and suffered cost increase and construction delay and higher probability of water leakage after the tunnel is completed. Therefore, a tunnel shall not pass directly through a pass into the mountain wherever possible. While meeting overall route requirements, it is sometimes worthwhile to bypass undesirable zones or increase tunnel length as appropriate to pass through bettergeological conditions. If this is impossible, reliable technical measures shall be taken to ensure safe completion of the tunnel.

4.2.2 Mountain tunnels generally face undulating and complex terrain and geology with highly variable physical conditions. Mountain height, thickness, mountain slope steepness, gully and tableland terrain on both sides of the mountain and distribution of main and branch gullies have a huge influence on mountain tunnel proposal, tunnel length, route development conditions, etc. Therefore this provision specifies: "the alignment and layout plan of the tunnel shall be determined based on large-area geologic mapping and comprehensive geological exploration."

4.2.3 On a downcut valleys, the river usually meanders along developed valley, accompanied by densely distributed branch gullies and frequent collapse, falling materials, talus, landslide, debris flow and bank erosion, with symmetrical or unsymmetrical tableland and steep mountain slope on the banks of the valley. When the tunnel runs along river and mountain, insufficient wall thickness on the free side, eccentric load, shallow embedment, high side slope at the portal, current scour hazard, passing through unfavorable geological zones, etc. shall be avoided; many short tunnels or bridge-tunnel connection, increase in the length and height of bridge works and in the number of retaining structures shall also be avoided. Construction site when the tunnel runs along river and mountain is narrow and construction is difficult; short tunnel groups require more construction access roads and tend to do more damages to the valley environment.

4.2.4 The improper location of the portal would result in difficult entry into the tunnel, high side slope at the portal, portal collapse, and even threaten construction safety, hinder construction progress and jeopardize long-term operation safety. Therefore, tunnel portals should avoid severely adverse geological zones and unfavorable terrain.

4.2.5 For a tunnel near reservoir, along the river or creek, the tunnel shall not be flooded or pavement submerged during flood period due to variation in water levels. When the tunnel structure safety is jeopardized by mountain collapse and landslide due to flood scouring and prolonged soaking, reliable engineering measures shall be taken to ensure tunnel structural and operational safety.

4.2.6 This provision has presented the flood frequency standard of tunnel design water level by reference to the current *General Specifications for Design of Highway Bridges and Culverts* (JTG D60). Flood observation shall include survey of reliable historical flood that may recur.

4.3 Tunnel alignment design

4.3.1 Whether the horizontal alignment of tunnel shall be straight or curved is not specified in this provision. According to the *Code for Design of Highway Route* (JTG D20—2017), "the highway shall be widened when the radius of circular curve is less than or equal to 250m" whereas highway tunnel should not use widened circular curve. Therefore, this provision specifies "If the design speed is 20km/h, the radius of the circular curve should not be less than 250m." Uniform internal contour cross-section shall be adopted in the tunnel to facilitate construction.

4.3.2 Separated tunnels involve simple structural design, and less mutual interference between two tunnels during construction. Their construction is fast and the cost is low.

1 Twin tunnels with small clearance involves long construction duration and high cost. The structures need strengthening. The construction of the left and right tunnels shall be staggered. They are adopted infrequently.

2 Twin arch tunnels involve complex structures, many construction procedures, long duration and high cost. Presently there are still some problems yet to be solved. Once completed, they suffer various distresses that are difficult to control. It is considered as the last resort only in special zones where the space is narrow, alignment design is difficult or tunnel entrance/exit is restricted by large structures/buildings.

3 For bridge-tunnel connection, narrow terrain at portals, long or extra-long tunnels with special requirements and portals of twin tunnels with small clearance or twin arch tunnels while the main tunnels are separated to reduce the adverse mutual interference of the two tunnels, the portal transition section may be arranged as branching-out tunnels where the twin-arch or twin tunnels with small clearance transit into separated tunnels.

4.3.3 The clear distance between two separated tunnels refers to the thickness of unexcavated rock mass between the two tunnels. If the clear distance is too large, land occupation of the route outside the tunnel will increase, resulting in many artificial side slopes in narrow terrain sections. The length of cross passage if any will increase, leading to higher cost and inconvenient management. If the clear distance is too short, forming twin tunnels with small clearance, the structure between the two tunnels and their construction will be impacted, leading to slow progress of construction and higher cost.

This revision has deleted the table of "minimumclear distance between separated two tunnels" in the *Specifications for Design of Highway Tunnel* (JTG D70—2004), as shown in Table 4-1, for the following reasons:

Table 4-1 Minimum clear distance between separated two tunnels

Surrounding rock class	I	II	III	IV	V	VI
Clear distance (m)	$1.0\times B$	$1.5\times B$	$2.0\times B$	$2.5\times B$	$3.5\times B$	$4.0\times B$

Note: B—width of cross-section of tunnel excavation

A tunnel is a linear structure that usually crosses surrounding rocks of several classes. Compliance with the specified distance between two tunnels in Table 4-1 can avoid mutual interference between the two tunnels, but the route is difficult to develop, resulting in huge waste. In fact since the beginning the 21st century, the distance between two parallel tunnels of expressway in China has become increasingly smaller. It is common for this distance to be 8-20m for a two-lane tunnel in Classes IV and V surrounding rocks. There is certain mutual influence with intersecting rock stress influence zones between the two tubes, but this influence is limited and controllable provided construction excavation and support sequences are restricted accordingly. Although the clear distance between two parallel tunnels is less than the value listed in Table 4-1, the design is mostly considered as separated tunnels.

4.3.4 The tunnel longitudinal gradient may be unidirectional or bidirectional. Both cases are seen in China and there is no uniform specification in this regard. From the standpoint of ride comfort and operational ventilation efficiency, unidirectional gradient is better though reverse gradient drainage may happen during construction. According to survey of construction contractors, water pumping performance and dewatering technology have advanced considerably in recent years. Therefore, reverse gradient drainage is not technically challenging. For long and extra-long tunnels with developed groundwater, bidirectional gradient can make drainage during construction easier and relieve drainage pressure in the tunnel during operation by allowing groundwater to drain toward two portals. When bidirectional gradient is adopted, its vertical curve radius shall be as large as practically possible to increase traffic safety and comfort and ensure clear line of sight.

However, it should be noted that drainage gradient at local zone near the bidirectional inflection point will be less than 0.3% and not beneficial to drainage. Therefore, the bidirectional inflection point shall avoid groundwater rich zones wherever possible.

4.3.5 Minimum longitudinal gradient in the tunnel shall be 0.3% so that after completion of the tunnel, water in the tunnel (including seepage water, inflow water, washing water and fire-fighting water) can be discharged by gravity. This gradient shall be 0.5% for long and extra-long tunnels where drainage distance in the tunnel is longer and drainage volume higher. Maximum longitudinal gradient in the tunnel shall take into account traffic safety, ride comfort and ventilation requirements during operation.

According to experience in construction of mountain roads restricted by terrain in recent years, if the longitudinal gradient of tunnel is required to be not greater than 3%, route development will be very difficult and route length will be increased. Consequently, this limiting value may be eased for medium and short tunnels or independent cut-and-cover tunnels (including shed tunnel structure) of expressway and Class-1 highway where alignment arrangement is very difficult, provided more operational safety measures are taken.

Such measures include installing warning signs, speed limit signs and speed bump, improving pavement anti-slip conditions and increasing the number of lanes in upgrade tunnel.

4.3.6 Due to abrupt changes in lighting, roadway width and environment at tunnel portals, accidents tend to occur at tunnel entrance/exit. Therefore, it is necessary to require horizontal and vertical alignments within a certain length inside and outside the tunnel be coordinated. The alignment of connection between the inside and outside of the tunnel has been adjusted according to the *Technical Standard for Highway Engineering* (JTG B01—2014).

Travelling lengths at design speeds are given in Table 4-2.

Table 4-2 Travelling lengths at design speeds (m)

Design speed (km/h)		120	100	80	60	40	30	20
Travelling length	3s	100	83	67	50	33	25	17
	4s	133	111	89	67	44	33	22
	5s	167	139	111	83	55	42	28

4.3.7 In construction of mountain roads, there have been some continuous tunnels with the distance between portals of two tunnels less than 100m; in this case, the two tunnels may be deemed to be one tunnel and the technical specifications of horizontal and vertical alignments shall be considered as if they are one tunnel.

4.4 Clear section design

4.4.1 The highway tunnel construction gauge not only provides the space for vehicle travel but also shall take into account traffic safety, convenience, comfort and disaster prevention. Therefore, the spatial relationship between lanes and highway features shall be studied fully and no civil structural elements shall encroach into the tunnel construction gauge. The construction gauge placed in tunnel internal profile is as shown in Fig. 4-1.

The tunnel construction gauge consists of roadway width W, lateral width L (L_L or L_R), width allowance C, maintenance access J or sidewalk R. This revision has made adjustments based on the Technical Standard for Highway Engineering (JTG B01—2014) and Code for Design of Highway Route (JTG D20—2017).

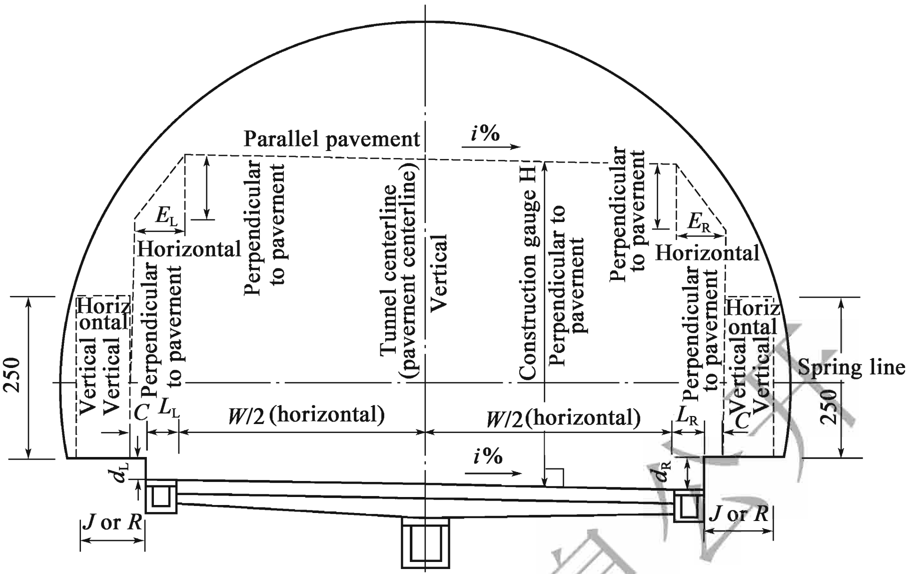


Fig. 4-1 Basic configuration of tunnel construction gauge (dimensions given in cm)

The width of top left corner of construction gauge, E_L, shall include width allowance C and measured from the starting point of top corner oblique line on the left of C toward the middle.

The width of top right corner of construction gauge, E_R, shall include width allowance C and measured from the starting point of top corner oblique line on the right of C toward the middle.

The lateral width L_L on the left side of expressway and Class-1 highway tunnel with 100km/h design speed has been changed from 0.5m to 0.75m.

4.4.2 The main purpose of maintenance access is to allow maintenance personnel to patrol and carry out general inspection and repair while the tunnel is in normal operation.

Where the left side of twin-arch tunnel in the traffic flow direction and Class-4 highway tunnel are not provided with maintenance access or sidewalk, a width allowance not less than 0.25m shall be provided to eliminate or minimize the psychological effect of fear induced by tunnel side walls on motorists (side wall effect) so as to ensure safe passage at certain speeds.

The maintenance access or sidewalk shall be higher than pavement such as to stop vehicles from climbing to the maintenance access or sidewalk and to ensure the safety of maintenance personnel or pedestrians. The curb of maintenance access or sidewalk is conspicuous and can better attract the attention of motorists than the lane side line; it may serve as travelling guidance line for motorists. The space under the maintenance access or sidewalk may be used for laying various pipelines and cables.

The height of maintenance access or sidewalk is dependent on tunnel length, pedestrian density in the tunnel area and space required to lay various pipelines. Based on investigation results, this revision has changed maintenance access height from "20cm ~ 80cm" to "250mm ~ 800mm".

4.4.3 The highway clear section clearance is a basic condition for the tunnel to fulfill its functions and of a size suited to the highway class. In addition to providing the space required for tunnel construction gauge, it shall also be able to accommodate in-tunnel drainage, ventilation, lighting, fire protection, monitoring, interior decoration, traffic works and auxiliary facilities, and some allowance. The minimum distance between tunnel internal contour and construction gauge shall be 50mm, as shown in Fig. 4-2.

Based on years of engineering practice and internal force analyses, to better carry loads, the tunnel internal contour shall consist of single arc tunnel profile or triple-arc tunnel profile or tunnel section and large-diameter arc shaped side walls. Appendix B to the Specifications gives reference drawings of tunnel construction gauge and internal contour for various classes of highway.

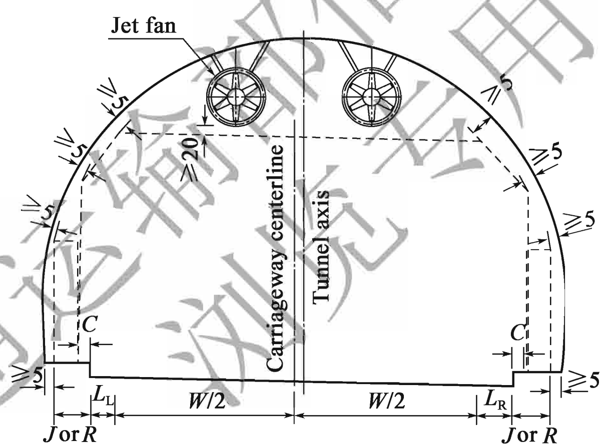


Fig. 4-2 Allowance required for tunnel internal contour (dimensions given in cm)

4.4.4 Side drain are arranged on both sides of roadway to facilitate separate discharge of clean and dirty water in the tunnel. In particular, when there is seepage water from tunnel linings, the side drain can collect and intercept the water so that it will not flow to the pavement.

4.4.5 The emergency stop zone is intended mainly to accommodate malfunctional vehicles and maintenance vehicles and allow rescue personnel to carry out rescue operations in an emergency. The emergency stop zone shall be provided for a two-lane tunnel. The emergency stop zone should be provided for a three-lane tunnel depending on tunnel length, traffic volume, traffic composition and surrounding rock conditions. The emergency stop zone is not mandatory for a four-lane tunnel due to its large cross-section of each tube, construction difficulty and high construction cost and good traffic conditions.

4.4.6 The width, length and spacing of emergency stop zones are specified according to China's traffic realities.

1 A distance of L_R is reserved between the emergency stop zone and carriageway to ensure traffic safety. Side drain are arranged along the maintenance access to avoid vehicles rolling on the side drain. This revision has amended the arrangement of emergency stop zone. Fig. 4-3 gives the cross-section arrangement of an emergency stop zone.

2 The *Specifications for Design of Highway Tunnel* (JTG D70—2004) recommends the length of an emergency stop zone as 40m by reference to the recommended value of the PIARC Committee on Road Tunnels (C5) and specified values of Japan and other countries. With increasing numbers of long vehicles in China in recent years, the length of the parking strip needs to be increased for parking of long vehicles. This revision has changed the length of emergency stop zone to 50m and its effective length to 40m in accordance with the *Technical Standard for Highway Engineering* (JTG B01—2014).

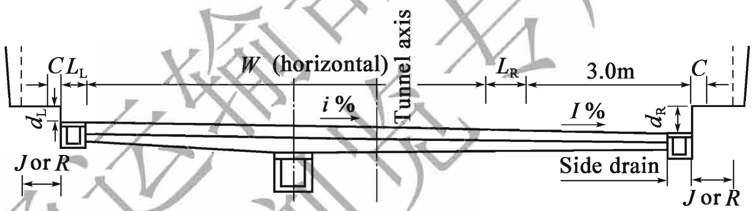


Fig. 4-3 Transverse arrangement of emergency stop zone

4.4.7 The emergency stop zone in the tunnel is an effective refuge place for pedestrians. In a tunnel with emergency stop zone, the spacing of refugees needs to take account of the refuge role of the emergency stop zone.

4.4.8 This provision has been added in accordance with Sub-clause 5 under Clause 8.0.3 of the *Technical Standard for Highway Engineering* (JTG B01—2014). An independent cut-and-cover tunnel or shed tunnel is a cut-and-cover tunnel or shed tunnel whose portals are far away from the portal of another tunnel.

4.4.9 This provision is in line with the *Technical Standard for Highway Engineering* (JTG B01—2014). Under normal circumstances, the tunnel has a construction gauge different from that of link line subgrade outside the tunnel. This sectional variation creates a passage bottleneck that affects traffic capacity and service level. Therefore, civil engineering measures are needed to address the smooth transition between subgrade and in-tunnel pavement widths. For example, where the tunnel construction gauge width is greater than the highway construction gauge width, subgrade in the transition section is widened to the tunnel width; where the tunnel construction gauge width is less than the highway construction gauge width, carriageway width in transition section is designed to highway standard to accommodate smooth transition of cross-sections taking into account traffic signs, markings, warning boards and guard rails so that motorists can adapt to changes in driving environment.

4.5 Cross passage and parallel adit

4.5.1 Pedestrian cross passages connecting two tunnels in parallel are intended to allow drivers and passengers to escape in an emergency, allow rescue personnel to quickly reach accident scene and provide easy access to tunnel maintenance personnel. The spacing of pedestrian cross passages has been changed from "not more than 500" in the Specifications for Design of Highway Tunnel (JTG D70—2004) to "shall not exceed 350m" because pedestrian cross passages spaced too far apart require people to walk a long distance to escape. The vehicular cross passages allow rescue vehicle to reach the nearest accident scene when one of the tubes is congested. The specific spacing of cross passages is dependent on tunnel length, surrounding rock conditions, traffic volume and in-tunnel facility layout, subject to adjustments within an appropriate range.

This revision has added a sub-clause: "The curb height of a vehicle cross passage, d, should be consistent with the height of the maintenance access on the left side of travel direction in the tunnel."

4.5.2 For a bidirectional single-tube extra-long tunnel, temporary refuge facilities for drivers and passengers in an emergency need to be considered. One such facility is a parallel adit connected to main tunnel by a cross passage. Temporary refuge facilities that have been built at home and abroad include: parallel adit, temporary refuge caverns at certain intervals, detour channel, cross passage leading directly to ground surface where topographic conditions allow as well as some guidance signages and traffic safety furniture. "Should provide" in this provision has been specified based on the experience from several bidirectional single-tube two-lane extra-long tunnels already completed in China. On the other hand, parallel adit in an extra-long tunnel is costly to construct and operate and difficult for maintenance management; discretionary consideration needs to be given to parallel adit for a tunnel with very low design traffic volume in a sparsely populated area where escape and rescue are relatively easy. No experimental data are available on the size and spacing of refuge facilities such as temporary refugees and detour channels. These issues need further study. The form of refuge facilities shall be determined according to local conditions, topographic and geologic conditions, vehicular traffic volume, human traffic volume, maintenance management conditions, etc.

Parallel adit shall be designed as specified below:

(1) The parallel adit shall be as close to main tunnel as possible to minimize the length of connecting cross passage.

(2) The parallel adit shall be located on the side of main tunnel with higher water volume, with its drainage invert elevation below the drainage invert of the main tunnel to facilitate main tunnel groundwater drainage through the parallel adit.

4.5.3 A cross passage leading to the ground surface can reduce escape distance and serve as ventilation channel at normal times to improve ventilation efficiency.

4.5.4 For long and extra-long twin tunnels, a connecting passage outside the portal is intended for tunnel maintenance, repair and vehicle turnover in emergencies.

4.6 Tunnel construction monitoring and measurement and advance geological forecast

4.6.1 Tunnels are buried underground where engineering geological and hydrogeological conditions are complex and variable. At investigation stage, it is very difficult to reveal geological conditions through which the tunnel passes, so tunnel support design assumptions may deviate from actual conditions. To better match tunnel support design with actual conditions and prevent personal injury and economic loss caused by geological disasters, tunnel construction monitoring and measurements are required during construction. For tunnels in complex geological conditions, advance geological forecast is required during construction.

4.6.2 Monitoring purpose, content and requirements are the basic requirements of tunnel construction monitoring and measurement and advance geological forecast during construction. The information obtained shall reflect the actual state of surrounding rock and tunnel lining.

4.6.3 Due to uncertainty of geological conditions encountered in tunneling project, geological survey data collected prior to tunnel construction need to be verified and adjusted in the construction process. Therefore, tunnel support parameters and construction method shall be adjusted in a timely manner during construction based on the geological conditions, surrounding rock, support stress and deformation conditions revealed so that the lining structure design and construction method are more suitable for actual surrounding rock conditions.

4.7 Construction plan

4.7.1 The construction plan referred herein is not the specific construction planning of the construction contractor, but considerations impacting the design.

1 The tunnel construction method referred herein is a collective term for excavation method, support means, in-tunnel haulage method, auxiliary engineering measures, ventilation method, etc. Excavation method includes full face heading, bench-cut method, drift excavation and sequential excavation. Support means include rockbolt, shotcrete, steel support, reinforcement mesh, member support and cast-in-situ concrete lining. The cast-in-situ concrete lining support may be of full face excavation or segmental type. In-tunnel haulage may be with or without track. Auxiliary engineering measures refer mainly to advance rockbolt, tremie, pipe roofing, chemical grouting, freezing, cement grouting, etc. for surrounding rock reinforcement and groundwater control.

2 For division of tunnel construction areas, the reasonable length and division point of construction sections shall depend on tunnel cross-section, longitudinal gradient, surrounding rock geological conditions, hydrogeological conditions, muck and cut-and-fill balance.

3 In general auxiliary channel is provided only for an extra-long tunnel. Design of the auxiliary channel shall take into account its other purposes after tunnel civil works are completed, such as serving as ventilation channel during operation or refuge channel. A multi-purpose auxiliary channel is economical.

4 Main machinery and equipment in the tunnel include: TBM, drifter, pneumatic drill, jumbo, pneumatic drill, shovel, dump truck, concrete truck, shotcrete machine, rockbolt drilling machine, full profile lining formwork trolley and water pump.

5 Temporary facilities outside the tunnel mainly include: explosive storage, building material storage yard, substation, air compressor house, material and member processing shop, concrete batching plant, laboratories, production facilities and amenities, temporary muck and sewage treatment plant. Temporary storage yard for tunnel muck shall be planned in advance considering the location and capacity of permanent muck area. The sewage treatment plant shall be arranged such that it can effectively collect sewage from tunneling and discharge the treated water. Existing roads shall be used as construction access roads wherever possible. Specific design is required for large-scale construction access roads.

Fig. 4-4 gives an example layout of large temporary facilities outside the tunnel.

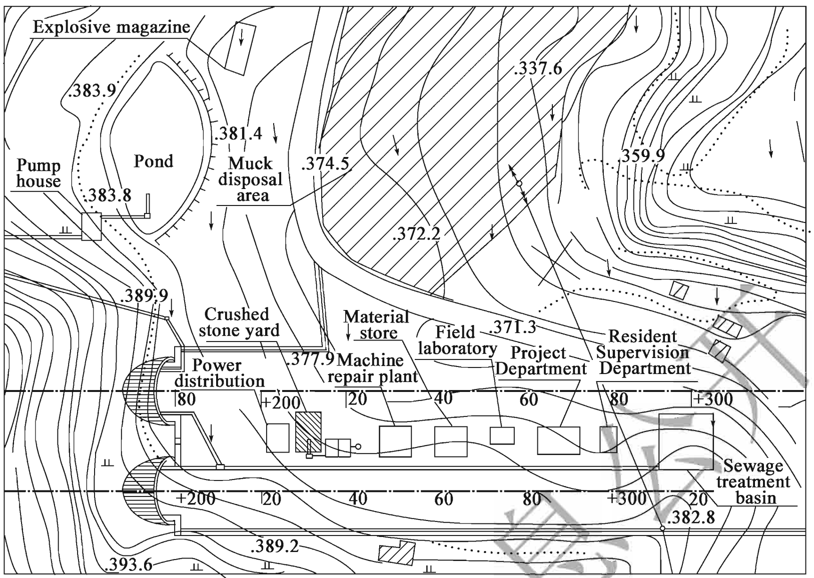


Fig. 4-4 Example layout of temporary facilities at a tunnel portal

5 Building Materials

5.1 General requirements

5.1.1 As specified in ISO 3893, the concrete grades are described as the strength grades of concrete (denoted by C), and the standard size of a concrete test specimen shall be a cube with a side length of 150mm.

1 The strength grade of concrete shall be determined by the standard compressive strength of cubes, which is the basic representative value for mechanical indices of concrete in the Specifications.

2 The strength grades of stones shall be determined by using cubes with a side length of 70mm as standard test blocks. The strength conversion coefficients for test blocks of other sizes are listed in Table 5-1. The strength grades of stones are represented by the saturated ultimate compressive strength of their standard test specimens.

Table 5-1 Conversion coefficients for stone strength grades

Side length of cube (mm)	200	150	100	70	50
Conversion coefficient	1.43	1.28	1.14	1.00	0.86

3 The strength grade of cement mortar shall be the same as that in the current *Code for Design of Masonry Structures* (GB 50003).

As C10 (strength grade of concrete) and M5 (strength grade of cement mortar) have not been adopted in recent years of tunnel design, they are canceled in this revision.

5.1.3 1 "They shall have the required structural strength and durability" is the basic requirement the building materials must meet. If a tunnel is built in a particular region or occasion, such as in a severe cold region, coal measure strata, saliferous strata and strata with aggressive water, the lining materials selected and used for this tunnel shall also have properties required to adapt to these particular conditions.

2 For tunnel construction in surrounding rocks with aggressive groundwater, some components of concrete will react with acid, alkaline and salt in water. Consequently, concrete will be corroded, thus seriously affecting the strength and safety of lining concrete. Hence the provision emphasizes that the tunnel lining concrete shall be made of erosion-resistant cement and aggregate where there is aggressive water.

3 In cold regions, the tunnel lining is often exposed to frost, and its surface erosion is more serious than in general regions under the cyclic action of freeze-thaw when the temperature is low and varies widely from day to night. Hence the provision specifies that "the strength grade of concrete shall be increased properly".

5.1.4

1 To ensure the quality of concrete, the provision specifies that "Alkali-reactive aggregate shall not be used for concrete batching".

4 Stone used for masonry shall meet the basic requirements of lining materials. The strength grade of stone is an important symbol of stone quality and can reflect other properties of stone, for instance, stone with lower strength is susceptible to weathering and has poor durability and low resistance to freezing and seepage.

5.1.5 1 Portland cement is preferred to make shotcrete as it contains much C_3A and C_3S and has short setting time and in particular good compatibility with accelerator.

2 To reduce rebound and pipeline blockage, the provision specifies that the particle size of coarse aggregate should not be greater than 16mm. It specifies that fine aggregate shall be composed of medium or coarse sand and should have a fineness modulus of more than 2.5. This is not only to ensure there is enough cement to cover fine aggregates but also to help obtain enough concrete strength. Moreover, this can reduce dust and the shrinkage of hardened concrete. The moisture content in sand should be controlled within a range of "5% to 7%" in order to reduce the loss of active cement particles, reduce dust, and facilitate full hydration of cement.

3 The mortar rockbolt should be made of HRB400 or HRB500 steel in accordance with the current *Steel for the Reinforcement of Concrete—Part 2: Hot Rolled Ribbed Bars* (GB 1499.2).

4 It specifies that the elongation at break (A) of hollow rockbolt shall not be smaller than 16% in order to ensure the rockbolt has certain ductility and to prevent brittle fracture. Refer to the provisions in the *Technical Regulations for Hollow Bolt* (TB/T 3209—2008).

5 The combination hollow rock bolts are composed of hollow bolts and rebars. The bolt materials shall conform to 3 and 4 of this clause.

5.1.6 To improve the properties of cast-in-situ concrete and shotcrete, it is a common means at present to add proper additives to concrete. The use of additives shall not have adverse impact on the original properties of concrete.

5.1.7 The contents shall be determined in accordance with the *Specification for Design and Construction of Steel Fiber Reinforced Concrete Structures* [the *Technical Specification for Fiber Reinforced Concrete Structures* (CECS38:2004)] and the development of relevant materials.

5.1.8 Steel lattice girders assembled and welded from rebars have been widely applied in engineering as they have such advantages as good bearing capacity, small mass, adjustable stiffness, saving steel, easy to fabricate and install, and good integrity for shotcreting. Structural steel rib supports are made of I-structural steel, H-structural steel, U-structural steel, steel rail or steel tube through cold bending and welding, among which, I-structural steel rib supports have been used more than others.

5.1.9 Pavement materials in tunnels are mainly determined based on the pavement grade and the type of surface course. For their use, especially the property indexes of relevant materials, refer to relevant specifications.

5.1.11 Common grouting materials include single liquid cement grout, cement-sodium silicate grout, ultrafine cement grout, soluble polyurethane grout and acrylate grout. They shall be selected and used through overall consideration of the geological environment of surrounding rocks, the groundwater environment and the construction cost of a tunnel.

5.1.12 High polymer modified bitumen type or synthetic polymer type waterproofing layers, such as EVA, ECB, PE (including HDPE and LDPE), are widely used in highway tunnels. Pre-applied fully bonded type (often called as self-adhesive type) membrane is a new type of waterproofing layer emerging in recent years, which has been gradually used in tunnel works due to its simplicity in construction and its unique waterproof property including preventing water channeling between structures and membranes.

There are two main types of pre-applied fully bonded membranes as follows:

(1) Waterproofing layer with synthetic polymer sheets as the base film and one side is coated with high polymer modified bitumen of self-adhesive materials type.

(2) Waterproofing layer with synthetic polymer sheets as the base film and one side is coated with self-adhesive polymer film (polymer resin plastogel layer). The self-adhesive polymer film reacts physically and chemically with the lining concrete that is cast later to generate a layer of plastogel sticking onto concrete surface.

5.1.13 Relevant studies at home and abroad indicate that the fiber made of polypropylene through antioxidant treatment can be used for a long time in any acid and alkali conditions. So, from the perspective of the long service life of a tunnel, geotextiles made of polypropylene are recommended.

5.1.14 A drainage pipe shall be such that the water seepage from the back of lining can rapidly enter it and that it will not be squashed down by concrete mixture during placing of concrete to ensure its drainage section is effective.

5.2 Material properties

5.2.2, 5.2.5 The values of the standard strength and the modulus of elasticity of concrete are quoted from the *Code for Design of Concrete Structures* (GB 50010—2010).

5.2.6 The standard test specimen of shotcrete is generally fabricated by slab cutting method. If a non-standard method is adopted, the strength can be obtained by conducting comparative tests in which the raw materials, mix proportion, process/technology and curing conditions for the works are the same. Values in Tables 5.2.6-1 and 5.2.6-2 are quoted from the current *Technical Code for Engineering of Ground Anchorages and Shotcrete Support* (GB 50086).

5.2.7 For the ultimate strength of shotcrete, its ultimate tensile strength is reduced properly considering that the ratio of tensile strength to compressive strength of shotcrete is smaller than that of moulded concrete, by reference to the current *Code for Design of Concrete Structures* (GB 50010) and *Technical Code for Engineering of Ground Anchorages and Shotcrete Support* (GB 50086).

5.2.9 For hot-rolled rebar with an obvious physical yield limit, its standard strength shall be subject to the yield strength as provided in the relevant Chinese national standard. For rebar without an obvious physical yield limit, its standard strength shall be subject to the tensile strength as specified in the relevant Chinese national standard. All standard values shall have a guarantee rate of not less than 95%. The design strength of rebar is also determined through comprehensive analysis by analyzing the structural reliability index and calibrating it with engineering experience. In the Specifications, the design strength of rebar is given in integers, and the partial factor, γ_s, for hot-rolled rebar (HPB300) is 1.25.

5.2.10 The provision on total elongation of rebar under the maximum force defines the requirements for the rebar ductility and reflected the uniform strain under the maximum force before the rebar breaks. The total elongation is not affected by fracture (necking).

5.2.11 The standard yield strength and the standard ultimate strength of rebar are taken from Table 4.2.-1 of the *Code for Design of Concrete Structures* (GB 50010-2010). The standard tensile or compressive strength, R_g, of rebar is the standard yield strength.

5.2.13 For the permissible stresses on stone and concrete block masonries under axial and eccentrically compressed, this provision is quoted from the current *Code for Design of Railway Tunnel* (TB 10003). Measured data, if available, shall prevail.

5.2.14 This provision is quoted from the current *Code for Design of Masonry Structures* (GB 5003) and *Code for Design of Highway Masonry Bridges and Culverts* (JTG D61). For description about masonry performance, see the *Code for Design of Masonry Structures* (GB 5003).

5.3 Properties of waterproof and drainage materials

5.3.1 The technical specifications of common waterproofing layers are formulated by reference to the current *Polymer Water-proof Materials—Part 1: Water-proof Sheet* (GB 18173.1), *Technical Code for Waterproofing of Underground Works* (GB 50108) and *Water-proof Materials Used in Railway Tunnel—Part 1: Water-proof Slab* (TB/T 3360.1) and in combination with the practice in highway railway engineering. The test methods for relevant parameters are given in the current *Polymer Water-proof Materials* (GB 18173) and other associated codes.

The technical specifications of pre-applied fully bonded waterproofing layers are obtained by reference to the current*Pre-applied and Wet Installed Waterproof Sheets* (GB/T 23457).

5.3.2 Specifications in this provision are obtained by reference to the current *Geosynthetics—Synthetic Filament Spunbond and Needle-punched Nonwoven Geotextiles* (GB/T 17639). The elongation at break, equivalent aperture O_{90} (O_{95}) and vertical permeability are increased properly considering the different purpose and function of nonwoven geotextiles in tunnel.

5.3.3 The specifications in this provision are obtained by reference to the *Polymer Water-proof Materials—Part 2: Waterstop* (GB 18173.2) and *Water-proof Materials Used in Railway Tunnel—Part 2: Waterstop* (TB/T 3360.2). The test method for rubber-metal bonding with steel-edged waterstop is given in the current *Rubber, Vulcanized—Method for Determination of Strength Properties of Adhesive to Metal in Shear by Tension Loading* (GB/T 13936). Rubber damage means the damage of rubber prior to the adhesion of rubber to metal in the tensile shear test of steel-edged waterstop.

5.3.5 With regard to circumferential and longitudinal drainage pipes made of other materials, refer to relevant Chinese national codes for their use and the inspection of their performance.

6 Load

6.1 General requirements

6.1.1 This provision specifies the classification of loads acting on tunnel structures.

(1) Permanent load means the load acting on structure that does not vary with time.

(2) Variable load means the load acting on structure that varies with time.

(3) Accidental load means the load acting on structure for a short time.

(4) Superimposed dead load of structure means the constant weight of equipment and facilities for tunnel operation.

(5) Water pressure needs to be considered in tunnel structures without drainage measures in watery or water-bearing strata.

(6) Construction load means some applied forces in construction phase, such as the dead weight of mechanical equipment and the loads from crowd, temperature action, hangers or other machine and tools, as well as the temporary load on members during fabrication, delivery and hoisting of members. Such loads are temporary loads likely to occur during tunnel construction.

Considering that the current statistical analysis and study of various actions on highway tunnel structures are not comprehensive and thorough enough, the Specifications do not follow reliability design principle for the structures in specifying the actions on highway tunnel structures. Further in-depth study needs to be made to improve the reliability design.

6.1.2 Since the rock load is not only directly related to surrounding rock conditions but also connected with construction methods, support time and support stiffness, there is a lot of uncertainty. At present, the rock load is still calculated by empirical equation in most cases. For tunnels with complicated geology, the calculated value and distribution rule of loads need to be determined through field measurement, in order to understand and master the nature, size and distribution of tunnel loads.

6.1.3 When the loads that may act on tunnel structures simultaneously are combined according to bearing capacity requirements, the basic combination and accidental combination are the main considerations:

(1) Basic combinations of loads

Combination 1: Permanent loads: rock load + dead weight of structure + superimposed dead load.

Combination 2:

①Permanent loads + basic variable loads:

Dead weight of structure e + superimposed dead load + soil pressure + highway load;

②Permanent loads + basic variable loads:

Dead weight of structure e + superimposed dead load + soil pressure + live load of train;

③Permanent loads + basic variable loads:

Dead weight of structure + superimposed dead load + soil pressure + flowing water pressure.

Combination 3: Permanent loads + other variable loads:

Rock load (soil pressure) + dead weight of structure + superimposed dead load + construction load + temperature loading.

(2) Accidental combination of loads

Combination 4: Permanent loads + accidental load:

Rock load (soil pressure) + dead weight of structure + superimposed dead load + seismic action or impact loading of falling rock.

When such loads are combined according to normal service requirements, the combinations for long-term and short-term effects are the main considerations:

(3) Combination for long-term effects of loads

Combination 5: Permanent loads:

Rock load (soil pressure) + dead weight of structure + superimposed dead load + concrete shrinkage and creep forces;

Combination 6: Permanent loads + basic variable loads or other variable loads

Dead weight of structure + superimposed dead load + soil pressure + highway load, live load of train or flowing water pressure.

(4) Combination for short-term effects of loads

Combination 7: Permanent loads + other variable loads:

Rock load (soil pressure) + dead weight of structure + superimposed dead load + concrete shrinkage and creep forces + temperature load + swelling pressure.

Load combinations meeting bearing capacity requirements are applicable to checking and calculation of the bearing capacity and stability of structures. Load combinations meeting normal service requirements are applicable to checking and calculation of structural deformation, cracking and crack width.

6.1.5 Special loads mentioned in this provision means all other loads that are not listed in Table 6.1.1 but are likely to occur.

6.2 Permanent load

6.2.2 The calculation of relaxation load in a deep tunnel is based on the provisions of the *Specifications for Design of Highway Tunnel* (JTG D70—2004) with adjustments and supplements according to the results of the thematic research and analysis on the rock load increase/decrease rate in a long-span tunnel

Eq. (6.2.2-1) and Eq. (6.2.2-2) are the empirical equations of uniformly-distributed vertical pressure when the rock load is relaxation load. Rock load calculation height (h) means the thickness of possible loose mass of surrounding rock above the arc crown.

When Eq. (6.2.2-2) is used to calculate the rock load, the surrounding rock class$S$ shall be selected and used in the absence of BQ or [BQ]. If the BQ or [BQ] value of surrounding rock is available, the modified value [S] of surrounding rock class S can be obtained continuously by interpolation method based on Rock Quality Designation BQ or its modified value [BQ]. S can be replaced with [S] to consider the continuous variations of loads of the surrounding rocks of the same class with the Rock Quality Designation. As the BQ or [BQ] value of Class I surrounding rock has no definite upper limit in the surrounding rock classification system, it is appropriate to set its upper limit at 800 according to the calculated results of BQ or [BQ] values of a large number of Class I surrounding rocks. Similarly, as the BQ or [BQ] value of Class V surrounding rock has no definite lower limit in the surrounding rock classification system, it is appropriate to set its lower limit at 0 according to the calculated results of BQ or [BQ] values of a large number of Class V surrounding rocks. Since BQ or [BQ] value of Class VI surrounding rock is not calculated, S equals to 6, without a modified value [S].

6.2.3 In a long and shallow tunnel, consideration shall be given to the longitudinal transitional change of rock load with the increase of depth after portal construction. Rock load should be calculated respectively for different typical buried-depth sections. It is inappropriate to calculate and design the long and shallow section only based on the calculated results of rock load at the portal or at the border between deep and shallow overburden.

6.2.4 According to the investigation into eccentric load tunnels, eccentric load generally occurs at the portals due to terrain and shallow depth, and it occurs less in main sections of the tunnels and is mostly caused by geologic settings when occurring. Eccentric load acting on tunnel lining shall be determined based on the terrain, geological conditions and the covering thickness of outside surrounding rock.

6.2.5 The calculation method of rock load of twin tunnels with small clearance is added in the Specifications in reference of the *Guidelines for Design of Highway Tunnel* (JTG/T D70—2010).

6.2.6 The calculation method of rock load of twin-arch tunnel is added in the Specifications in reference of the Study on Key Technologies of Twin-arch Tunnel Construction (No. 2002-318-000-22) and the Promotion of Key Technological Achievements in Twin-arch Tunnel Construction

(No. 2007-318-799-80) under the Science and Technology Program on Traffic Construction in Western China of the Ministry of Transport. This method is applicable to twin-arch tunnels which are constructed by excavating center heading first, then constructing the center wall and finally excavating tunnels on both sides.

6.2.7 When the backfill of a cut-and-cover tunnel is placed and compacted manually or by small machines and tools, the physico-mechanical indices of backfill may be taken as those listed in Table 6-1.

Table 6-1 Physico-mechanical indexes of backfill

Description	Unit weight γ (kN/m^3)	Calculated friction angle ϕ_c (°)
Dry rubble	20	50
Backfill	19	35

6.3 Variable load

6.3.1 When tunnel structures bear the loads of automobiles (such as a cut-and-cover tunnel over which a road passes), the loads shall be calculated in accordance with relevant provisions of the current *General Specifications for Design of Highway Bridges and Culverts* (JTG D60) or *Code for Design of the Municipal Bridge* (CJJ 11).

6.3.2 The live loads of trains are generally not considered in the design of mountain highway tunnels. Only when the tunnel structures bear the live loads of trains (such as a cut-and-cover tunnel over which a railway passes, and the exterior walls of such cut-and-cover tunnel with a deep foundation) can the loads be calculated in accordance with relevant provisions of the current *Fundamental Code for Design on Railway Bridge and Culvert* (TB 10002.1).

6.3.3 Grout set is the leading cause of concrete shrinkage. Dry environment can also result in concrete shrinkage.

Factors affecting the concrete shrinkage and creep are very complicated and include the nature, quantity and quality of concrete components, the concrete age of loading for structures, and the weather conditions. Calculation shall be made based on actual data and considering the difference in construction and working conditions of structures. In case of data deficiency, calculation can be made based on elastic medium.

6.3.5 Construction load means the temporary loads acting on structural members during fabrication, transportation and hoisting, and the temporary loads from the transfer of machinery, other members and materials, including the dead weight of mechanical equipment and the loads from crowd, temporary transfer of members and materials, temperature action, hangers or other machines and tools. Temporary loads are mainly used for checking and calculation in construction phase. Their values shall be determined based on the specific construction phase, methods and conditions.

6.4 Accidental load

6.4.1 The calculation of the impact loading of falling rock is not specified because the current research on it is not thorough enough and few measure data are available. In specific design, it is determined mainly through field investigation, and simplified calculation method may be used for verification, if necessary.

6.4.2 Calculation of seismic load in the load combinations is done by static method.

7 Portal & Portal

7.1 General requirements

7.1.1 This provision sets out the technical principle of "early in and late out" to reflect the design philosophy that "not damaging the environment is the best way to protect it", to avoid forming high side and heading slopes at the portal, prevent such unfavorable geologic hazards as landslide, rockfall and collapse, to reduce damage to original landform and to protect natural environment.

7.1.2 Geological conditions at tunnel portals are typically poor, with fractured, strongly weathered rock mass and frequently encountered loosened talus. Excavation of portal changes landform and forms side and heading slopes, leading to potential geological disasters including collapse, eccentric load and landslide. Dealing with these disasters is difficult and costly. After becoming operational, the portal is extremely susceptible to the threat of natural disasters; and construction of sections at portals faces interference from adjacent works and possible impact on residents' life. Consequently, selection of a good location for portals is requisite to protect the environment, ensure operational safety, cut construction cost and guarantee smooth progress.

From the standpoint of terrain, the location of portal takes the following forms (Fig. 7-1):

(1) Perpendicular to slope surface——tunnel axis orthogonal with contour lines; this is the most ideal form.

(2) Oblique to slope surface——tunnel axis crossing obliquely the contour lines, side slope surface crossing obliquely the portal; eccentric load frequently occurs; the form of portal shall take into account possible impact of eccentric load.

(3) Parallel to slope surface——tunnel axis nearly parallel to contour lines, a relatively

extreme case of oblique crossing; the outside overburden over a long section of the tunnel after the portal is thin and subject to prominent eccentric load. In this case, the impact of eccentric load shall be considered as per Fig. 7-2.

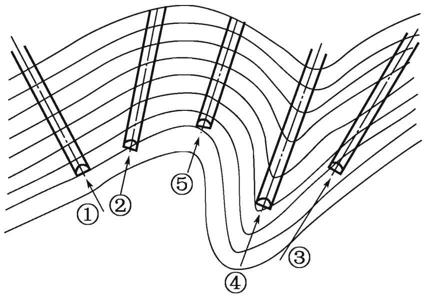


Fig. 7-1 Schematic relationship between portal and terrain

①Orthogonality with slope surface; ②Oblique crossing of slope surface; ③Parallel to slope surface; ④Entry at ridge; ⑤Entry at cleuch

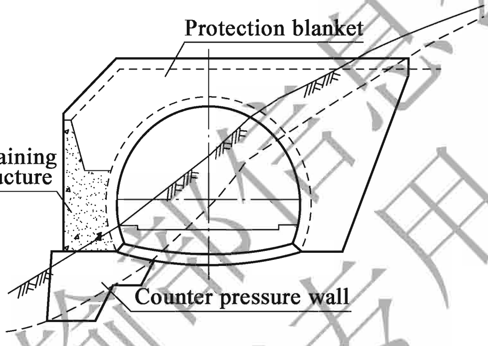


Fig. 7-2 Schematic anti-eccentric load structure

(4) Entry at ridge——the ridge is generally stable, but attention shall be paid to the effect of flood in the gullies on both sides on portal.

(5) Entry at cleuch——there are unstable sedimentary deposits such as talus, high groundwater table and possible natural disasters such as flood, debris flow and accumulated snow.

7.1.3 The purpose of portal drainage is to prevent water above the portal from scouring portal side and heading slopes at construction and operation stages; prevent water on pavement at portals from entering the tunnel; and from impacting the on construction of portals.

7.1.4 Portals shall play the role of intercepting possible debris flow, rolling rock and collapsed material from the side and heading slopes.

7.1.5 The accumulated snow hazard comes primarily from accumulation of snowdrift, snowfall and snow slide. Accumulation of snow on side and heading slopes may lead to collapse of the slopes. Accumulated snow on top of the tunnel increases loading on the tunnel and seepage of surface water. Snow on the pavement affects traffic safety and results in traffic congestion.

7.1.6 Portal side and heading slopes, portals and portal wall back require inspection and maintenance; debris deposits at wall back need to be removed; water interception drain and drain ditch require dredging and repair. Therefore, consideration shall be given to ease of inspection and maintenance.

7.1.7 Compatibility of portals with natural environment means to protect and reinstate original terrain and natural landscape to the greatest degree, reduce excavation traces, avoid excessive artificial decoration, downplay or conceal the existence of retaining structure.

7.2 Portal engineering

7.2.1 Improper location of portal would undermine mountain stability, induce geologic disaster, inhibit tunnel excavation, threaten construction safety, jeopardize long-term operation safety and do damage to the environment. Portals shall avoid the following locations:

(1) Deposit body, landslide, loosened rock formation, fractured rock formation, steep terrain and places susceptible to collapse and rockfall.

(2) Where the contour intersects tunnel axis at small angles.

(3) Places threatened by flood and debris flow.

The selection of portal location on gentle slope is flexible. A long cutting when used is inexpensive and occupies a large area of land, forming a long cutting at the portal. A long tunnel (generally extended cut-and-cover tunnel) when used requires early entry and is costly and occupies a small area of land (the top of cut-and-cover tunnel can also be backfilled and reclaimed and planted with trees). For twin-arch tunnels and twin tunnels with small clearance, it is better to adopt a long cutting; for separated twin tunnels or single tunnel, it is better to adopt a longer tunnel.

7.2.2 This provision is prescribed to ensure portal construction and operation safety.

1 Minimizing excavated height of side and heading slopes and controlling excavation scope are required to keep side and heading slopes stable and reduce damage to original landform and portal landscape. In general, forepoling measures such as pipe roofing are installed at the opening to reduce excavated height.

2 Portal side and heading slopes shall be protected appropriately based on the conditions at the portal. For side slopes in rock, fractured rocks, rocks with locally unstable mass, rocks with bedding, or, if debris, slide, collapse, falling off and other threats exist following excavation of side and heading slope, respective measures, including removal, retaining and extending cut-and-cover tunnel or anchorage, installing flexible net and other means are required for protection. For side and heading slopes in soil, their stability shall be ensured by making the slope gentler, landscaping or providing retaining wall.

3 When encountering zones of flood and debris flow, they are frequently bypassed via mined tunnel or cut-and-cover tunnel. If a gully passes the tunnel top, the cut-and-cover tunnel shall be extended and an aqueduct provided on its top. For portals located at cove or valley, besides considering general drain trench and interception drain, other ditches fulfilling flood release requirements shall also be provided based on storm and flood and water catchment conditions.

4 Cliff is generally stable and shall not be cut to avoid disturbance. If hanging rocks are present at the cliff, they shall be removed and if necessary, protected with shotcrete and rockbolt or protection net while extending the cut-and-cover tunnel to move the portal outward or installing shed.

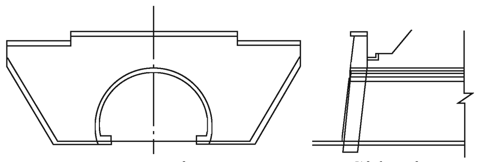
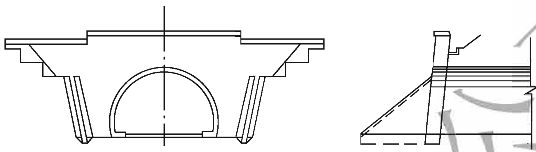
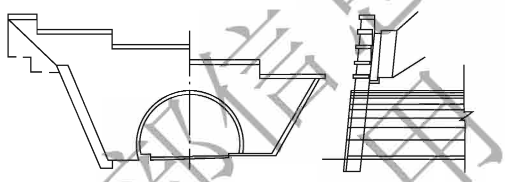
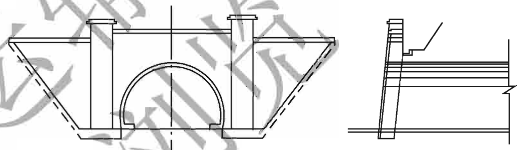
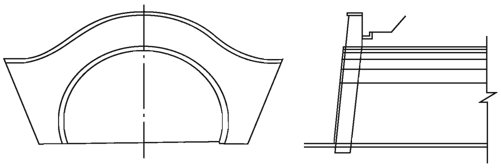
7.3 Portal structures

7.3.1 Portals are the only exposed part of a tunnel that links tunnel lining and subgrade structure outside the tunnel. They are an essential component of tunnel structure and a structure marking the tunnel. The role of tunnel portals is to retain heading slope and cut slope, intercept small amounts of spalling and falling materials from the heading slope, maintain the stability of side and heading slopes and direct water on the slope away from the tunnel. Tunnel portals are designed considering tunnel span, topographic and geologic conditions, hydrological conditions, surrounding buildings/structures, local natural landscape and cultural landscape.

Highway tunnel portals are broadly grouped into two categories: end-wall tunnel portal and cut-and-cover tunnel portal.

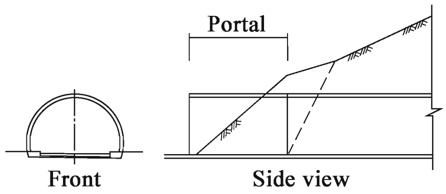
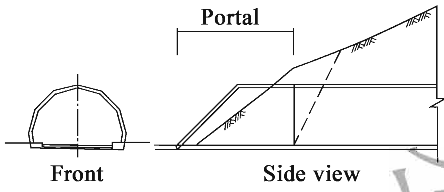
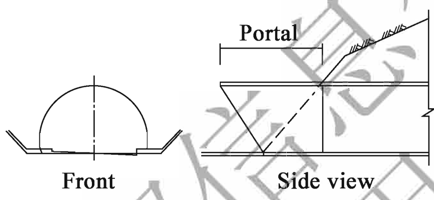
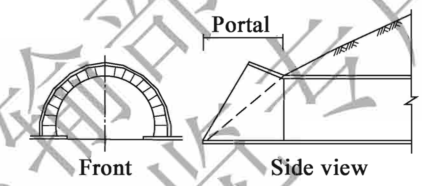
End-wall tunnel portal includes wall portal, wing wall portal, bench portal, column portal and wing wall portal. This type of portal is generally perpendicular to tunnel axis. The wing wall is subgrade slope retaining structure parallel to the route at portals that is connected to the end wall, as shown in Table 7-1.

Table 7-1 End-wall tunnel portal

Classification	Name	Sketch	Remarks
End-wall tunnel portal	End-wall tunnel portal		Suitable for narrow zones with steep heading slope, narrow areas at oblique crossing terrain
	Wing wall tunnel portal		
	Bench tunnel portal		
	Post tunnel portal		
	Arch wing wall portal		

Cut-and-cover tunnel portal includes straight cut portal, inclined portal in the form of bamboo cutting, arch inclined portal in the form of inverted bamboo cutting, trumpet portal, shed tunnel portal and frame portal. Cut-and-cover tunnel portal (except for shed tunnel portal and frame portal) is a lining structure of the tunnel behind the portal that protrudes from the mountain slope, as shown in Table 7-2.

Table 7-2　Cut-and-cover tunnel portal

<table>
<tr><th>Classification</th><th>Name</th><th>Sketch</th><th>Remarks</th></tr>
<tr><td rowspan="4">Cut-and-cover tunnel portal</td><td>Straight cut portal</td><td></td><td rowspan="4">Suitable for zones with open terrain, low side and heading slopes, gentle heading slope and tunnel axis orthogonal or nearly orthogonal with the contour lines</td></tr>
<tr><td>Inclined portal in the form of bamboo cutting</td><td></td></tr>
<tr><td>Arch inclined portal in the form of inverted bamboo cutting</td><td></td></tr>
<tr><td>Trumpet portal</td><td></td></tr>
</table>

Shed tunnel portal and frame portal are a form of cut-and-cover tunnel portal. The former is used in zones with high side and heading slopes and the potential for rockfall. The latter is used where the overburden above the tunnel is thin and a highway crosses over the tunnel or other buildings are present above the tunnel.

7.3.2　Whatever the relationship between tunnel axis and contour is, orthogonality of portal with tunnel axis is pleasing to the eye and beneficial to traffic safety. This is the common practice in China.

7.3.3　This provision is for design of end-wall tunnel portal.

1　The calculation method for soil pressure on portal end wall and wing wall is presented in Appendix K. Structural dimensions of portal wall and wing wall are determined from calculated results of wall strength, stability and overturning resistance or by engineering analogy. When necessary, wall structure shall be checked for stability and overturning resistance. Minimum thickness of wall means the minimum dimensions of the load-carrying part of the wall.

2 There is some horizontal distance from the toe of heading slope above the tunnel to the portal wall back. This is to prevent rock and soil on the heading slope from falling onto the pavement and ease provision of drain trench between portal end wall and heading slope. Fill of certain thickness is required between the bottom of drain trench on top of the tunnel and outer edge of lining crown to prevent falling rock damaging the arch ring. The end wall top is 0.5m higher than crown backfill surface to prevent falling rock and soil from rebounding to the pavement and serve as safety barrier when inspection and maintenance personnel carry out maintenance at the crown. The wall back backfill surface refers to the top surface of extrados backfill close to wall back. It is usually the top surface of side wall of drain trench on top of the tunnel, as shown in Fig. 7-3.

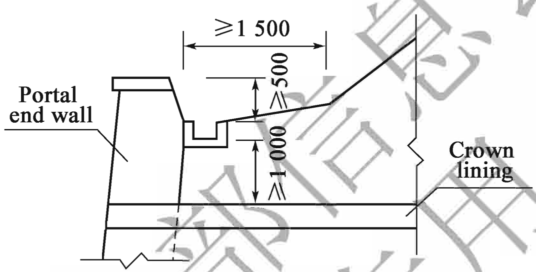


Fig. 7-3 Structure of top of portal wall back (dimensions given in mm)

3 Deformation joints and drainage holes in the portal wall shall be configured according to retaining wall requirements.

4 The embedded depth of portal wall foundation depends on the geological conditions. The embedment depth is the depth after removing highly weathered layer on the surface. If the weathered layer is thick and difficult to remove completely, the foundation shall be embedded in the bedrock according to the degree of weathering of bedrock and the corresponding ground bearing capacity. The base on sloping ground needs to be made into benches to prevent sliding of wall.

According to the experience in placing foundation in highway engineering, this provision requires the base be placed not less than 0.25m below the frozen depth. If the frozen depth is shallow and construction is difficult, the base shall be removed and replaced with non-freezing sand and gravel, or treated by providing pile foundation. In non-swelling ground such as rock, gravel, cobble and sand, embedment depth is not restricted by frozen depth.

5 In case of insufficient ground bearing capacity, common measures to stabilize the ground

include extended foundation, pile foundation, raft foundation, replacement of subsoil and grouting.

7.3.4 This provision is for design of cut-and-cover tunnel portal.

1 Portal structure is part of portal lining as well as cut-and-cover tunnel lining which requires the use of reinforced concrete structure.

2 The portal lining shall extend not less than 500mm beyond the original slope surface or design heading slope surface (Fig. 7-4). This is to prevent slope water and mud from entering the intrados of lining.

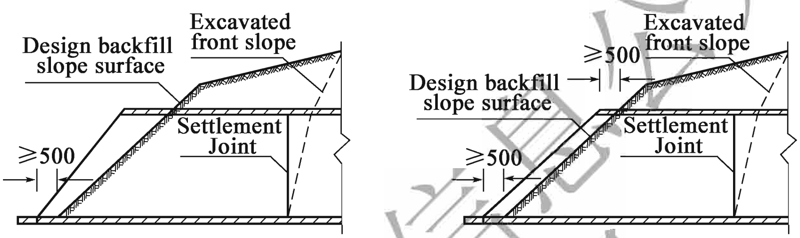


Fig. 7-4 Configuration of portal lining extending beyond slope surface (dimensions given in mm)

3 The exposed end of portal lining can take different forms, such as straight cut, inclined portal in the form of bamboo cutting, arch inclined portal in the form of inverted bamboo cutting or trumpet. When the lining end face is vertical, the portal is straight cut; when it is upward inclined, the portal is inclined portal in the form of bamboo cutting; when it is downward inclined, the portal is arch inclined portal in the form of inverted bamboo cutting; when it is in trumpet shape, the portal is trumpet portal.

5 The design backfill slope should be backfilled per the gradient of natural slope. This is to restore the terrain to its original state. If rock and soil are used as backfill material, the slope ratio should not be steeper than 1∶1. This is to ensure the stability of backfill slope surface. The slope is generally protected with local plants compatible with surroundings or with steel mesh.

8 Design of Lining Structure

8.1 General requirements

8.1.1 Main forms of tunnel lining are shotcrete and rockbolt lining, monolithic lining and composite lining.

(1) Shotcrete and rockbolt linings are a collective term for ①shotcrete support, ②shotcrete + rockbolt support, ③shotcrete + rockbolt + reinforcement mesh support, and ④shotcrete + rockbolt + reinforcement mesh + steel rib support. They are a form of support lining that can take full advantage of the self-supporting capability of the ground to reinforce rock mass and control rock deformation. It provides timely support, is flexible, in tight contact with the rock and deforms together with the rock. It is more advantageous than monolithic lining under loading conditions and can significantly speed up construction progress, save labor and raw materials and reduce cost, and ensure long-term stability of the surrounding rock. However, due to its low stiffness, shotcrete and rockbolt lining has certain limitations in long-term stability and capability to resist water erosion in surrounding rock of Classes IV-VI with low self-stability. In such rock, the material and construction process need improving. Therefore, the provision requires "composite or monolithic lining be employed in surrounding rock of Classes IV-VI" and shotcrete-and-rockbolt support is not used alone as permanent lining.

(2) Monolithic lining is a form of lining widely applied. It has high support capability, waterproofing capability and durability, can provide long-term reliable support, is supported by long-term engineering practical experience and mature technology, and adaptable to various surrounding rock conditions. Therefore, use of monolithic lining is proper and reliable in sections next to the tunnel portal, sections with shallow overburden and weak surrounding rocks with poor conditions. At present, monolithic lining for mountain tunnels is constructed of cast-in-situ concrete or cast in situ reinforced concrete.

(3) Composite lining consists of two layers of lining. The first layer is called primary support (generally shotcrete and rockbolt lining) and the second one secondary lining (generally monolithic lining). A waterproofing layer is sandwiched between the two layers. Composite linings have been commonly applied to high-class highway tunnels in China.

The secondary lining of composite lining provides good surface finishes that meet basic surface finish requirements of the tunnel. A waterproofing layer is placed between primary support and secondary lining to address water seepage through the lining. Therefore, this provision prescribes that "Expressway, Class I and Class-2 highway tunnels should be provided with composite linings".

For highway tunnels up to Class III which carry small volumes of traffic and are used infrequently, shotcrete and rockbolt lining may be applied in goodsurrounding rock conditions to control capital cost.

The section next to the portal refers to a length of mined tunnel from the portal where overburden thickness is less than 1-2 times the excavation width (Fig. 8-1), with typically poorersurrounding rock conditions than the tunnel proper. It is shallow and greatly affected by topographic and environmental conditions. The portal section requires a high anti-weathering capability and durability. There is inadequate experience of shotcrete and rockbolt lining in terms of stability and resistance to water erosion. Its material and construction process need improving. Therefore, this provision specifies "monolithic or composite lining shall be used for the portal section".

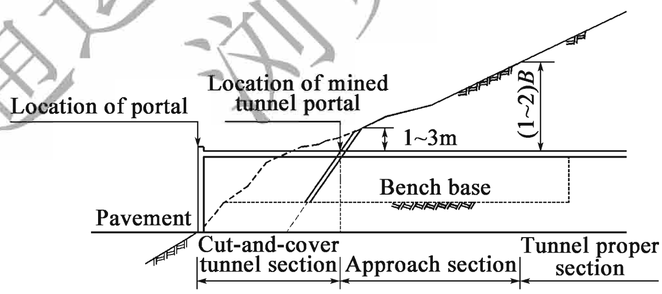


Fig. 8-1 Approach section

B-excavation width (m)

8.1.2 Making the most of and taking full advantage of the self-supporting capacity of surrounding rock is a basic principle that shall be followed in tunnel lining structure design. The surrounding rock itself has certain structural effect. This characteristic can be utilized by employing effective engineering measures, reasonable type of lining and appropriate construction method to keep the surrounding rock stable and save investment. Tunnel lining, as permanent critical structure, is difficult to restore once damaged during operation and costly to maintain. This presents great difficulty in traffic operation management. Therefore, this provision specifies that "The lining shall have sufficient strength, stability and durability to ensure the long-term operational safety of the tunnel".

8.1.3 The lining structure design for highway tunnels is currently based primarily on engineering analogy method. Given the complexity of geologic conditions, different surrounding rocks have different bearing capacity. The class of surrounding rock, embedment depth, excavation method, support means and support timing directly dictate in-situ stress state and structural loading. Sometimes engineering analogy alone is not sufficient to guarantee design rationality and reliability. In such cases, theoretical check is required. At the tunnel design stage, the designer can hardly predict various complex changes accurately. During project implementation, field tunnel construction monitoring and measurements and observation of deformation and changes in surrounding rock and primary support are carried out to understand the actual conditions of surrounding rock and stress in the support structure so that timely adjustments can be made to support parameters. During construction, support parameters are lowered appropriately if surrounding rock conditions are good with small deformations which tend to stabilize; on the contrary, support parameters are strengthened. This is called observational approach. Support parameters shall be determined by test for important works, special zones and where engineering analogy is not applicable.

8.1.4

1 Common lining sections for tunnel and underground works are curved wall arch lining and straight wall arch lining. Highway tunnels generally have large spans, loadings and deformations. According to a large number of engineering practices and mechanical analyses, the curved wall arch lining of a highway tunnel is favourably loaded; with stable surrounding rock and structure and a high capacity to resist lateral pressure, compared to the straight wall arch lining. It is adaptable to many kinds of surrounding rock conditions. Based on survey in cold regions, damages to the wall of curved wall lining are far less frequent than the straight wall lining. Therefore, the curved wall arch section is recommended.

For small-section tunnels such as vehicular cross passage, pedestrian cross passage and ventilation channel as well as fan cavern and work cavern generally with good geologic conditions and special requirements for cross-section, straight wall arch lining is generally employed.

2 In Class V surrounding rock with low self-stability, high lateral pressure and low foundation bearing capacity, to ensure structure integrity, safety and settlement control, a

closed lining section with arch invert is employed. For Class IV surrounding rock, whether to install arch invert or not shall be determined based on tunnel cross section, geologic structure, formation lithology and groundwater conditions. In many cases (especially the portal section) in engineering practice, surrounding rock conditions at the arch are very poor and even pipe roofing and other auxiliary engineering measures are needed. But surrounding rock geological conditions at base of side walls and below are relatively good such that the base bearing capacity and stability can meet structural stress requirements. In such cases, arch invert is not provided to save investment and simplify construction. Therefore, this provision specifies "An arch invert may not be required if the rock surrounding the tunnel invert is good and the bearing capacity and stability of the side wall base are sufficient."

3 The tunnel portal section is generally shallow with poor geologic conditions, easily susceptible to the environmental impact and weathering and less stable in long term than in the tunnel. The lining stress state is more adverse than in the tunnel. At times, the lining is subjected to longitudinal thrust in the direction of heading slope. Therefore, "The portal section requires reinforced linings". The purpose of specifying minimum length of the reinforced section is to ensure the effective role of the lining in this section. For structural design of reinforced lining, the class of surrounding rock at the portal is generally reduced by one class.

4 Rock load and deformation vary with surrounding rock geological conditions surrounding the tunnel proper. The boundary between classes of surrounding rock is difficult to accurately define since it changes gradually at times. Extending the lining in poor surrounding rock conditions to good conditions enables the lining to adapt to this change as a transition.

5 Extending the unsymmetrically loaded lining toward conventional lining is also based on the consideration mentioned above.

6 An in-tunnel intersection is bifurcations where two caverns intersect at the arch, subject to complex stress and with complicated calculation and construction. To ensure safety of such structure, the intersection between main tunnel and vehicular cross passage with clear width greater than 3.0m, refuge room and horizontal ventilation adit shall be strengthened. The extent of strengthened intersection extends from the edge of the intersection by not less than 5.0m toward main tunnel and not less than 3.0m toward the cross passage to ensure the structural stability of the intersection, as shown in Fig. 8-2. The extended length shall increase with span.

Small-section caverns such as pedestrian cross passage, fire equipment cavern and control cabinet intersect the main tunnel at its side wall, with span and height typically less than 3.0m. Special consideration is not required.

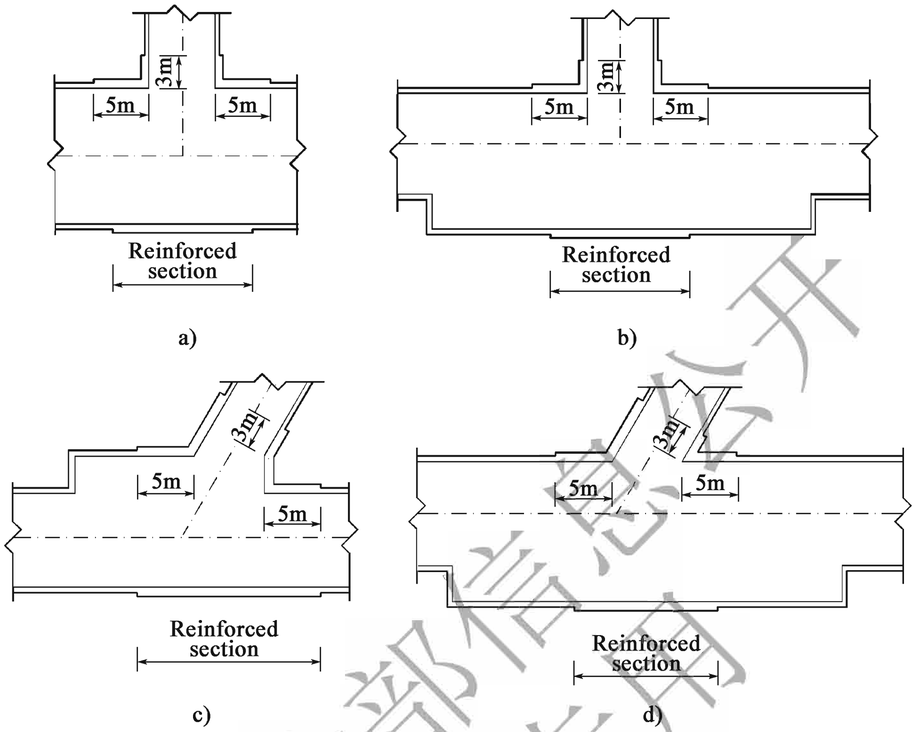
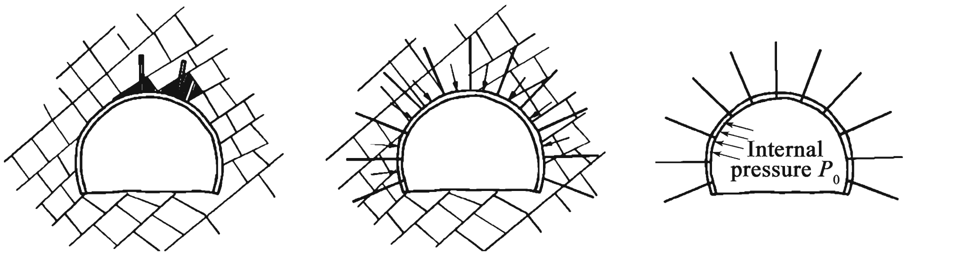


Fig. 8-2 Intersection

8.2 Shotcrete and rockbolt lining

Shotcrete is a support method by which concrete mixture is directly sprayed onto the tunnel wall using pump or compressed air through shotcreting machine, material delivery pipe and nozzle. Shotcrete is a structure to maintain stability of the rock surrounding the tunnel and has the advantages of not needing formwork, rapid application, high early strength, good compactness, tight adhesion to surrounding rock with no gap. Prompt application of shotcrete support following excavation can seal the rock face, prevent weathering and loosening, fill concave and crack, maintain and increase rock mass integrity, help the rock with its self-stability, adjust stress distribution in surrounding rock, prevent stress concentration, control rock deformation and prevent rock from falling off and collapse.

As part of the shotcrete-and-rockbolt support, rockbolt is a rod anchored inside the rock and integrated into the rock to improve mechanical property of rock, adjust stress in the rock, inhibit rock deformations, reinforce the rock and maintain rock stability. The rockbolts make use of the benefits of their effects of suspension [Fig. 8-3a)], combined arch [Fig. 8-3b)], span reduction and strengthening by squeezing [Fig. 8-3c] to increase rock mass integrity by holding blocks with joints and fissures in place.



a)Suspension effect　b)Combined arch effect　c)Squeezing reinforcement effect

Fig. 8-3　Rockbolt effect

Leveraging the rockbolt support to surrounding rock requires: 1) effective bond length; 2) rockbolt being grouted full length to be part of the rock mass; and 3) avoiding loosening, corrosion damage.

8.2.1　Due to shotcrete shrinkage, a shotcrete layer less than 50mm thick tends to suffer shrinkage induced cracking; meanwhile, it is too thin to resist rock movement.

Shotcrete is required to have certain flexibility. In general, shotcrete for a two-lane tunnel shall not exceed 300mm in thickness. For a large size tunnel with more than three lanes, shotcrete layer is relatively flexible. In unstable Class V surrounding rock, shotcrete thickness may be required to be greater than 300mm. With the advancement in mechanized tunneling and construction technology, it is technically possible to construct shotcrete layer more than 300mm thick. Therefore, this revision has removed the provision that shotcrete thickness "should not be greater than 300mm".

C20 shotcrete is the basic requirement for shotcrete strength.

8.2.2　Placing reinforcement mesh in shotcrete helps increase shotcrete shear strength and bending strength, its capability to resist shear and bending, enhance shotcrete integrality and reduce shrinkage cracking in shotcrete. The construction sequence of reinforcement mesh shotcrete: first apply initial shotcrete; install reinforcement mesh; then apply final shotcrete to cover the reinforcement mesh.

1　The shotcrete layer is not thick. Reinforcement mesh with 6mm diameter bars can improve shotcrete performance. The reinforcement mesh is required to be installed along the undulating surface of the rock. Larger bars of mesh will add to installation difficulty. The use of larger diameter bars is not encouraged.

2　According to practice, if bar spacing is less than 150mm, shotcrete rebound is serious and voids tend to form between the bar and wall surface, resulting in less compact reinforcement mesh shotcrete. If the bar spacing is greater than 300mm, the effect of reinforcement mesh in shotcrete will be reduced considerably. Therefore, the bar spacing for reinforcement mesh is prescribed as 150mm ~ 300mm. Different combinations may be employed: 150mm × 150mm, 150mm × 200mm, 200mm × 200mm, 200mm × 250mm, 250mm × 250mm, 250mm × 300mm and 300mm × 300mm.

3 The lap length requirement for reinforcement mesh is consistent with that for reinforced concrete structure, i. e. the bar lap length shall be 30d (d is bar diameter).

4 The thickness of bar cover shall not be less than 20mm, which is consistent with the specification for ordinary reinforced concrete. When two layers of reinforcement mesh are used, the spacing between them shall be beneficial to the efficiency of reinforcement mesh. In actual engineering practice, it is discovered that the 60mm spacing between the two layers of reinforcement mesh is small. This revision has changed it to 80mm.

5 Reinforcement mesh requires certain cover thickness. Since the location of bar installation is impossible to be very accurate, the thickness of reinforcement mesh shotcrete shall not be less than 80mm and that for two-layer reinforcement mesh shotcrete shall not be less than 150mm. This is to ensure sufficient cover thickness for reinforcement mesh and the spacing between two layers of reinforcement mesh.

6 The reinforcement mesh is tied or welded to rockbolt so that it is fixed to rock face firmly. If there is no rockbolt or rockbolt will be installed later, temporary short rockbolt not less than 0.3m long may be used to fix the reinforcement mesh.

8.2.3 Adding certain amount of steel fiber or synthetic fiber to shotcrete can improve its performances compared to ordinary shotcrete.

2 Steel fiber reinforced shotcrete has ductility 10-50 times higher than plain concrete and shock resistance capability 8-30 times higher than plain concrete. Adding 40 ~ 60kg/m^3 steel fiber increases the compressive strength by 10.3% ~22.3% and splitting strength by 41% ~68%, compared with concrete with no steel fiber. The mechanical property of steel fiber reinforced shotcrete increases with the amount of steel fiber added, but excessive amount of steel fiber will reduce mix evenness and spray smoothness. In fact the amount of steel fiber added is mainly dictated by shotcrete process. When the amount exceeds 4% of the weight of dry concrete mixture, the mix evenness and shotcrete workability during construction will deteriorate and rebound will increase. Therefore, the amount of steel fiber added should be 33 ~ 96kg per cubic meter of shotcrete, i. e. 1.5% ~4% of weight

of dry concrete mixture.

3 Synthetic fiber reinforced shotcrete is to mix into concrete thin fibers (such as polypropylene fiber) made from chemical materials with certain tensile strength to considerably improve shotcrete tensile strength, ductility and crack resistance without affecting concrete application process. At present, the types of synthetic fibers, performance parameters and the amount added will influence the mechanical performance of shotcrete. Experience in this regard is inadequate. It is hard to specify a uniform standard. Therefore, it shall be determined by test. Adding synthetic fiber has an inconspicuous effect on improving concrete compressive strength.

4 High performance shotcrete has high strength, durability and good waterproofing performance. Application of high performance concrete involves adding a small amount of fiber, silica fume, ground granulated blast furnace slag, fly ash and efficient superplasticizer to steel fiber reinforced shotcrete to form shotcrete lining with high strength grade, high impermeability and durability. For application of high performance concrete, the design strength grade shall be C40 and C50 and impermeability index not less than B12. Works already executed using such concrete suggest certain effectiveness.

8.2.4 The type, length and spacing of rockbolts are important parameters in rockbolt design and shall be selected based on the tunnel surrounding rock geological conditions, tunnel cross-section dimension, the role of rockbolt and construction process conditions.

Rockbolts are categorized by action principle as follows: ① Fully bonded rockbolts including ordinary anchor rod with cement mortar, anchor rod with early strength cement mortar, resin grouted rockbolt, cement grouted rockbolt, hollow grouted rockbolt, combined rockbolt and self drilling grouted rockbolt. Firmly bonded to the rock wall of borehole using cement mortar or resin as the binder, the rockbolt provides frictional resistance; and through the restraining force on rock wall from bearing plate and nut mounted at the hole mouth, inhibits rock deformations and withstands relaxation load from the rock. This type of rockbolt can be used for systematic rock bolting, local rockbolt and rockbolts at toe of wall as permanent support. ②end anchored rockbolts including mechanically anchored rockbolt and end bonded rockbolt. Through mechanical or bonded anchorage, the rockbolt front end is anchored to the rock at the bottom of rockbolt hole and tensioned via bearing plate and nut at hole mouth, to introduce an axial force into the surrounding rock. This type of rockbolt is used mainly for prestressed rockbolt and local rockbolt to provide temporary support and when filled with mortar provide permanent support. Mechanical rockbolts are divided into wedge joint anchor (rock bolt), expansion rock bolt and arch invert wedge rockbolt. Mechanical rockbolts can be used in hard rock support. End bonded rockbolts include resin bonded end rockbolt and rapid hardening cement spool end anchored rockbolt. End bonded rockbolts are used in hard and moderately hard rocks as well as soft rock. ③friction type rockbolts include slot-tube rockbolt and water expansion rockbolt and are mainly used for local rockbolt to provide temporary support.

Rockbolts are divided by construction process into ordinary mortar rockbolt, hollow grouted rockbolt, combination hollow rock bolt and self drilling rockbolt.

1 Rockbolts used as permanent support shall have good long-term effect. As with rebars in reinforced concrete, rockbolt body needs certain cover. Filling the rockbolt hole with cement mortar or resin not only guarantees the frictional force between mortar and rockbolt and between mortar and hole wall to ensure rockbolt-rock interaction, but also provides a cover to the rockbolt. End anchored rockbolts are subject to corrosion from groundwater or moist air effect and reduction in anchorage force due to surrounding rock creep, and therefore cannot be used as permanent support. If filled with mortar, they can serve as permanent support. The not less than M20 strength of cement mortar or resin in the rockbolt hole is to ensure rockbolt strength and durability.

2 For surrounding rock with a short stand up time, fully bonded, resin anchored rockbolts or early strength anchor rod with cement mortars should be used so as to give play to early effect of the rockbolt.

3 Prestressed rockbolts are not common in highway tunnels. Their application shall comply with the *Technical Code for Engineering of Ground Anchorages and Shotcrete Support* (GB 50086).

4 In zones of fractured rock where rockbolt hole formation is difficult and the hole tends to collapse after extraction of drilling rod, self drilling rockbolt can be used. The self drilling rockbolt uses drilling rod as rockbolt with drilling bit installed at the front tip of the rockbolt. The process of drilling is the process of installing the rockbolt, after which the drilling rod is not pulled out. When the rockbolt has been driven into place, the rockbolt hole is grouted through the hole in the rod.

5 Rockbolt diameter: generally 20 ~ 25mm for ordinary mortar rockbolt; 25 ~ 28mm for hollow rockbolt and combination hollow rock bolt; 25 ~ 52mm for self drilling rockbolt.

6 At the exposed end of the rockbolt, a bearing plate is fastened by bolt against the rock face at the hole mouth to introduce an axial force into the surrounding rock. This can increase the extent of rockbolt action and the effectiveness of rockbolt. In actual engineering practice, the bearing plate thickness of 6mm has been found to be insufficient. This

revision changes it to 8mm.

8.2.5 Systematic rock bolting mainly reinforce the surrounding rock as a whole by forming an arch bearing structure to certain depth into the surrounding rock to leverage the rock characteristics of high compressive strength and increase its self-supporting capacity.

1 Systematic rock bolting are usually arranged radially along tunnel excavation line. It should be noted that when the rockbolts are parallel to or intersect main structure plane of rock and rock layer plane at small angles, the anchorage effect and the combined arch effect of the rockbolts are poor. When arranged at large angles, the rockbolts can pass through more structure planes and knit adverse structure planes or rock layers together.

2 "Rockbolts shall be arranged in rectangular or quincunx form" in item 2 of 8.2.9 of the previous edition has been changed to "rockbolts should be arranged in quincunx form" in item 2 of 8.2.5 in this edition, with spacing of rockbolts arranged in quincunx form specified. The number of rockbolts arranged in quincunx form is smaller than that in rectangular form. From survey of application in more than 10 years, it is appropriate to reduce the number of systematic rock bolting.

3 The length and spacing of systematic rock bolting are determined using engineering analogy method or using the following equation:

Rockbolt length:

$$L = \frac{1}{3}W - \frac{1}{5}W \tag{8-1}$$

Rockbolt spacing:

$$P = 0.3L \sim 0.5L \tag{8-2}$$

where

L—rockbolt length (m);

W—tunnel excavation width (m);

P—(transverse) spacing of rockbolts (m).

4 Staggered arrangement of long and short rockbolts can reduce the quantities of rockbolt used.

5 For highway tunnels, the excavation width is generally more than 10m and excavation cross-section area is large. The length requirement for systematic rock bolting aims to ensure formation of an arch bearing structure within certain depth of surrounding rock. Based on survey and statistics of engineering cases in China, the systematic rock bolting

length is generally not less than 2.0m for a single-tube two-lane tunnel, not less than 2.5m for a single-tube three-lane tunnel and not less than 3.0m for a single-tube four-lane tunnel.

6 The provision of "not installing systematic rock bolting in surrounding soil" is added based on the research findings from the "test and research on not installing systematic rock bolting in soil tunnel" program of Department of Transportation of Heilongjiang Province and research results on loess tunnel. Relevant tests and researches suggest rockbolts are extremely difficult to install in surrounding soil and that installation of systematic rock bolting are costly and time consuming with poor effect. In lieu of systematic rock bolting, reinforced steel rib support and anchor pipe to lock the base of walls are employed with good effect achieved.

8.2.6 The primary role of local rockbolts is to prevent unstable rock block from collapsing or sliding by anchoring them to stable rock mass using rockbolt (Fig. 8-4). Effective anchorage end must be placed into stable rock mass. Anchorage force T is calculated from the following formulae based on field geologic survey:

$$T \geqslant W - f \tag{8-3}$$

$$T = \frac{T'}{\cos\theta} \tag{8-4}$$

$$T' \geqslant w - f' \tag{8-5}$$

$$f' = f\cos\alpha \tag{8-6}$$

When the anchorage force of a single rockbolt, t, is less than T,

$$n \times t \geqslant T \tag{8-7}$$

where,

W—gravity of sliding rock mass (kN);

f—frictional force between sliding rock mass and stable rock mass (kN);

n—number of rockbolts (piece);

θ—the angle between rockbolt anchorage direction and vertical direction (°);

α—the angle between sliding plane of sliding rock mass and vertical direction (°).

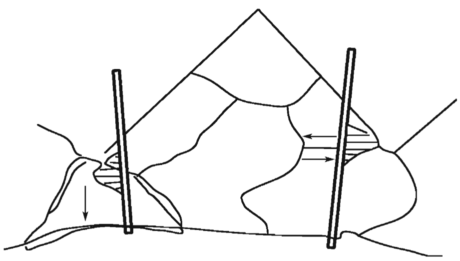


Fig. 8-4 Schematic anchorage effect of local rockbolts

8.2.7 The role of steel rib support (i. e. steel arch frame support) is to increase shotcrete stiffness and strength as an effective measure to control rock deformations and loosening and enhance the support capacity of shotcrete and rockbolt lining. The steel rib support includes steel lattice girder and structural steel rib support. Common structural steel rib supports are I-steel rib support, U-steel rib support and H-steel rib support. The I-steel rib support is formed using a cold bending machine. U-steel rib support does not require connecting steel plates; instead, it employs lap connection and is bolted, providing some flexibility by allowing small adjustments to camber.

1 Steel rib support requires sufficient strength and stiffness, and the ability to withstand 1-3m relaxation load from rock column while maintaining its own stability.

2 Compared with structural steel rib support, the steel lattice girder has the advantages of good stress resistance quality, light weight, adjustable stiffness, field processing and fabrication, convenient installation and the ability to tightly bond to shotcrete. It provides steel arch rib support with certain stiffness and strength and its use shall be promoted.

3 The spacing of steel lattice girder supports is determined by the class of surrounding rock, excavation width and round length, subject to adjustments based on construction tunnel construction monitoring and measurement. If the spacing is too small, the shotcrete density behind the steel rib support cannot be ensured; if the spacing is too large, the rock blocks between two steel rib supports are susceptible to collapse due to the limited support width of the steel rib support, leading to weaker support effect. According to survey and investigation, when steel rib support is employed, it is rare to adopt a spacing greater than 1.2m. This revision has changed the spacing of steel rib supports from "0.5 ~ 1.5m" to "0.5 ~ 1.2m". To avoid overlapping of rockbolts with steel rib supports so that they play their respective role, the longitudinal spacing of steel rib supports shall be the same as that of rockbolts (Fig. 8-5).

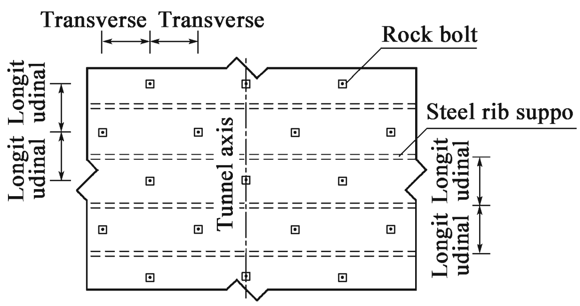


Fig. 8-5 Schematic relationship of steel rib support and rockbolt layout

4, 5 The stiffness of a single steel rib support is very low, similar to a slender pole, with a low bearing capacity. Simultaneous use of more than 3 steel rib supports with adjacent two frames connected transversely ensures the load is exerted on the overall steel rib supports and increases lateral stability.

6 To ease erection, each steel rib support shall be fabricated in segments, and the length of each segment should fit with excavation cross-section. The segments are bolted or welded via steel plate.

7 For steel rib support, the thickness of shotcrete cover on the side against rock wall which is uneven shall not be less than 40mm; the thickness of shotcrete cover on the free side shall not be less than 20mm given its good compactness. If the tunnel only employs shotcrete and rockbolt lining, the thickness of cover shall not be less than 40mm because the shotcrete-and-rockbolt is exposed to air for long periods of time.

8 Tunnel excavation section shall take into account deformation allowance. The steel rib supports shall be installed against surrounding rock so that they deform along with the surrounding rock. Once deformed under load, their dimensions will be smaller than at the time of fabrication; they may encroach upon design clearance or secondary lining. Therefore, the shape and dimensions of steel rib supports shall be determined based on excavation section.

8.2.8 The plane of connecting steel plates at both ends of steel lattice frames shall be perpendicular to the axis of the steel rib support to ease installation. U-shaped bars shall be added to aid the welding of the connecting steel plate to the main bar of the steel lattice frame, as shown in Fig. 8-6, so as to ensure secure welding.

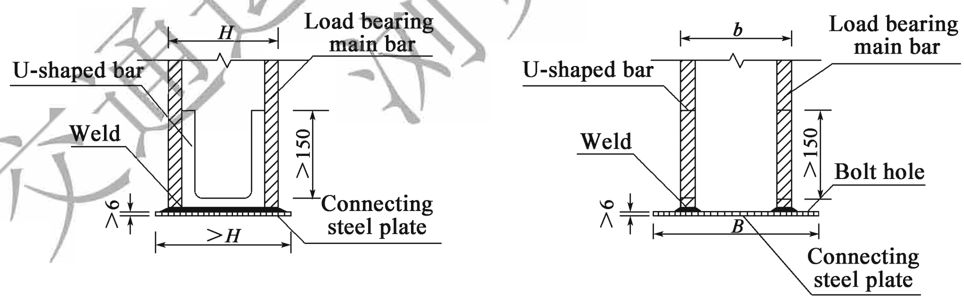


Fig. 8-6 Schematic of welding connecting steel plate to main bar of steel lattice frame (dimensions given in mm)

H-sectional height of steel rib support; *b*-sectional width of steel rib support; *B*-width of bearing plate

8.2.10 Where forepoling is installed, the tail end of the forepoling needs stronger support.

8.2.11 Due to the large number of influencing factors, support parameters for shotcrete and rockbolt lining design are frequently determined using engineering analogy method. Parameters presented in Table P.0.3 appended to the Specifications are summary of experience gained from

China's highway tunnel and underground engineering.

8.3 Monolithic lining

8.3.1The current monolithic lining in tunnel is a concrete or reinforced concrete structure cast by one continuous operation. It can be used independently in tunnel support structure, but more frequently it is used as secondary lining in composite lining. Monolithic lining generally has uniform section; variable section may be considered when it is subjected to unsymmetrical load or large vertical load. Where an arch invert is provided, to ensure effective connection of the arch invert to side walls, the arch invert thickness shall not be less than side wall thickness. The side wall thickness is the thickness less the space occupied by longitudinal drainage pipe. (Fig. 8-7).

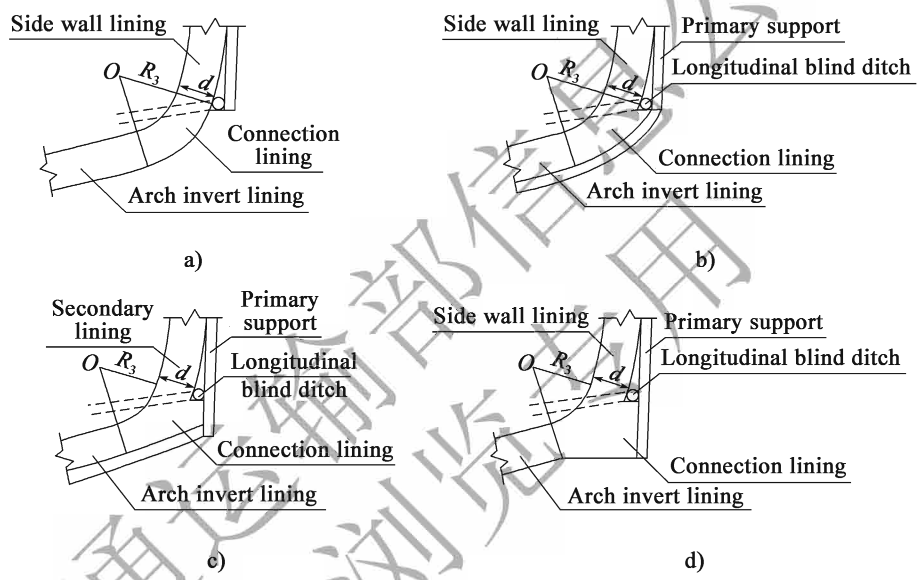


Fig. 8-7 Side wall lining shall be connected to arch invert

d- side wall thickness

8.3.2 In some special zones reinforced concrete structure is required because of its high bearing capacity.

1 In zones with obvious eccentric load caused by loosening and sliding of surrounding rock due to terrain and tectonic structure, such as mountain side tunnel, shallow tunnel whose axis intersects obliquely the contour lines, tunnel whose axis is parallel or nearly parallel to the orientation of steep dip rock formation, tunnel whose axis is parallel to vertical structure plane and zones with short-duration eccentric load resulting from construction operation, anti-eccentric load lining is employed in design to resist eccentric load from the surrounding rock. An unsymmetrically loaded tunnel has complex stress conditions and should employ reinforced concrete structure.

2 When a vehicular cross passage or ventilation channel intersects the main tunnel, an intersection cavern is formed. The scope of intersection expands to the arch. Due to large exposed area and complex stress in structure, the lining at the intersection should be reinforced concrete structure to ensure structural strength and prevent cracking.

3 In Class V surrounding rock whose self-stability is low, secondary lining needs to provide support as soon as possible because primary support of shotcrete and rockbolt has relatively low stiffness. On the other hand, primary support of shotcrete and rockbolt may lose some of its bearing capacity under long-term pressure from surrounding rock and the secondary lining needs to carry high loads from the surrounding rock. Based on tunnel measurements in China, the lining in Class V surrounding rock carries large loads and should be reinforced concrete structure.

4 The single-tube four-lane tunnel has a large cross-section and high mid-span moment of the cavern. To reduce structural dead weight, the structure thickness should not be excessive; it is more reasonable to use reinforced concrete structure.

5 Based on survey data on areas where earthquake took place, underground structures have good seismic capacity. In areas with seismic peak ground acceleration less than 0.20g, the effect of general earthquake on underground structures is insignificant. Due to insufficient data on high intensity seismic zones with a seismic peak ground acceleration greater than 0.20g, tunnel lining in these zones cannot avoid cracking or failure for sure when an earthquake takes place. In addition, surveys on seismic hazards suggest the reinforced concrete lining is not susceptible to collapse, falling off and other damages endangering traffic and pedestrian safety during earthquake. Therefore, in areas with a seismic peak ground acceleration greater than 0.20g, the lining in the approach section should be reinforced concrete structure.

8.3.3 When employing reinforced concrete structure, monolithic lining needs to meet basic requirements for lining structure.

1 With good impermeability and air tightness, concrete above C30 can effectively protect rebars in the concrete and slow carbonization of surface concrete.

2 If too thin, the structure cannot give full play to the role of rebars. Therefore, the lining thickness should not be less than 300mm.

3 Load bearing main bars spaced too closely in reinforced concrete structure will undermine the quality of lining concrete placing and reduce the bonding force between concrete and

rebar.

8.3.4 Settlement joint and Expansion joint are collectively termed deformation joint.

1 At tunnel portals, lining stress and ground bearing capacity vary greatly. In addition there are many influencing factors in the approach section. Most linings suffer obvious transverse displacement. Therefore, settlement joints shall be provided.

2 Different types of lining with varying thickness are used in different classes of surrounding rock. Different types of lining are subjected to different pressures from surrounding rock; ground bearing capacity is also different. Provision of settlement joint prevents shear failure resulting from differential settlement or deformation. Such difference is obvious in cave-in section, so settlement joint should be arranged.

3 In continuous weak surrounding rock with low ground bearing capacity, long-term loading effect may cause certain settlement deformation. The settlement joint is placed to accommodate these deformations.

4 The effect of cold shrinkage tends to result in cracking in the lining structure. To accommodate temperature change, the placement of expansion joint in zones with large impact from temperature change is to prevent cracking in the lining due to temperature stress.

5 The purpose of providing settlement and expansion joints is to separate structures with different bearing capacities and structures subjected to different pressures from surrounding rock so that the settlement deformation and stress deformation are independent of each other. It is common practice to provide an isolation layer of certain thickness in the deformation joint using asphalt wood board or bitumastic oakum or flexible materials with certain durability and waterproofing ability. The direction of loading on the structure is considered to be vertical perpendicular to tunnel axis. The settlement joint should be vertically placed perpendicular to tunnel axis. "Perpendicular to tunnel axis" means the perpendicular to tunnel axis on the plane.

6 Settlement and Expansion joints themselves can serve as construction joint. When at different locations, settlement and Expansion joints can be shifted to the location of construction joint to reduce a process step.

8.3.5 In zones with no arch invert, the ground bearing capacity is good, but the integrity of ground shall not be damaged by excavation of cable trench and side drain, leading to hollow footing of side wall and affecting ground bearing capacity. The side drain is generally some distance away from side basement. The bottom of side drain is below the bottom of cable trench. The excavation boundary of side drain shall be more than 500mm from side basement so as to ensure certain width for footing protection. The footing of end-wall tunnel portal wall is at great depth. The excavation of the pit for the portal wall may damage the base of tunnel lining side wall. The base of tunnel lining side wall shall be placed at the depth of the base of portal wall.

8.4 Composite lining

8.4.1 Composite lining consists of a waterproofing layer sandwiched between primary support and secondary lining. All expressways, Class I and Class-2 highways in China have employed composite lining. Many Class III highway tunnels also employ composite lining. Its structural stability, waterproofing and surface finishes can meet basic requirements for use in highway tunnels, with high adaptability to various geological conditions and mature technology. Composite lining has become the standard form of structure for highway tunnel linings.

1 Primary support is part of the composite lining structure. In design of shotcrete, rockbolt, reinforcing steel cage, steel rib support and other supports, parameters shall be determined based on engineering geological, hydrogeological, tunnel section size, overburden thickness, etc.

2 The use of monolithic cast-in-situ concrete lining with high stiffness, good integrity and smooth appearance as secondary lining meets basic requirements for appearance of highway tunnels.

3 Once the surrounding rock is excavated and exposed, some deformations will take place. To reduce the distortional pressure exerted on the lining, certain deformations of the surrounding rock is permitted to release some energy. Therefore in determining excavation size, certain amount of deformation shall be allowed for. Deformation allowance may be determined by computational analysis or engineering analogy method. Table 8.4.1 presents deformation allowance according to application in recent years in China and field tunnel construction monitoring and measurements. In Class I-II surrounding rock, the deformation is small and overcut is common. Therefore, deformation allowance is not considered. In Class III-IV surrounding rock, deformations will occur to different degrees, especially for weak surrounding rock (including shallow tunnel) where the conditions are complex and surrounding rock deformation is directly related to surrounding rock conditions, excavation method, support means and support timing. It is inappropriate to specify uniform deformation allowance, so the deformation allowance needs to be adjusted

during construction based on field measurements.

Deformation allowance is the permissible deformation of surrounding rock under support control conditions.

8.4.2 Due to characteristics of geotechnical engineering, the pressure on support structure from surrounding rock is not constant. It is related to rock mass property, structural stiffness and support time. Tunnel excavation method, support time and support stiffness have a big effect on structure stress. In actual application, it is very difficult to accurately know this pressure. Therefore, most tunnel support parameters are placed under dynamic design and determined by engineering analogy with the help of calculation. Primary support and secondary lining in a four-lane large section tunnel or in weak surrounding rocks require necessary check of strength. Tables P.0.1 and P.0.2 of Appendix P to the Specifications are proposed on the basis of the previous edition, according to statistics and actual application of design support parameters adopted in recent years for two-lane and three-lane highway tunnels in China.

8.4.3 For a tunnel in poor geological conditions or with a large span, the problem lies not only in structural strength design but also in how to execute. Compared with a smaller span tunnel, a large span tunnel carries large loads which increase quickly; excavation and completion of each structure take a long time; during construction necessary temporary support measures or auxiliary measures are required. Therefore, excavation method design, excavation sequence design and temporary support design are also required in parallel with lining structure design.

8.4.4 Weak flowing ground, swelling surrounding rock and high stress surrounding rocks keep deforming many years after completion of tunnel construction, so the effect of increasing surrounding rock deformation and pressure after lining completion shall be considered.

8.5 Cut-and-cover tunnel lining

8.5.1 A tunnel built by cut and cover method is called cut-and-cover tunnel. The extrados of cut-and-cover tunnel is usually covered with backfill material; it may also be exposed or partially exposed. The cut-and-cover tunnel is built for the following reasons:

1 The overburden above the tunnel is thin; surrounding rock conditions are not right for tunnel formation, resulting in difficulty in building tunnel using mining method. The cut and cover method is better than mining both technically and economically; it is more beneficial to environmental protection and it is easier to guarantee construction conditions, construction duration and safety.

2 Subgrade or at tunnel portals subject to unfavorable geological hazards which are difficult to control; zones that cannot be avoided due to tunnel alignment and where clearing would cause more severe distress.

3 Zones where important buildings/structures present on both sides of the road may be affected; cutting excavation would jeopardize the safety of buildings/structures or the noise and fume from future traffic operation would have serious effect on the users of buildings/structures.

4 Where a highway, railway, canal and other artificial structure crosses the road and this zone is unavoidable due to limitations from topographic, geological and route conditions, cut-and-cover tunnel structure may be used to replace road-over bridge, aqueduct, etc.

5 To conserve physical environment at the portal and reduce or prevent the harmful effect of portal excavation or side and heading slopes at the portal on tunnel portal, the tunnel is usually extended in the form of a cut-and-cover tunnel.

8.5.2 The cut-and-cover tunnel structure has an arch or rectangular shape.

1 From the standpoint of structure characteristics, cast-in-place arch cut-and-cover tunnel structure has better integrality and higher bearing capacity and can resist higher vertical pressure. In general, arch cut-and-cover tunnel should be adopted.

2 A closed arch structure has very high stiffness and a high capability to resist sliding.

8.5.3

2 The semi-cut arch cut-and-cover tunnel is subjected to unsymmetrical loading since the side of the lining next to the mountain needs to be backfilled with rock and soil. Thicker outside side wall and arch ring can increase the structure's ability to resist eccentric load. Where topographic conditions allow, counter pressure backfill or counter pressure wall may be employed to offset the effect of unsymmetrical loading thereby minimizing or eliminating the adverse effect of eccentric load on the structure.

3 When building a cut-and-cover tunnel in zones of loose soft ground such as soil layer, accumulative layer, backfill soil and loess or with higher lateral pressure, arch invert is required.

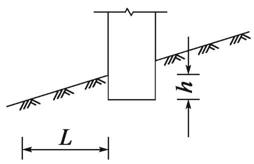
4 The cut-and-cover tunnel structure has some resistance to sliding. Coupled with other measures (such as lining thickening, surface drainage, load reduction, counter pressure, knee wall, anti-slide pile and subsurface French drain), it is an effective means to overcome landslide.

5 Settlement joint is placed to reduce damage to the structure resulting from differential stress or deformation. Expansion joint is placed in areas with highly variable temperature to reduce lining shrinkage, deformation and cracking. The spacing of settlement and Expansion joints is governed by the length of cut-and-cover tunnel, overburden thickness, temperature difference and geological conditions.

8.5.4

3 For the foundation for a cut-and-cover tunnel on a sloping ground, to ensure the stability of its base, the foundation needs to penetrate into stable strata and keep an appropriate horizontal distance from the edge of the outside stable strata. Minimum depth of cut-and-cover tunnel basement embedded into bedrock and minimum width of foundation protection are presented in Table 8-1. In cold areas, the cut-and-cover tunnel foundation shall be buried at least 250mm below the frozen depth.

Table 8-1 Minimum depth of cut-and-cover tunnel basement embedded into bedrock and minimum width of foundation protection

Type of rock formation	Embedment depth h (m)	Width of foundation protection L (m)	Illustration
Relatively intact hard rock	0.25	0.3	
General rock formation (such as sand-shale interbed)	0.60	1.0	
Loose soft rock (such as phyllite)	1.00	1.5	
Sand with gravel	1.50	2.5	

4 When designing a cut-and-cover tunnel for a highway along mountain and river, the possibility of bank scouring affecting foundation stability shall be taken into account; bank protection shall be provided according to topographic, geologic and water velocity conditions.

5 If the embedment depth of cut-and-cover tunnel foundation is more than 3.0m below the pavement, reinforced concrete transverse horizontal tie rod is arranged to reduce the foundation slenderness ratio and ensure the integrity and stability of the whole structure.

8.5.5 The cut-and-cover tunnel can prevent rockfall and collapse. It can also allow the alignment to pass beneath a highway, railway or canal; avoid debris flow and other hazards; protect natural landscape at the portal or meet style design needs. Its purpose dictates the thickness and gradient of backfill soil on top of the tunnel. Therefore, the thickness and gradient of backfill soil on top of the cut-and-cover tunnel shall be determined based on its purpose and functionality.

1 For a cut-and-cover tunnel designed to defend against rockfall and collapse, fill thickness on its extrados shall be sufficient to prevent the rockfall and collapsed material from directly acting on the arch ring. Based on decades of experience in construction of highway and railway tunnels, the fill thickness shall not be less than 1.2m [Fig. 8-8a)] and the gradient of backfill surface on top of the tunnel shall be such as to smoothly drain surface water away.

2 Exposed extrados of cut-and-cover tunnel has been widely applied in highway tunnels in recent years to very good effect in protecting natural landscape at the portal and beautify the environment [Fig. 8-8b)]. On the exposed extrados a screed or decoration layer of not less than 20mm is laid for waterproofing and aesthetic purposes.

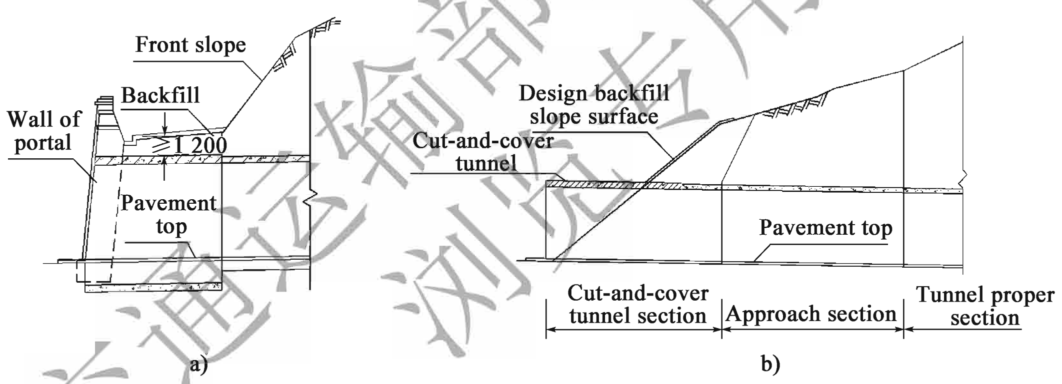


Fig. 8-8 Backfill for cut-and-cover tunnel (dimensions given in mm)

3 This provision addresses the special case of flyover cut-and-cover tunnel design. How or whether to place soil shall be determined flexibly by the designer according to tunnel function and realities of the structure and environment. The forepoling or elephant foot, depending where it is used can increase the bearing capacity of arch cut-and-cover tunnel and the protection capacity of the extrados itself. The forepoling or elephant foot, depending where it is used is generally constructed of stone pitching or concrete to a thickness of 0.8 ~ 1.2m.

4 For a drain ditch, canal carrying irrigation water for farmland, etc. above a cut-and-cover tunnel, the bottom of the ditch shall not be less than 1.0m from the outer edge of the

tunnel top so as to avoid adverse effect of drainage above the tunnel on the tunnel. If the drained water is mountain torrents or carries mud, consideration shall also be given to overflowing due to mud sedimentation and damage to ditch bottom and body from the passage of boulder.

8.5.6 Depending on the surrounding rock conditions, the side wall back of a cut-and-cover tunnel is excavated either vertically or in a sloped way from the wall base. The backfill between side wall and side slope shall be designed according to these two cases, depending on the type of cut-and-cover tunnel, class of surrounding rock and construction method.

1 For various cut-and-cover tunnels in Classes II, III and IV surrounding rock, side wall is generally excavated vertically, with sloping at top of the wall. The side wall shall be in tight contact with surrounding rock. The design shall take into account the elastic resistance of surrounding rock. If there is overcut at wall back, it shall be backfilled with concrete or stone pitching depending on overcut size to accommodate stress in side wall.

2 For a cut-and-cover tunnel in Classes IV and V surrounding rock, the wall back is generally excavated by sloping. In general, the friction angle of backfill at side wall back is not smaller than the calculated friction angle for the strata to avoid lateral pressure coefficient exceeding the calculated value. If active soil pressure at wall back of a cut-and-cover tunnel is calculated based on the calculated friction angle for strata, then the friction angle of backfill material at side wall back shall not be smaller than the calculated friction angle for the strata; however, if the design is based on calculated friction angle for backfill material, then the internal friction angle of backfill material shall not be smaller than the friction angle used in the design.

8.6 Structural requirements

8.6.1 Table 8.6.1 specifies minimum sectional thickness considering mainly the construction requirements of various materials to guarantee construction quality, including masonry structure lining for parallel adit, cross passage, auxiliary channel and auxiliary caverns and excluding shotcrete-and-rockbolt support. If the lining is constructed of reinforced concrete, minimum sectional thickness is the same as concrete.

8.6.2 The permissible maximum value of the angle between slope line and vertical line of extended foundation bench, α, varies with the type of foundation material. Based on test data from abroad and domestic experience in design and use of retaining wall, this provision specifies the rigid angle as 45° for concrete and 35° for masonry.

8.6.3 Tunnel lining structures are largely in direct contact with surrounding rock (or soil) and situated in an environment different from that for general surface structures. Their construction conditions are poor. Too thin a concrete cover will not be able to protect rebars from corrosion due to errors in rebar binding. Therefore, the specified thickness of the concrete cover is slightly greater than that for reinforced concrete structure on the surface. For a reinforced concrete member not in direct contact with surrounding rock, its cover thickness shall comply with the *Code for Design of Concrete Structures* (GB 50010-2010).

8.6.4 The purpose of specifying minimum reinforcement ratio for a compression member is to improve its brittleness property, avoid sudden failure of concrete and enable the compression member to have necessary stiffness and the ability to resist accidental eccentricity. When the load bearing main bar is HRB400, minimum reinforcement ratio shall be adjusted downward by 0.1%. It is to be noted that this adjustment is only for all load bearing main bars in the section whereas minimum reinforcement ratio of load bearing main bars on one side of the compression member shall remain not less than 0.2%.

8.6.7

1 Since the performance of rebars transferring load through joints is always poorer than a whole bar, the bar joints shall be arranged at the location with lowest stress.

2 The mechanical joint is added. After joined mechanically the bars shall have the same strength as prior to joining. When the diameter is greater than 25mm, lapping can no longer guarantee the same strength at the locations with joints and without joints. Therefore, welding or mechanical connection shall be adopted. As for the specification for lap length of plain bars with smaller diameter, the value is generally obtained from test per equal strength requirement.

8.6.8 Loads on axial compression reinforced concrete member with load bearing main bar and general stirrups are carried by the bars and concrete together. The purpose of specifying minimum reinforcement ratio is to make the member bear part of the bending moment and reduce the effect of concrete shrinkage and creep. In general engineering practice, bending moment is present in all axial compression members and can be borne by placing a specified number of bars so as to delay failure of the member. The specified ratio varies from country to country, ranging from 0.4% to 1.0%. The Specifications adopt 0.6% in line with the *Code for Design of Concrete Structures* (GB 50010-2010). Maximum reinforcement ratio is specified to avoid excessive density of bars which would hinder concrete placing and compaction. Minimum diameter of longitudinal bar and stirrups and maximum spacing of stirrups are specified to ensure sufficient stiffness of compression bars and certain safety reserves from longitudinal bending failure when the bars are under pressure. Meanwhile stirrups provide lateral constraint to concrete to increase their ultimate bearing capacity so that the member is protected from sudden failure.

8.6.9 Relevant specifications are added for reinforced concrete linings in tunnel according to characteristics of tunnel lining structure and the requirements of the *Code for Design of Concrete Structures* (GB 50010-2010).

1 The load bearing bar in tunnel lining refers to circumferential bars in the tunnel.

3 The tunnel lining distribution bars are bars arranged longitudinally on inner and outer sides of lining section along the tunnel; they are also called longitudinal bars.

4 Lining stirrups are generally single bars with hooks at both ends as shown in Fig. 8-9a).

5 In consideration of the constraints and structural stress of tunnel lining structure, stirrup type and role, stirrups shall be located at the intersection of load bearing main bars and distribution bars to constraint the load bearing main bars and distribution bars and arranged in quincunx form, as shown in Fig. 8-9b).

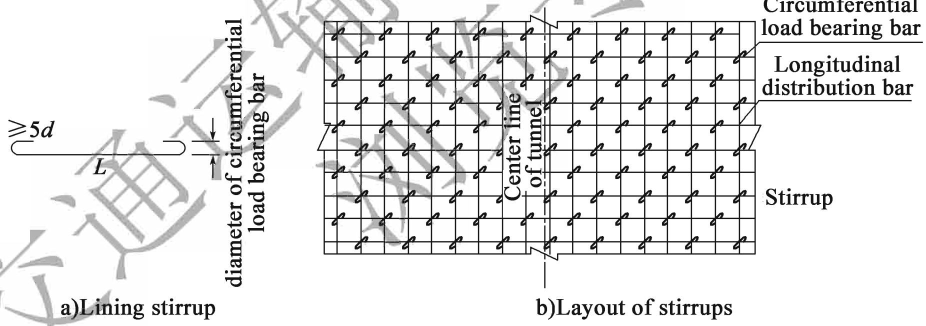


Fig. 8-9 Schematic of stirrups

6 Stirrups can limit "outward expansion" of the spacing of two layers of circumferential load bearing bars and distribution bars, but cannot limit the decrease in the spacing of two layers of bars on the inside and outside. In engineering practice, the spacing between the two layers of bars commonly decreases even to zero, reducing the role of the bars considerably. Therefore, this revision has added the specification for position limit bars to guarantee the spacing between inner and outer bars. The position limit bars play a primary role of supporting. Rectangular layout can ensure straight "support" without skew. Fig. 8-10 presents a form of position limit bars.

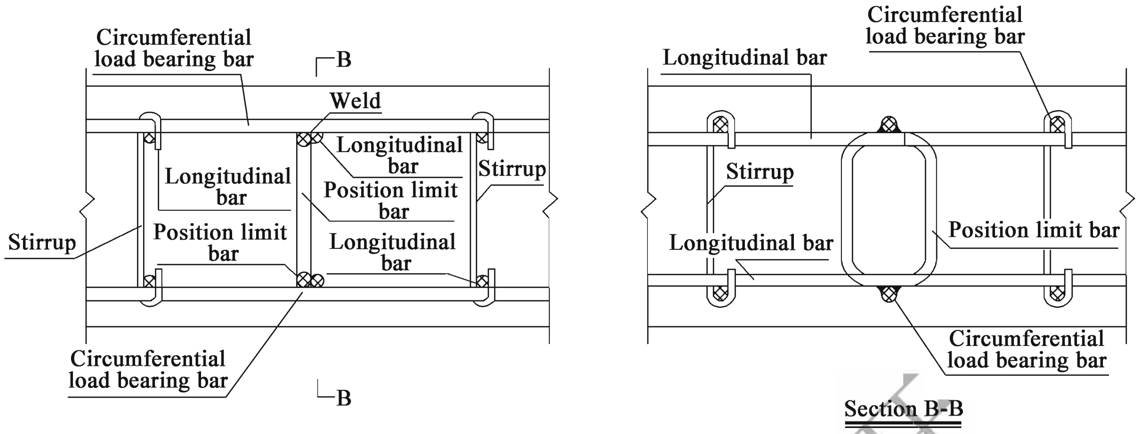


Fig. 8-10 Schematic of position limit bars

8.6.10 Cracks referred to in this provision are structural stress cracks caused by external loads such as rock load, water and soil pressure and dead weight of the structure, excluding cracks resulting from shrinkage, creep and temperature.

9 Structural Calculation

9.1 General requirements

9.1.1 In the field of structural design most engineering structures have been designed using probabilistic limit state design method to measure the reliability of structural member with reliable indexes and using equations expressed in partial coefficient. Due to the uncertainty of pressure from rock surrounding highway tunnels and lack of samples and special research results, the strength of member section is still verified by plastic stage design method. Limiting crack development width in tunnel lining is a basic condition to increase its service life. Therefore it is specified that in tunnel structural design calculation be carried out based on bearing capacity and the limit of crack development width. For a concrete member, reinforcement quantity is calculated based on the limit of crack development width when necessary.

9.1.2 In seismic design for mountain tunnel, calculation of structure seismic response is generally performed using static method and covered in this chapter.

9.2 Lining calculation

9.2.1 Theoretically monolithic lining (secondary lining) in composite lining shall be calculated by ground-structure method. Since there is experience in application of load structure method in the past, this provision specifies the load structure method may also be used.

9.2.2 Model test and theoretical analysis suggest deformations of loaded tunnel lining are constrained by surrounding rock, thereby improving the lining work state and increasing its bearing capacity. In load structure model design, the constraint effect of surrounding rock on lining deformation shall be considered. The effect of the tangential frictional force between surrounding rock and lining on the internal force in the lining is deemed to be safety margin of the lining structure.

9.2.3 Where arch invert lining is provided, the arch invert shall be constructed prior to side wall and arch ring. Consideration shall be given to the effect of arch invert on internal force in tunnel lining structure. The arch invert is calculated based on curved beam on elastic ground.

9.2.4 Safety factors listed in Tables 9.2.4-1 and 9.2.4-2 are based on survey and practical experience from 41 existing and nearly 400 newly constructed railway tunnels in China, and past engineering practices have verified that the structures are safe. Therefore, with no major changes in structural calculation theory and material indices, these safety factors can be considered to be appropriate. Given the characteristics of underground structure (such as poor conditions for lining construction, difficult quality assurance, highly variable loading and difference between structural calculation sketch and actual stress state), the selected value of structural strength safety factor shall be slightly higher than that for surface structure to ensure necessary safety margin for tunnel structure under normal design and construction conditions.

In checking the strength at construction stage, loads from rock load will generally not reach the maximum value at service stage because the duration at construction stage for the tunnel lining structure is much shorter than at service stage. In addition, in calculation assumptions for checking the strength at construction stage, a spatial structure with good characteristics of resisting loading is often simplified as a plane structure with large internal forces, ignoring or taking very low values of factors that are beneficial to lining stress, such as bonding strength of construction joint, elastic resistance of surrounding rock, frictional force and adhesion between lining and surrounding rock. Therefore, this provision specifies safety factors at construction stage may be obtained by applying a reduction coefficient of 0.9 to the value for service stage.

9.2.5 Properties of rock and soil media are characterized by obvious uncertainties. Engineering analogy is a commonly used design method. Since primary support is in tight contact with surrounding rock, primary support is mainly designed using engineering analogy method.

For two-lane and three-lane tunnels, experience shows Classes I-III surrounding rocks have high self-supporting capability and require only a thin layer of shotcrete and small amounts of rockbolts to maintain stability without the need for calculation, whereas for Classes IV and V surrounding rock checking calculation is needed after selection of support parameters based on experience. For a four-lane tunnel in Classes III-V surrounding rock, checking calculation is needed after selection of support parameters based on experience.

For checking of primary support, continuum mechanics finite element method can be used. Internal force and deformation can be calculated by design model of ground-structure method which can better simulate the effect of excavation procedures.

In calculation by ground-structure method, release factor may be applied to released load to control the force exerted on primary support so that primary support can share released loads with secondary lining as per a reasonable proportion. The proportion allocated to secondary lining shall be such to assure permanent safety of the support structure while that allocated to surrounding rock and primary support shall be such to assure safety in construction. For the selection of specific sharing proportion at service stage, reference may be made to Table 9-1.

Table 9-1 Recommended sharing proportion for released loads for a two-lane tunnel

Surrounding rock class	Sharing proportion	
	Surrounding rock + primary support	Secondary lining
IV	60% ~80%	40% ~20%
V	20% ~40%	80% ~60%

Notes:

1. If surrounding rock conditions are good, take high values for primary support and low value for secondary lining. If surrounding rock conditions are poor, vice versa.

2. When secondary lining has not been constructed at construction stage, the sharing proportion for checking the carrying of released loads by surrounding rock and primary support is higher than the value given in the table, up to 100%.

9.2.6 In calculating surrounding rock and primary support stability using ground-structure method, the concept of tunnel strength margin safety factor is introduced to better judge the stability of surrounding rock. During construction surrounding rock strength is reduced primarily by excavation, blasting or water seepage into surrounding rock and moist air, leading to tunnel failure during construction. During operation, tunnel stress generally varies little; for a deep tunnel, even variation in ground surface loads does not have significant impact on tunnel stability; distress typically results from reduced strength of surrounding rock due to water seepage or weathering. Therefore, strength margin safety factor may be applied. For a homogeneous tunnel, the strength margin safety factor means the ratio of actual rock/soil strength at the location of tunnel failure (fracture plane) to the failure strength

For the common Mohr-Coulomb material in the ground, the strength reduction safety factor ω can be defined as:

$$\tau = (c + \sigma \ tan\varphi)/\omega = c' + \sigma \ tan\varphi' \tag{9.1}$$

$$c' = c/\omega \tag{9.2}$$

$$tan\varphi' = (tan\varphi)/\omega \tag{9.3}$$

where

τ——shear strength of surrounding rock (kPa);
c——cohesion of surrounding rock (kPa);
σ——normal stress on shear sliding plane (kPa);
φ——internal friction angle of surrounding rock (°);
c'——cohesion of surrounding rock after reduction (kPa);
φ'——internal friction angle of surrounding rock after reduction (°).

In design calculation, three kinds of safety factor need to be considered for the tunnel. The first is surrounding rock safety factor after primary support is installed. This safety factor would directly affect construction safety. If surrounding rock safety factor exceeds 1.30 after installation of primary support, then surrounding rock and primary support are both safe and secondary lining can provide safety margin. If the safety factor is 1.15 ~ 1.30, then secondary lining needs to carry some load. If the safety factor is less than 1.15-1.20, then primary support is inadequate and cannot meet construction safety requirement without being reinforced or supplemented with other auxiliary measures. The second kind is surrounding rock safety factor after secondary lining is installed. This safety factor is higher than the first kind and can remain above 1.3. The third kind is the safety factor of secondary lining. If secondary lining serves only as safety margin, its thickness shall be determined based on experience. If secondary lining is designed to carry some load, then it shall undergo mechanical analysis.

9.2.7 For secondary lining of composite lining for a two-lane tunnel in Classes I-III surrounding rock, since primary support acting as permanent structure can maintain the stability of surrounding rock, the thickness of secondary lining may be selected per structural requirement without the need for checking calculation. For three-lane and four-lane tunnels with large cross-section in Class III surrounding rock, primary support and surrounding rock cannot assure stability, requiring the secondary lining to carry some load. The secondary lining shall undergo mechanical analysis as load-bearing structure, using the same calculation principle and method as for primary support in the same class of surrounding rock. Since there has been experience in calculation by load structure method, this method can also be used.

9.2.8 Given the randomness that characterizes mechanical property of formation lithology, parameter values selected based on geologic data or obtained in accordance with the specifications in engineering design are somewhat different from realities. Therefore, the Specifications stipulate that following tunnel excavation the parameters shall be corrected according to actual geological conditions at construction site and tunnel construction monitoring and measurement results.

9.2.9 The previous edition limits deformation of tunnel primary support through permissible relative convergence at tunnel perimeter. It is not easy to determine the permissible relative convergence at tunnel perimeter and its value varies among two-lane, three-lane and four-lane tunnels; and it is not necessarily correlated to tunnel stability. Therefore, the Specifications have canceled the concept of permissible relative convergence at tunnel perimeter. To assure tunnel structure thickness and cross-section size, deformation allowance is adopted instead to control deformations of tunnel primary support.

9.2.10 The eccentricity of eccentrically compressed members of cast-in-situ concrete lining and cut-and-cover tunnel lining is specified as "should not" for the following reasons:

1 In checking sectional strength of the lining, both safety factor and eccentricity requirements shall be satisfied. However, the safety factor requirement is generally easy to satisfy whereas eccentricity tends to exceed its limit. In this case if the arch axis is nonadjustable, the lining section thickness needs to be increased to satisfy eccentricity requirement. This is often unreasonable and sometimes impossible.

2 The tunnel lining has high stiffness and the space behind the lining is generally backfilled densely with good constraints. The lining structure typically will not lose stability due to large eccentricity.

3 In past calculation of linings, there was no problem when actually calculated eccentricity is slightly larger than permissible eccentricity, mainly because there is an interval between the lining service stage and failure stage. Therefore at the lining service stage allowing no occurrence of cracking, it is not necessary to excessively restrict eccentricity.

The purpose of specifying lining section eccentricity is to ensure proper selection of lining structural form and give full play to the compressive capacity of concrete. When eccentricity has exceeded a certain value, lining section will be controlled by tensile strength and have much lower bearing capacity. Therefore besides meeting strength requirement the eccentricity shall also be controlled.

9.2.11 Eccentric effect factor α is the ratio of the ultimate bearing capacity of a concrete member when under eccentric pressure to the ultimate bearing capacity of the same-strength, same-section-size concrete member when under axial pressure, to reflect the degree to which the member's ultimate bearing capacity reduction relative to axial compression as a result of eccentric pressure. Given the complexity of realities and the many influencing factors, the relationship between α and relative eccentricity e_o/h is in fact a random process that is difficult to fully summarize and express using simplified assumptions and theoretical calculation. A better way is to find its statistical characteristics through numerous tests. Values in Table 9.2.11-2 and calculation formula for α adopted in the Specifications are based on statistical characteristics from test results on concrete of various strength grades, 6 eccentricities and over 300 unsymmetrically loaded and uniaxial compression test specimens. The values of longitudinal bending coefficient ϕ presented in Table 9.2.11-1 are proposed by reference to the *Code for Design of Concrete Structures* (GB 50010-2010).

9.2.12 For the convenience of calculation, this provision derives the eccentricity, e_0 of 0.2h as the boundary between compressive strength control and tensile strength control for eccentrically compressed rectangular concrete member, based on concrete compressive and ultimate tensile strength (R_a, R_1), safety factor and eccentricity effect coefficient given in the Specifications,. This control boundary eccentricity is not intended as the actual boundary eccentricity of first failure either at the tension zone or the compression zone pre-failure, but the boundary eccentricity for assessing tensile or compressive control. It is not obtained from tests but from calculation. Its value varies with $N_{compression} - e_0/h$ and $N_{tension} - e_0/h$ curves, the ratio of tension to compression safety factors and R_a/R_1. At the control boundary eccentricity, tensile and compressive bearing capacities are equal; on either side of this boundary eccentricity is compressive control bearing capacity or tensile control bearing capacity (Fig. 9-1).

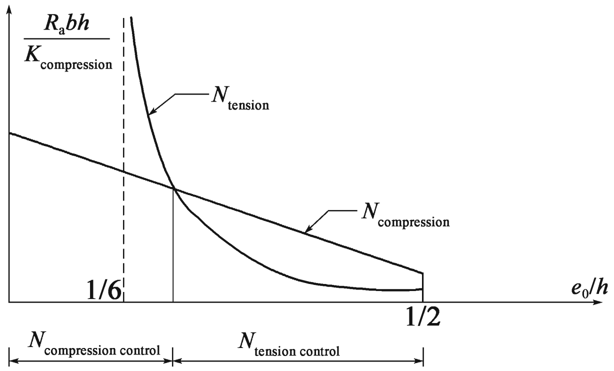


Fig. 9-1 Compressive and tensile control bearing capacities

In the figure:

$$N_T = \frac{1.75R_1bh}{K_T(\frac{be_0}{h} - 1)} \quad (K_T = 3.6)\ N_C = \frac{R_abh(1 - 1.5\frac{L_0}{h})}{K_C} \quad (K_C = 2.4);$$

For a rectangular-section concrete member, if $e_0 \leqslant 0.20h$, compressive strength controls the bearing capacity and it is not necessary to use this equation for calculation; if $e_0 > 0.20h$, the tensile strength controls the bearing capacity and it is not necessary to use Eq. (9.2.11) for calculation.

9.2.13 For intermittent placing of concrete or arch footing with masonry side wall and concrete arch ring, their characteristics are similar to masonry member section and may be considered as masonry, requiring checking only of its compressive strength. In fact cracking in masonry mortar joint does not affect the structure's function. If masonry quality is not good, cracks have long existed at mortar joints. Therefore the Specifications stipulate for an eccentric masonry member only compressive strength shall be checked and controlled to $e_0 \leqslant 0.3h$ so that the crack will not become too wide.

9.2.1 4In Appendix N to the Specifications, the formulae given in N.0.1 ~ N.0.10 are for calculation of reinforcement quantity in design by plastic stage design method. The formulae given in N.0.11 ~ N.0.12 are for calculation of reinforcement quantity in design based on limiting crack development width.

9.2.16 Deformations of flexural reinforced concrete members refer mainly to deflection and deformed steel bars.

9.2.17 Where the tunnel is constructed by sequential excavation method, the tunnel structure is in a dynamic process of structural transition, and the structural form and stress state during construction are very different from after construction. Therefore it is necessary to simulate and analyze structural safety during construction. During simulation consideration shall be given to actual structural form and stress conditions in the construction process.

9.2.18 For a tunnel where side slopes may suffer creep deformation or sliding, the approach section shall be able to resist the sliding force of side slope and the interaction between side and heading slopes and the tunnel structure shall be considered. In such case the lining in the approach section shall be reinforced to attain its required strength.

9.3 Calculation for cut-and-cover tunnel

9.3.1 When filling the overcut portion behind lined walls of a cut-and-cover tunnel with concrete or stone pitching, the effect of elastic resistance shall be considered; when it is filled with earth and rock, the effect of calculated soil pressure shall be considered.

9.3.2 The strength safety factor for cut-and-cover tunnel structure shall be taken as the same value to that for tunnel lining.

9.4 Calculation for portals

9.4.1 If end-wall tunnel portal is employed, the major external force acting upon the portal is soil pressure, so the portal wall can be deemed as retaining wall and calculated using the same method to that for subgrade retaining wall. In accordance with provisions on retaining wall design in the current Code for Design of Highway Subgrades (JTG D30), strength calculation for portal wall shall be performed using limit state design method with partial coefficient. For this reason, all relevant strength formulae, parameters (coefficients) for calculation, symbols and units shall conform to this code.

The portal wall check requirements presented in Table 9.4.1 are consistent with the check requirements for subgrade retaining wall. In checking portal wall, compressive stress and eccentricity are generally used for control. But in the case of a high portal wall (including high retaining wall for portal cutting), the sectional tensile stress shall also be controlled to avoid excessive tensile stress. The control value of tensile stress may be obtained by applying an appropriate safety factor (1.5 ~ 2.0 recommended) to the value of R_l in Table 5.2.5 herein. It is generally easy to satisfy base eccentricity $\leqslant B/4$, calling for no control design.

For checking of gravity retaining wall stability, the chances of overturning failure are higher than sliding failure in engineering practice, indicating the previous stability against overturning safety factor is low. Therefore, this revision changes it from the previous 1.5 to 1.6 in line with the current *Code for Design of Building Foundation* (GB 50007).

10 Waterproofing and Drainage

10.1 General requirements

10.1.1 “Prevention”: tunnel lining structure and waterproofing layer shall be waterproofed to prevent groundwater from entering the tunnel through waterproofing layer and lining structure.

“Drainage”: The tunnel has unobstructed drainage facilities to drain seepage and ponding behind the lining and below pavement structural layer into the center drain or side drain in the tunnel. Draining ponding behind the lining can reduce or eliminate water pressure behind the lining. The better the drainage, the less likely for the lining to leak water and the easier for waterproofing. Draining ponding below the pavement structural layer can prevent waterboil, frost boil from the pavement and structural failure.

“Interception”: provide interception drain to channel surface water, karst cave water and ponding in goaf that may leak into the tunnel. Surface water seepage shall be reduced by backfilling potholes, sealing surface leakage points and providing interception drain. Ponding in karst cave and goaf shall be drained away.

“Plugging”: In zones with rich rock fissure water, fault water and cavern water, groundwater in surrounding rock shall be contained by grouting and constructing cutoff wall to prevent or reduce groundwater loss.

Surface and ground water is often related with each other. Therefore tunnel waterproofing and drainage design shall properly address surface and ground water by taking reliable waterproofing and drainage measures taking into account tunnel lining structural design to create a complete, unobstructed water resistance and drainage system inside and outside the tunnel.

10.1.2 ~ 10.1.3 Wet stain: moist stain with obvious colour change on structure surface. On the surface the change in colour is visible but with no obvious feel of water when touched by hand; the water leakage is roughly equal to the amount of evaporation.

Water seepage: water seeps from lining concrete to form obvious, non-flowing water film on the inner surface of the lining. Water seepage results from inadequate compaction of lining concrete or adverse crack (wider than 0.2mm). Due to the small amount of water, a non-flowing, non-dripping water film is formed on the lining surface under the effect of concrete absorption and surface tension; its seepage flow rate from a single point is about 0.05L/d.

Water leakage: water seeps from lining concrete to form dripping water on the inner surface of lining arch or flowing water film on side walls. Water leakage results from large amounts of seepage water due to inadequate compaction of lining concrete or adverse crack (wider than 0.2mm), represented by dripping water at the arch and flowing water on side walls. According to observation, when dripping speed at a single location is higher than 300 drips per minute, it looks like a continuous small stream.

10.1.4 Large amounts of drainage may result in groundwater loss, reduction in local farmland irrigation and domestic water, loss of surrounding rock particles, formation of underground cavity and even surface collapse, reduction in surrounding rock stability and changes to water environment in the area. Water plugging measures for surrounding rock are intended to avoid and minimize the effect of tunnel construction on surrounding water environment, groundwater loss and the possibility of secondary disasters.

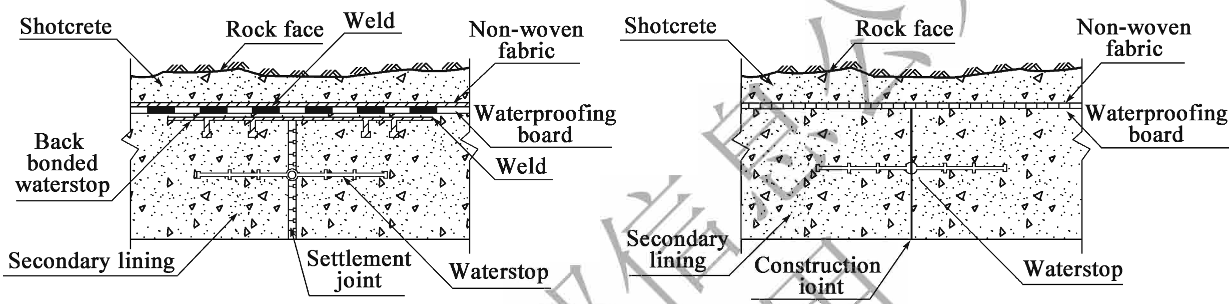
10.2 Waterproofing

10.2.1 Groundwater sources are mainly surface water. Common measures to reduce surface water percolation include drainage, pointing, paving and leveling creek, ditches and water pond on ground surface; paving the bed of water conveyance channel, hydraulic tunnel and other water conservancy facilities; and filling and sealing abandoned pit and borehole susceptible to ponding percolation.

10.2.2 Some tunnels in China have used waterproofing membranes bonded to non-woven fabric. Tests show the water filter and diversion performances of such waterproofing layer are reduced considerably. Therefore non-woven fabric should not be bonded to the waterproofing membrane.

10.2.3　Engineering practice in recent years has shown water seepage and leakage from cast-in-situ concrete in tunnel is very prominent, calling for further improvement of concrete resistance to seepage. In addition, with improved concrete placing process, its impervious performance has improved. Therefore this revision specifies " Grade of concrete impermeability should not be less than P8. "

10.2.4　Construction joint, expansion joint and settlement joint in cast-in-situ concrete lining are weak areas of water seepage and leakage prevention. Accordingly the provision prescribes " reliable waterproofing measures shall be taken ". Fig. 10-1 gives commonly used waterproofing configuration of construction joint and settlement joint in cast-in-situ concrete lining.



a)Settlement　　b)Construction

Fig. 10-1　Main configuration of settlement joint and construction joint in secondary lining

10.2.5　Where corrosive groundwater is present, different types of corrosion resistant concrete, anti-corrosion cement and waterproofing and drainage materials shall be adopted depending on the type of erosion. Increasing concrete compactness (impermeability) is an important means to increase its resistance to corrosion.

10.3　Drainage

10.3.1　After completion of the tunnel, groundwater in surrounding rock is generally clean, whereas washing water and fire water during operation is dirty. Separate discharge helps to reduce groundwater contamination. This restriction does not apply to short tunnels with little groundwater.

10.3.2

1　Side drain on both sides of the pavement facilitate channeling away operational washing water, fire water and other sewage to prevent such sewage flowing across the pavement.

3 Side drain are divided into open and covered side drain. Engineering practice shows the open ditch has small size, small discharge cross-section and shallow depth; garbage on the pavement tends to fall into the ditch and is thus difficult to remove and easy to block the ditch; it is easy to be damaged by vehicles and has low maintainability. Therefore, this revision has canceled provisions on "open ditch". Rectangular cove type side drain has a large discharge cross-section and high discharge capacity and is easy to clean since pavement garbage cannot enter the ditch easily. The rectangular cover type side drain is open drain and blind ditch (Fig. 10-2).

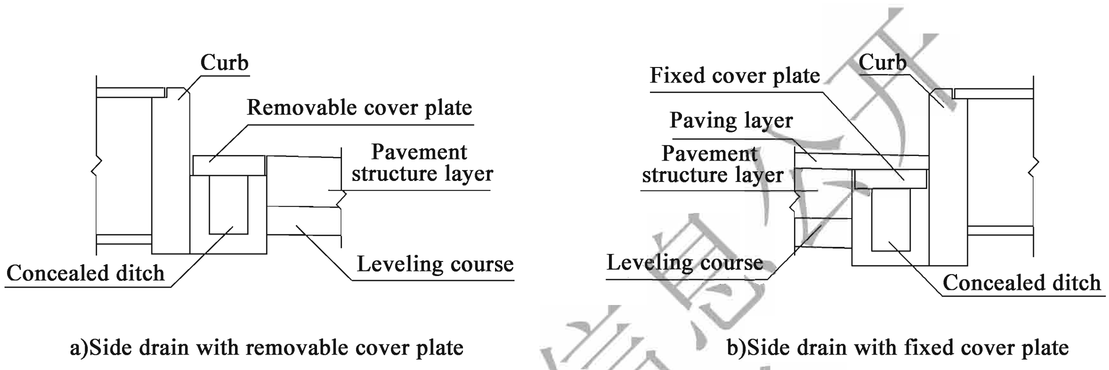


Fig. 10-2 Cover type side drain

5 Water in cable trenches will affect equipment operation and freeze in cold areas, and shall be prevented by taking appropriate measures. Years of experience shows 50mm × 50mm ~ 80mm × 80mm longitudinal trough provided at the cable trench bottom can collect water in the cable trench, which is directed into side drain by arranging drainage holes at 10-20m intervals along tunnel length between the cable trench and side drain.

10.3.3 The center drain below the pavement structure layer is intended to drain away ponding behind the lining and seepage water from the tunnel invert and also needed to separate clean water from sewage. In cold areas the center drain can provide thermal insulation and protection against freezing. Short tunnels depend on specific situations.

1 The center drain is intended to drain groundwater behind the lining and water from tunnel invert, which is generally clean water. The center drain is separately arranged from side drain without connectivity to avoid polluting clean water.

2 The center drain arranged in the middle of the tunnel is usually "single ditch" whereas the center drain arranged on both sides of subgrade is "double ditches", one on each side. For a two-lane tunnel, due to arch invert restriction and drainability requirement, "single ditch" is usually provided. If single ditch is employed, to avoid occupation of two lanes at the same time when maintaining the center drain, the ditch shall be arranged deviating

from carriageway centerline. For three-lane and four-lane large size tunnels, the middle of arch invert is very deep and construction of center drain is difficult when it is located in the middle of arch invert fill layer. In addition the transverse pipe from side walls to the center drain is long. Therefore for three-lane and four-lane large size tunnels, the center drain may be arranged on both sides.

3 Survey in recent years has found that sectional dimensions of the center drain are small, leading to full flow and inadequate drainability in rainy season. Therefore the sectional dimensions of the center drain shall accommodate maximum calculated discharge. From the standpoint of maintenance, sectional dimensions of the center drain should be large rather than small to facilitate dredging. Sectional shape of the center drain may be rectangular or circular. Rectangular ditch has large discharge cross-section and is difficult to be plugged and easy to clean. Its body and cover plate may be prefabricated. Use of rectangular section is therefore encouraged.

4 Manholes are required for center drain. They are generally located in the scope of pavement traffic, having some effect on traffic flow. Given the low frequency of maintaining and cleaning the center drain, manhole cover may be covered by the surface course of pavement to reduce the impact on traffic. Manholes are covered and open manholes. The cover of open manhole is flush with pavement surface course and easy for inspection and maintenance, but affects ride comfort. The cover of an open manhole is lowered and concealed by pavement surface course. When local plugging of the center drain calls for inspection, the pavement surface course on the cover of the well can be broken for inspection. The location of concealed manholes shall be clearly marked on side walls of the tunnel. The spacing of settling ponds and manholes shall be determined by surrounding rock geological conditions, groundwater characteristics and maintenance conditions. The spacing shall be large if fault fissure water and groundwater is clear and small in the case of karst water and groundwater susceptible to crystallization. Sedimentation pond is usually combined with manhole.

10.3.4 This provision relates to drainage under pavement.

1 The pavement bed course (leveling course) or arch invert fill course top is in contact with the underside of pavement structure. Certain transverse drainage gradient is beneficial to drainage at pavement bottom. Where a center drain is provided, tilting toward the center drain helps rapid drainage of groundwater. Where no center drain is provided, the transverse drainage gradient of bedding course (leveling course) or arch invert fill course shall be consistent with pavement.

2 The transverse filter pipe is arranged between the bottom of pavement structural layer and the top of bed course (leveling course) or arch invert fill course within the scope of pavement, at the construction joint of the bed course or arch invert fill course and water emission location at tunnel bottom to facilitate drainage at pavement bottom. In zones of abundant groundwater, transverse filter pipes shall be spaced closer longitudinally.

5 The filter pipe consists of highly permeable pipe into which groundwater flows and is drained through the pipe to side drain or center drain. In general, steel ring reinforced drainage pipe is adopted.

10.3.5 The tunnel lining drainage is shown in Fig. 10-3 and 10-4.

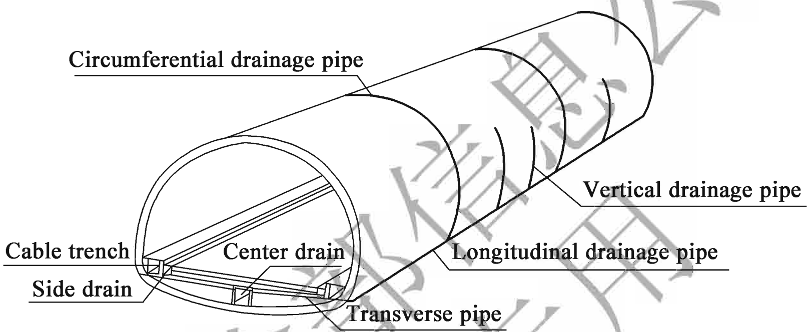


Fig. 10-3 Schematic of tunnel lining drainage

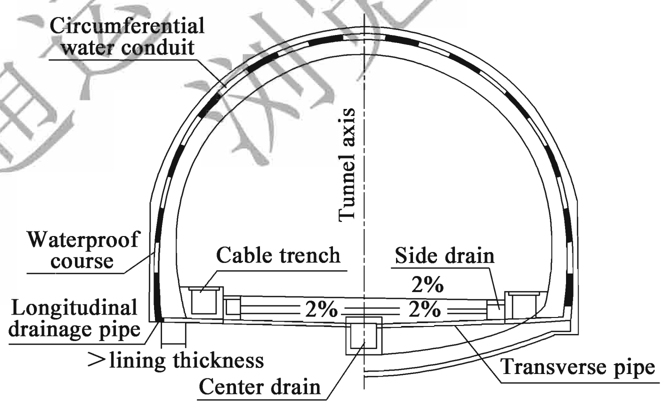


Fig. 10-4 Cross-section of tunnel lining drainage

1 The longitudinal drainage pipe behind secondary lining is mostly permeable round pipe of 100 ~ 120mm diameter. Permeable filter pipes with other sectional shape are also used.

2 Circumferential and vertical drainage channels between waterproofing layer and primary support are intended to quickly direct groundwater behind the lining to side basement to

prevent water accumulation behind the lining. The drainage channels may consist of circumferential filter pipe, vertical filter pipe, sheet drain and water drainage board. When using circumferential and vertical filter pipes, the spacing is generally not greater than 10m and diameter not less than 50mm, depending on volume and area of water inflow. One drainage channel is required every 10m even if groundwater is absent during construction because groundwater behind the lining may change after tunnel lining is completed. At the location of concentrated inflow from surrounding rock, pipe shall be directly inserted for drainage. Circumferential and vertical filter pipes are connected to longitudinal drainage pipe at the base of side walls to form a complete drainage.

3 The transverse pipe is intended to channel groundwater behind the lining out and into the center drain or side drain. It needs to pass through secondary lining and the bottom of cable trench, and to be buried below pavement structure layer or in the arch invert fill course where center drain is provided. Different from transverse drainage pipe, it is usually a closed impermeable pipe.

10.3.6 If the groundwater volume is high, blind ditch and center drain alone are insufficient to drain the groundwater. In this case intercepting and drainage facilities shall be provided based on actual conditions or auxiliary passage and drainage tunnels used to eliminate groundwater hazard to tunnel lining structure and reduce the impact on safety of traffic in the tunnel.

13.3.7 If the water inflow in the tunnel is predicted to be high, the discharge section of center drain and side drain shall be enlarged wherever possible in design to accommodate instantaneous maximum flow in extreme weather and avoid expansion of the discharge section due to insufficient capacity of the side drain and center drain.

10.4 Waterproofing and drainage for portals and cut-and-cover tunnel

10.4.1 When there is a large catchment area above the excavation of side and heading slopes at the portal or natural gulley that may cause adverse scouring to side and heading slopes, intercepting drain is provided to drain surface water on side and heading slopes away from the portal. When the tunnel portal is located at mountain mouth, and the catchment area above the heading slope excavation is not large enough to cause adverse scouring to side and heading slopes, intercepting drain is not required or may be provided in sections. Given its possible impact on side and heading slope landscape, the intercepting drain shall be concealed in grass or woods wherever possible according to topographic conditions. Rectangular intercepting drain requires less excavation and does less damage to the landscape than one with trapezoidal section.

10.4.3

1 As with lining in the tunnel, the cut-and-cover tunnel lining requires waterproofing layer. For this purpose, the externally bonded waterproofing membranes are easy to install and assure quality and effectiveness.

2 The junction of cut-and-cover tunnel and mined tunnel is often the weak area of water seepage and leakage, requiring overlapping of waterproofing layers in a sealed way.

3 To prevent infiltration of surface water, clay waterproofing layer laid on top of backfill soil is intended to reduce rainwater percolation. The lap joint between the drainage layer and side slope is often water infiltration channel and requires proper connection with side slope. Laying 200mm thick planting soil on the clay drainage layer is to prevent loss of water resisting function in dry season when the clay cracks. This also facilitates grass and tree planting.

4 Side drain on top of backfill soil above cut-and-cover tunnel is an effective measure to prevent water accumulation on the extrados. For an end-wall tunnel portal, the drain ditch on top of backfill of the cut-and-cover tunnel shall be located at back of the end wall, the junction of the backfill face and excavated slopes or other locations as needed. For a cut-and-cover tunnel portal, the drain ditch may be located at heading slope platform and the junction of backfill face and excavated side and heading slopes. The drain ditch is generally rectangular with a cross-section not less than 200mm × 200mm.

5 If the extrados of the cut-and-cover tunnel is exposed, shall be provided with waterproof mortar layer or ceramic tiles for waterproofing, protection and decoration purposes.

10.5 Waterproofing and drainage for tunnels in cold regions

10.5.1 In areas with coldest-month average temperature of -10 ~ -15℃ and maximum frozen depth of 1.0 ~ 1.5m, side drains are at limited depths and susceptible to freezing, thus requiring center drain. In areas with coldest-month average temperature of -15 ~ -25℃ and maximum frozen depth of 1.5 ~ 2.5m where the center drain is not enough to meet drainage and cold-proof requirements, cold-proof drainage tunnel shall be provided. The cold-proof spillway has proven effective for Dabanshan Tunnel and Gonghe-Yushu Expressway Tunnel in the Alpine region of Qinghai.

11 Special Types of Tunnels

11.1 General requirements

11.1.1 ~ 11.1.2 Variations in layout plan and structural form from ordinary separated tunnels have led to special forms of tunnels including twin tunnels with small clearance, twin-arch tunnels, branching-out tunnels, shed tunnel, overlapping tunnels, underground tunnel interchanges and spiral tunnel. These special tunnels vary in applicable conditions, structural features, construction difficulty, environmental impact and construction cost. Therefore on the premise of fulfilling functional requirement, the form of tunnel shall be determined properly taking into account terrain, geology, economy, environmental protection, maintenance cost and other factors. Given the many successful cases of special tunneling technology, this revision has added technical content on branching-out tunnel and shed tunnel, but not including overlapping tunnel, underground tunnel interchanges and spiral tunnel.

11.2 Twin tunnels with small clearance

11.2.1 Twin tunnels with small clearance is two tunnels with so small a clearance between the them that they impose adverse effects on each other. Whether to design the tunnel structure as twin tunnels with small clearance depends on comprehensive analysis based on realities and the presence/absence of adverse effect between the two tunnels. This effect is closely related to tunnel cross-section size, surrounding rock classification, formation lithology, tectonic structure, construction method and excavation sequence. Due to its high cost and long construction duration, twin tunnels with small clearance is generally not used except at portals with narrow space and for medium or short tunnel restricted by surrounding buildings or to reduce land occupation. In local sections at the portal of long and extra-long tunnels with bridge-tunnel connection, twin tunnels with small clearance have also been adopted.

11.2.2

1 Use of composite lining structure is a basic requirement for twin tunnels with small clearance. The twin tunnels with small clearance may employ a wide range of clear distance, with different mutual effect between them and varying levels of construction difficulty. It is therefore inappropriate to give universal lining support parameters. These parameters shall be based on engineering analogy and computational analysis according to surrounding rock class, clear distance, construction sequence and excavation method. Based on research findings from the Design and Construction Key Technologies for Twin tunnels with small clearance in Fujian Section of Beijing-Fuzhou Expressway and the Design and Construction Key Technologies for Two-tube Twin tunnels with small clearance, Table P.0.4 in Appendix P gives support structure parameters for twin tunnels with small clearance for reference.

2 The rock mass between the two tunnels is called rock pillar. Keeping its stability is necessary to tunnel stability. In twin tunnels with small clearance design, construction sequence and temporary support measures shall be identified depending on terrain, lithology, structure and formation attitude. Based on years of engineering experience, if the secondary lining of the first tunnel is more than 2.0 times the tunnel diameter ahead of the face of following tunnel, the mutual disturbance between the two tunnels is small, so is the disturbance to the rock pillar. Decisions on whether to reinforce the rock pillar depend on its stability. The rock pillar may be reinforced by increasing the length of systematic rock bolting, installing cross tie, tremie grouting, etc.

3 The length of tunnel where the clear distance between them is less than 0.8 times the excavation span of unsupported tunnel should not be greater than 1,000m. This is based on considerations of construction difficulty, construction cost, duration and future maintenance cost.

11.3 Twin-arch tunnel

11.3.1 When their artificial structures are connected together, two arch tunnels running in parallel are called twin-arch tunnel. According to engineering practice on twin-arch tunnel in recent years, many problems remain to be solved for twin-arch tunnels. This kind of tunnel form is generally not adopted due to its high cost, long duration, and numerous distresses after completion which are hard to control. It is a last resort in some special zones where the space is narrow, alignment is difficult or tunnel portals are restricted by large structures/buildings.

11.3.2

1 According to the form of rock pillar, twin-arch tunnels can be divided into monolithic rock pillar and composite rock pillar tunnels.

① The monolithic rock pillar type is shown in Fig. 11-1. The rock pillar is cast as a whole and the secondary arch ring lining of the tunnel on both sides is supported by the rock pillar.

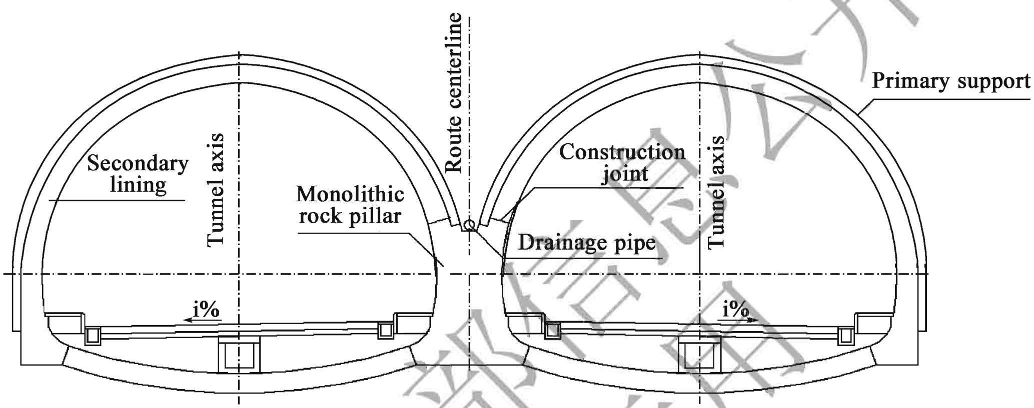


Fig. 11-1 Twin-arch tunnel with monolithic rock pillar

② The composite rock pillar type is shown in Fig. 11-2. The rock pillar actually consists of 3 parts, with core in the middle to support the center heading and primary support for the two tubes. Concrete on both sides of the rock pillar is placed with secondary lining for tunnel arch wall in the same way as for single-tube tunnel.

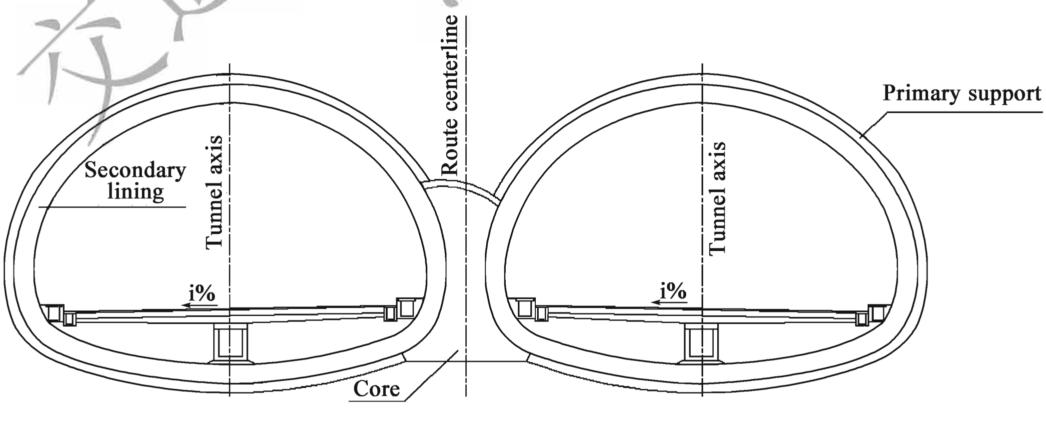


Fig. 11-2 Twin-arch tunnel with composite rock pillar

To construct twin-arch tunnels, a center heading generally needs to be excavated first. Where monolithic rock pillar is adopted, the top of the pillar and crown of the center heading are usually not backfilled to full compactness and cannot provide effective support to surrounding rock at the crown of the center heading. When excavating the main tunnel, the actual excavation span is wider than the main tunnel itself, leaving rock surrounding the tunnel in an unfavourable stress state. Primary support for monolithic rock pillar and secondary lining arch need to rest on the pillar top; the waterproofing layer needs to bypass primary support and is thus not continuous. Meanwhile, the longitudinal construction joint in secondary lining at top of rock pillar forms a weak area of waterproofing, resulting in leakage. Where composite rock pillar is adopted, the top of the pillar can be backfilled densely and in tight contact with the top of the center heading. The excavation span of main tunnel is relatively small and beneficial to surrounding rock stability. Only primary support of main tunnel rests on top of the rock pillar; the secondary lining and waterproofing layer are the same as for separated tunnels; the construction process is simpler; quality is easy to control; waterproofing and drainage are complete. Therefore composite rock pillar should be adopted.

2 Support parameters for twin-arch tunnels shall be based on engineering analogy and computational analysis according to surrounding rock class, clear distance, construction sequence and excavation method. When determining support parameters using engineering analogy, reference may be made to Tables P.0.5 and P.0.6 in Appendix P.

3 Survey in recent years finds cracking is common in the secondary lining of twin-arch tunnels. The use of reinforced concrete structure is beneficial to crack control.

4 Since the twin-arch tunnel structure is under complex stress and involves many construction steps, the surrounding rock stability and structural stress are more closely related and sensitive to construction sequence and excavation method. Therefore the design shall identify construction method and temporary support measures based on geologic conditions and cross-section size. Based on past experiences and lessons, the proper construction method for twin-arch tunnels is: first excavate center heading and cast rock pillar; then excavate main tunnel, with working faces of left and right drifts staggered and the secondary lining of the pilot tunnel more than 2 times tunnel diameter ahead of the tunnel face of following tunnel headings. Analyses of adverse structure stress and surrounding rock stability during construction are intended to know and predict the trend of surrounding rock stress and deformations so that appropriate countermeasures can be taken.

5 Construction of twin-arch tunnels with eccentric load shall follow the principle of "performing work on the outside before inside and doing difficult work before easy work" according to research findings and engineering practice on some twin-arch highway tunnel works in China. Constructing the tunnel on the outer side of eccentric load first is more beneficial to construction safety.

6 For twin-arch tunnels with monolithic rock pillar, waterproofing and drainage on top of the

pillar is difficult. Longitudinal drainage pipe on top of the pillar is intended to drain ponding from the top of the pillar. Vertical drain inside the pillar is intended to channel water in the longitudinal drain on the pillar top into side drain.

7 The axis of pilot tunnel deviates from route centerline so that the center heading crown can be filled with concrete, as shown in Fig. 11-3.

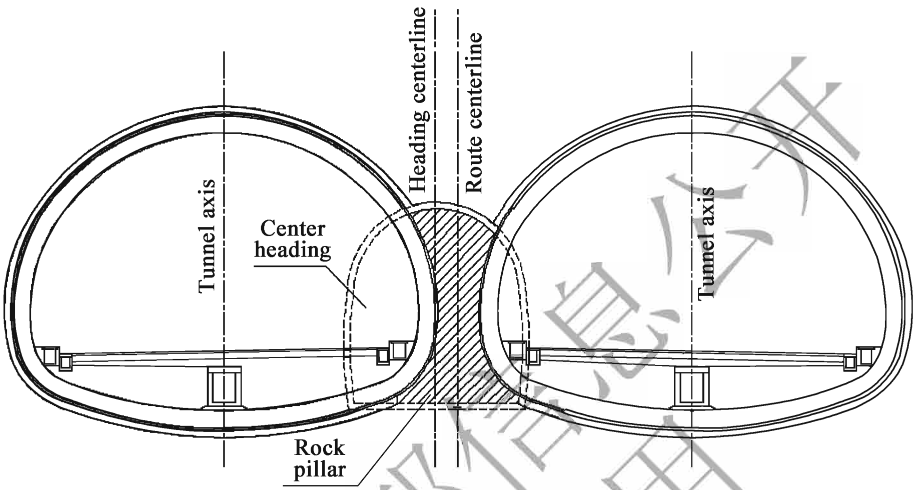


Fig. 11-3 Arrangement of offset from Center of heading

8 Due to structural integrality of twin-arch tunnels, deformation joints in left and right tunnels and in central pillar shall be located at the same section.

9 Due to staggered construction of left and right tunnels of twin-arch tunnels, the thrust on central pillar from the main tunnel arch structure is unsymmetrical during construction. Actions need to be taken to reduce the damage done to the pillar by unbalanced thrust.

10 From experience in constructing twin-arch tunnels across China, it is mostly adopted for tunnels less than 500m in length. Taking into account various factors, it is specified that twin-arch tunnels are suitable mainly for short tunnels.

11.4 Branching-out tunnel

11.4.1 Branching-out tunnel is a new form of tunnel developed during construction of mountain expressway under more complex topographic and geologic conditions. It transitions from a bidirectional large span tunnel, twin-arch tunnels or twin tunnels with small clearance to a separated twin tunnels and has the characteristics of separated tunnels, twin tunnels with small clearance, twin-arch tunnels, large span tunnels, etc.

According to the layout of the branching-out section, the branching-out tunnel can be divided into two types: ① a bidirectional large span tunnel in the portal section transitioning to twin-arch tunnels with monolithic central pillar, twin-arch tunnels with composite rock pillar, twin tunnels with small clearance and separated twin tunnels (Fig. 11-4). When the centre median at the tunnel portal is less than 1.4m wide, the portal section should be designed as large span tunnel; when it is 1.4 ~ 3.5m wide, the portal section may be designed as large span tunnel or twin-arch tunnels with monolithic rock pillar. ②twin-arch tunnels in the portal section transitioning to twin tunnels with small clearance and twin tunnels with one for up traffic and the other for down traffic (Fig. 11-5). This layout may be considered when the centre median at the tunnel portal is around 2.5m wide.

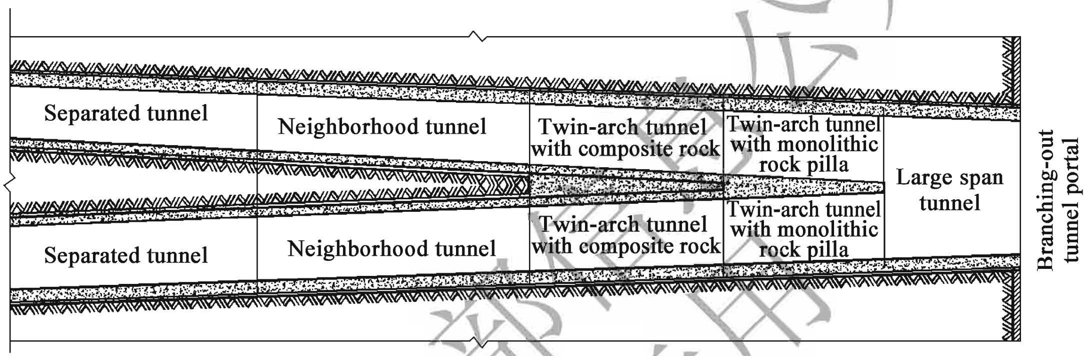


Fig. 11-4 Layout plan of type I branching-out tunnel lining

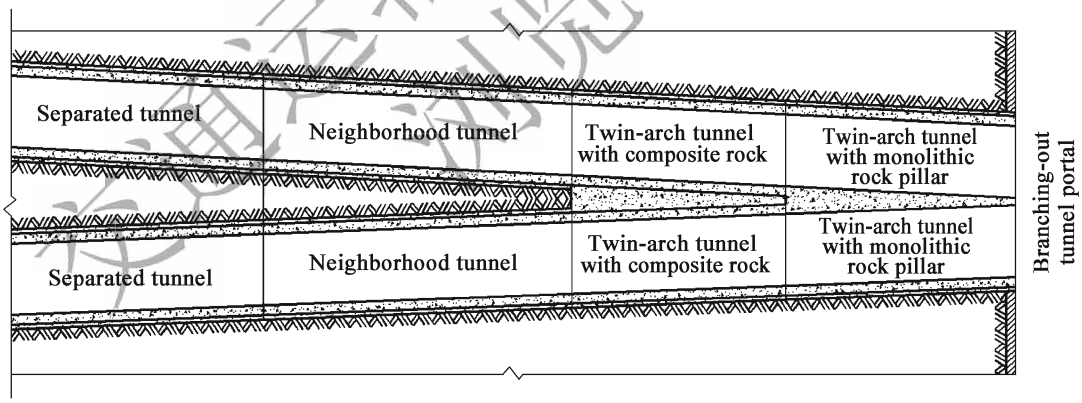


Fig. 11-5 Layout plan of type II branching-out tunnel lining

If the centre median at the tunnel portal is more than 4.0m wide, the portal section may be designed as twin tunnels with small clearance which transitions to separated twin tunnels.

If the portal section is relatively long in Class V surrounding rock or for a bidirectional eight-lane tunnel, the adoption of branching-out tunnel involves high construction risk and is not economical.

11.4.2

1 Horizontal alignment design for branching-out tunnel shall take into account terrain at the portal, geologic conditions, structures outside the tunnel, carriageway width, mutual influence of ventilation between left and right tunnels, other structures nearby, etc. When the horizontal alignment of route centerline in the portal section is curved, the horizontal alignments for left and right tunnels shall be curved in the same direction to facilitate gradual separation of the two tunnels within a short distance. When the horizontal alignment of route centerline in the approach section is straight, "S"-curve with a small deviation angle is needed to facilitate separation within a longer distance.

2 Once tunnel horizontal alignment is established, the length of branching-out section is basically finalized; the division of structure boundary in the branching-out section is determined based on geologic conditions, proposed construction method and construction experience, subject to adjustments during construction according to the conditions of exposed surrounding rock, construction method, duration requirement and the technical skills of the contractors. Under the same topographic and geologic conditions, the levels of construction difficulty, construction safety, structural stress complexity and construction cost will be more for bidirectional large span tunnels than for twin-arch tunnels; more for twin arch tunnels than for twin arch tunnels with small clearance; and more for twin arch tunnels with small clearance than for separated tunnels.

3 The branching-out tunnel has a large span and complex structure and is located at the tunnel portal where geologic conditions are typically poor, the influence of topographic conditions is great and design experience is insufficient. For structural design, the most unfavorable section shall be selected to perform surrounding rock stability analysis and structural check.

4 The construction and excavation methods for each type of support structure shall be defined, with emphasis on staggered excavation of left and right tunnels. The requirements for structural stress and surrounding rock stability analyses under various adverse conditions during construction are the same as for twin-arch tunnels.

5 At the structural boundary within the branching-out section, the structural form changes abruptly involving many transition procedures, the structure stress is complex and the cavern stability problem is more prominent. Therefore, appropriate structure reinforcement is required.

6 If the portal section is shallow, cut-and-cover tunnel structure may be adopted for

bidirectional large-span section and some twin-arch tunnel sections to take advantage of its clear structural stress and convenience of construction.

7 At locations of changes in structural form, deformation joints are required due to varying stress conditions and inconsistent deformations.

8 Bulkhead walls shall be designed in conjunction with waterproofing considering its support to surrounding rock to ensure effective connection to the lining structure.

11.4.3 Due to the proximity of two openings of a branching-out tunnel, the exhaust released from one opening might be absorbed by the other opening when longitudinal ventilation is employed. To avoid this, a dividing wall is provided for the large span tunnel along the subgrade dividing strip within the tunnel and a certain extended length outside the tunnel at the portal; for twin arch tunnels or twin tunnels with small clearance the left and right tunnel portals are staggered. A separate exhaust vent may be provided if conditions allow.

11.5 Shed tunnel

11.5.1 A shed tunnel is a "canopy" built along the highway with one side next to a mountain and the other open space. The side next to the mountain consists of a protective wall and the open side consists of columns, frames or arch windows. The roof top is closed and backfilled with rock and soil to form a semi-bunker structure. The shed tunnel has been applied in recent years. Its functions include:

(1) Prevent frequent weathered materials and collapsed materials on the side next to the mountain from falling directly to the highway, such as Moxi Shed tunnel in Sichuan, Daqi Shed tunnel of National Highway 317 and the shed tunnel at the portal of Xiushan Tunnel of Chongqing-Huaihua Expressway.

(2) Protect the environment, reduce extent and height of excavation on the side slope, minimize damage to mountain vegetation from highway construction and maintain mountain slope stability; backfilling on the roof of shed tunnel with planting soil enables the reinstatement of vegetation and creates beautiful landscape, such as Laoshan Shed tunnel in Nanjing.

(3) Prevent avalanche and snowdrift from accumulating on the pavement and blocking traffic so that the highway remains unobstructed, such as Haxilegen snow-proof corridor on Tianshan National Highway 217 in Xinjiang.

11.5.2 According to stress characteristics, the shed tunnel structure is divided into frame structure, simply supported structure and integral structure; according to the geometry of superstructure cross-section, it is classified into arch shed tunnel, semi-arch shed tunnel and frame shed tunnel. Common forms of shed tunnel classified by geometrical shape are given in Fig. 11-6.

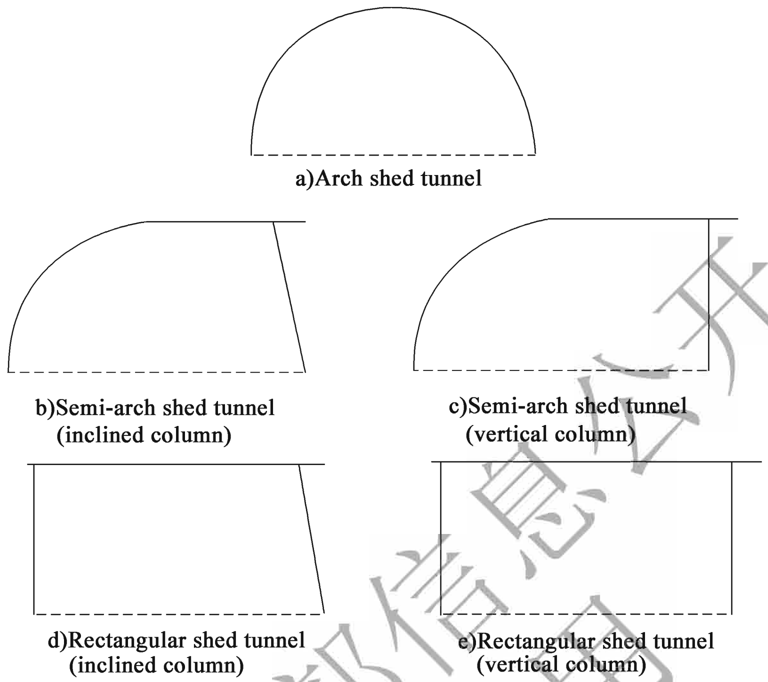


Fig. 11-6 Schematic of structural forms of shed tunnel

(1) Arch shed tunnel is generally a monolithic construction, with extrados on the side next to the mountain backfilled densely to provide certain support to side slope and the open side consisting of pattern lattice.

(2) Semi-arch shed tunnel is generally an integral structure. Its side next to mountain is arch shaped; the open side consists of vertical or inclined columns. The outside of tunnel roof is flat. The extrados shall be backfilled densely to provide some support to side slope. The tunnel roof shall be covered with rock and soil or planting soil.

(3) For frame shed tunnel, the side next to mountain consists of retaining wall against the rock; the open side consists of vertical or inclined columns . The tunnel roof may be a simply supported structure or connected to side wall to form a frame structure. The upper part is flat roof or single pitch roof. The roof shall be covered with rock and soil or planting soil.

11.5.3 The construction gauge of a shed tunnel at or near tunnel portal shall be the same as that of the tunnel. A shed tunnel is typically short. For an independent shed tunnel on expressway far from a tunnel, its construction gauge width may be the same as carriageway width so as to maintain smooth traffic flow and traffic safety.

11.5.4 Backfill behind the inside wall of a shed tunnel constraints the side slope and is able to resist lateral thrust of side slope. Therefore, slope protection design within the height of shed tunnel needs to consider the retaining role of the shed tunnel structure and horizontal thrust on the shed tunnel structure from slope deformations. If the shed tunnel is designed to protect against falling material, accumulation of fallen materials on its top is permitted; backfill and surcharge loads are considered as permanent loads. For a shed tunnel with thick backfill and accumulated material, the accidental impact of rock falls may be neglected.

11.5.5 Main structure of shed tunnel mainly resists upper surcharge load and lateral load from mountain deformation. The stability and strength of main structure of the shed tunnel shall be calculated on the basis of full analysis of shed tunnel stress conditions, according to engineering geological and topographical conditions, the type of shed tunnel and available engineering experience.

11.5.7 Requirements for shed tunnel foundation design are consistent with those for cut-and-cover tunnel. If the outside of the shed tunnel consists of vertical columns, stringers are provided to increase connection between vertical columns.

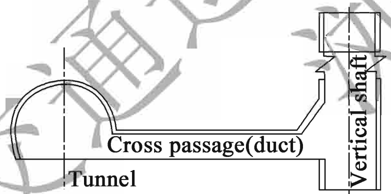
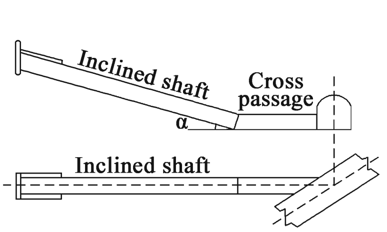
11.5.9 French drain behind the wall on the side next to mountain and drainage holes at the foot of the wall are intended to prevent water accumulation behind the wall.

12 Auxiliary Channel

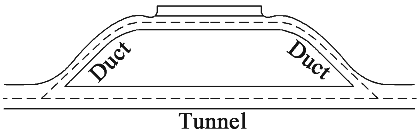
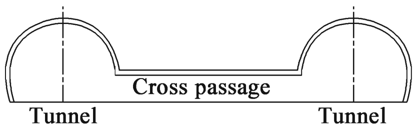
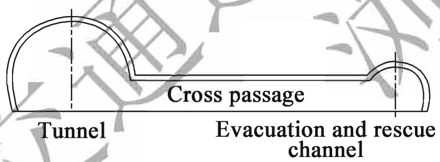
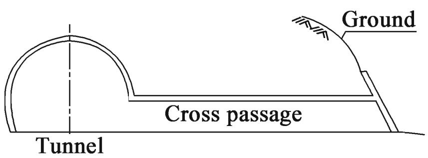
12.1 General requirements

12.1.1 There are two main purposes of providing auxiliary passages. Operation auxiliary passages are provided for operational ventilation, rescue, drainage or anti-freezing and heat preservation in highway tunnel and include vertical shaft, inclined shaft, parallel heading, cross passage, duct and spillways. Construction auxiliary passages are provided for increasing tunnel face and include vertical shaft, inclined shaft, parallel heading and cross passage. See Table 12-1 for the types, main purposes and application conditions of auxiliary passages.

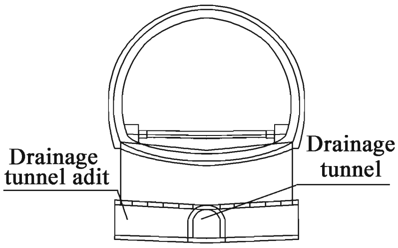
Table 12-1 Type, purpose and application condition of auxiliary passage

Type of auxiliary passage		Main purpose	Application condition
Vertical shaft		Operational ventilation	Segmented longitudinal mechanical ventilation in extra-long tunnel
		Increasingtunnel face	Long or extra-long tunnels where the geological conditions are favorable and there are no conditions for providing cross passage and inclined shaft to the ground; some parts of the arc crown where the overburden is thin.
Inclined shaft		Operational ventilation	Segmented longitudinal mechanical ventilation in extra-long tunnel
		Increasingtunnel face	Long or extra-long tunnels where the geological conditions are favorable with shallow burial depth or there is low-lying terrain alongside the tunnel or tunnel on valley line.

continue

Type of auxiliary passage		Main purpose	Application condition
Duct		Operational ventilation	Connection of vertical or inclined shaft to tunnel for ventilation; connection of fan room to tunnel;
Cross passage		Operational evacuation and rescue passage (vehicular and pedestrian cross passages)	Connection of left and right tubes of a separated tunnel
		Serving as construction links; increasing tunnel faces; facilitating muck removal & transportation and construction ventilation.	
Evacuation and rescue passage		Operational evacuation and rescue passage (vehicular and pedestrian cross passages)	Connection of single-tube long tunnel or extra-long tunnel to evacuation & rescue passage; a second-track tunnel is planned for the long term.
		Serving as construction links; increasing tunnel faces; facilitating muck removal & transportation and construction ventilation.	
		Drainage passage in tunnel	Large groundwater volume; mostly used in karstic and water-rich areas
Cross passage		Increasing tunnel face	Along mountain and river, and low-lying area on one side of tunnel; connection of tunnel to ground transversely.

continue

Type of auxiliary passage		Main purpose	Application condition
Drainage tunnel		Anti-freezing, heat preservation and drainage	High and cold region, and region with extremely large groundwater flow

12.1.2 Auxiliary passages shall be arranged in sections with good topographical and geological conditions and without severe unfavorable geologic conditions, wherever possible. In karst and abundant groundwater zones, the karst or fissure water may be discharged through auxiliary passages into tunnel, which makes the drainage more difficult, so these zones should be avoided.

12.1.3 In general, tunnel length is the basic condition for provision of auxiliary passage. For extra-long tunnel construction, duration is always the main constraint. When the progress of tunneling from tunnel faces on both ends cannot meet the duration requirement, auxiliary passages may be provided to increase tunnel faces and accelerate construction. For long or extra-long tunnels, auxiliary passages need to be provided for operational ventilation and evacuation & rescue. In addition, different types of adits may be provided for drainage. For instance, drainage tunnel may be used for drainage of karst water or underground river water. In severe cold areas, cold-proof drainage tunnel may be provided for draining the water accumulated behind lining, thus achieving the goal of eliminating cold damage. According to the roles of auxiliary passages, the operational ventilation and evacuation & rescue passages which must be provided shall be used as construction accesses wherever possible, so as to avoid waste in works and the further disturbance to mountain.

12.1.4 The cross-sections of various auxiliary ducts for operation purpose shall be determined by ventilation requirements, pipeline arrangement, disaster prevention & rescue and drainage, and other function and construction conditions.

(1) For ventilation ducts for operation, their cross-sectional areas shall be considered according to relevant requirements of highway tunnel ventilation design so that the cross-sectional areas meet the operational ventilation volume requirements. Moreover, the space required for excavation, transportation and pipelines shall be taken into account.

(2) For drainage passages, their cross-sectional areas shall be based on groundwater amount

and convenience for construction.

(3) When an operational auxiliary passage doubles as a construction air passage, its cross-sectional area shall be checked based on the air volume required for construction ventilation, and the air velocity shall be controlled within the permissible range. The velocity can be calculated as follows:

$$v = Q/F \leqslant v_{\text{Permissible}} \tag{12-1}$$

Where,

v——Velocity of air flow through adit (m/s);

Q——Air volume required (m^3);

F——Cross-sectional area of channel (m^2);

$v_{\text{Permissible}}$——The permissible highest velocity of air through adit, $v_{\text{Permissible}} = 6m/s$.

(4) After checking calculation of cross-sectional area of auxiliary channel, if the result cannot meet the construction ventilation requirements of auxiliary channel, the cross-section shall be determined according to construction ventilation requirements.

(5) The mechanical equipment used for construction of main tunnel shall be determined according to the construction needs of main tunnel if traffic capacity puts high demands on clear section.

12.1.5 The operational auxiliary channels need to serve the tunnel for a long time and are not easy to reconstruct and expand upon completion, so they shall be designed as permanent structures with specified strength, stability and durability. To meet their functional requirements, composite lining is generally required. For an auxiliary channel serving as an air channel, the composite lining will contribute to flat surface and small wind resistance; in addition, the secondary lining of composite lining will facilitate the installation of ventilation partitions. For other auxiliary channels under favorable surrounding rock conditions, shotcrete and rockbolt lining can be used, so as to save investment. Operational auxiliary channels need to be provided with waterproofing and drainages.

12.1.6 For the cross-section of an auxiliary channel only for construction purpose, consideration shall be given to the length of auxiliary channel serving the construction of main tunnel, the mechanical equipment provided, and the construction ventilation, pipeline arrangement and drainage requirements, in addition to the geological conditions.

12.1.7 Auxiliary channels for construction purpose only belong to temporary works and present no strict requirements on waterproofing and appearance of lining, so shotcrete and rockbolt lining may be used provided that it meets surrounding rock stability and lining safety requirements during construction. After completion of main tunnel, measures shall be taken for construction auxiliary channels to ensure they will have no impacts on the main tunnel and surrounding residents.

1 Upon completion of the entire works, the temporary construction auxiliary channels generally need to be backfilled to the original status. As the excavation of construction auxiliary channels leads to the formation of underground cavities and the change in stress status, surface subsidence may be caused if cave-ins occur, especially in populated regions, which might endanger the lives and properties of local residents; moreover, the safety of main tunnel might be affected. Hence, in sections with the potential for collapse, the auxiliary channels shall be strengthened or backfilled. For auxiliary channels not to be backfilled, their long term stability shall be ensured to avoid collapse; if they will not be used for a long time, they shall be inspected regularly by maintenance personnel, so they shall be made accessible to maintenance personnel.

12.1.8 The muck, waste water, exhaust and noise generated in construction of an auxiliary channel leading to the outside will have a negative effect on the environment. Therefore, the entrance/exit location of auxiliary channel (shaft), construction site layout and muck disposal shall be compatible with the general arrangement of environmental protection and road traffic. Flood control and safety protection measures shall be taken if the channel entrance/exit is located in a valley and low-lying area.

12.1.9 Due to the nature of its structural layout, an inclined (vertical) shaft passes through multiple strata and the groundwater is drained downward and collected at the bottom of shaft, which affects the structural safety and the normal use of equipment in the machine room at the shaft bottom and increases the drainage pressure in main tunnel. Therefore, waterproofing and drainage design for inclined (vertical) shaft should combine plugging with drainage. Plugging may be done by means of radial grouting to plug the groundwater and retain it within the surrounding rock and minimize the groundwater flow into the main tunnel.

12.2 Vertical shaft

12.2.1 Vertical shafts should not be arranged on the tunnel centerline. Arrangement of them on tunnel centerline will cause disturbance to construction of main tunnel and is unsafe; moreover, the joint treatment of lining structures at crown with the vertical shafts is complicated, and water leaking from vertical shafts may directly drop on the carriageway of main tunnel, threatening the traffic safety. Arrangement of them on one side of tunnel centerline can avoid the abovementioned defects. The plane spacing between the position of a vertical shaft and the tunnel is determined based on the shaft top terrain and the arrangement of connecting caverns at the shaft invert, as well as the impact of vertical shaft construction on tunnel. The horizontal distance from the plane position of a vertical shaft to the tunnel excavation boundary shall be taken into overall consideration together with the arrangement and construction influence of Tunnel Ventilation Fan (TVF) Room and other caverns.

12.2.2 A set of special facilities are required for vertical shaft construction, including hanging scaffold, clamshell, bucket and hoisting frame. For selection of shaft site, the surface terrain at the shaft shall have the space required for arrangement of hoisting equipment, unloading of materials and removal of muck. When vertical shafts are used as ventilation ducts for operation and ground fan room is adopted, the overall arrangement of shaft site shall be coordinated with fan room and ventilation shafts, which shall be not only convenient for management but also conducive to air emission, thus reducing impact on surrounding environment.

12.2.3 Circular cross-section of vertical shaft is beneficial to make use of the bearing capacity of surrounding rock and the lining structure is in good stress condition. Rectangular or square cross-section can be adopted to meet special functional requirements. For cross-section size, the clear section requirements during construction need to be considered in addition to the functional requirements. See Table 12-2 for the minimum clearances between hoisting containers and between the most prominent part of hoisting container and sidewall, shaft guide as well as shaft beam in vertical shaft cross-section during construction.

Table 12-2 List of spacing between equipment in vertical shaft (unit: cm)

Clearance category and the arrangement of shaft guide and shaft beam		Between container and sidewall	Between containers	Between container and shaft guide	Between container and shaft beam	Remarks
Arrangement of shaft guide on one side of container		15	20	4	15	Spacing between cage shoe and shaft guide clamp is 2.
Arrangement of shaft guide on either side of container	Large shaft guide	39	—	5	20	With the volume for unloading pulley; and increase of clearance between pulley and shaft guide by 2.5
	Steel shaft guide	15	—	4	15	
Arrangement of shaft guide on the front of container	Wood shaft guide	20	20	5	20	—
	Steel shaft guide	15	—	4	15	

continue

Clearance category and the arrangement of shaft guide and shaft beam	Between container and sidewall	Between containers	Between container and shaft guide	Between container and shaft beam	Remarks
Steel-rope guide	35	45	—	35	If rubber rope is provided, the minimum clearance between containers will be 20.

12.2.4 Composite lining structure can ensure the long-term stability and meet the flatness and smoothness requirements of vertical sidewall surface. Waterproofing layer is not required for vertical shaft lining due to the following reasons: ① the requirement for waterproofing is stringent for vertical shaft; ② Waterproofing is difficult to construct, and drainage will be difficult if water seepage occurs; ③ The primary support is separated from cast-in-situ concrete lining by waterproofing membrane, which is adverse to relying on surrounding rock to support the dead weight of the lining.

12.2.5 Similar to tunnel portal, capping beam at shaft surface is an important structure to prevent top of shaft from collapse and rock falls and to ensure construction safety in construction. Since the top of most shafts is located in loose and soft surface soil layers or weathered and fractured rock stratum, it is thus stipulated that "the shaft top shall be provided with concrete or reinforced concrete capping beam". The type, dimension and material of capping beam shall be determined according to such factors as the geological conditions at shaft top, the load transferred from headframe and shaft top structure to capping beam, as well as the construction method.

12.2.6 The connection of shaft invert with the connecting passage (chamfered section between shaft and passage) is the intersection of horizontal structure and vertical structure where the structural connection is special with complicated stress, and need to bear the forces transferred from shaft body and tunnel wall, so "composite or monolithic lining shall be employed".

12.2.7 In support of vertical shaft, support without shaft crib can be adopted in general geological conditions. When monolithic cast-in-situ concrete lining and composite lining are adopted in poor geological conditions, provision of shat crib can enhance the interaction between sidewall and surrounding rock and enable the dead weight of sidewall and the load transferred by sidewall to be transferred to the surrounding rock since the adhesion stress between cast-in-situ concrete lining and stratum is weak and the friction force of sidewall is low.

12.2.8 As to provision of ladder escape, ladder escape can be utilized during operation to inspect and repair the shaft structure and the equipment in the shaft; besides, it can serve as the main facility for escape and handling such accident as the seizing-up of shaft guide in case of emergency and power outage during construction.

12.3 Inclined shaft

12.3.1 Inclined shaft is one of the most commonly used auxiliary channels in tunnel works and can be classified into inclined shaft with rail haulage and inclined shaft with trackless haulage. The setting and length of an inclined shaft can be determined according to topographical and geological conditions and the functions and transportation modes. A tunnel along river and mountain, with thin overburden on side face, or with gully and low-lying area available for use on surface is favorable to setting of inclined shaft. For ridge crossing long or extra-long tunnel, when it is difficult to meet the requirements of construction duration or disaster prevention & rescue or operational ventilation, the location of the tunnel may even depends on the location of inclined shaft.

At the same tunnel depth, the length of inclined shaft with rail haulage is only about 1/3 of the length of inclined shaft with trackless haulage. The inclined shaft with rail haulage has such advantages as fast speed of shaft construction, high capacity of muck removal and good construction environment in tunnel, but its construction management is complicated and high reliability of system is required. The inclined shaft with trackless haulage has high transport and material delivery capacity, but the construction environment is poor and ventilation needs to be enhanced.

12.3.2 The cross-section of an inclined shaft shall be in a shape identical to the shape of a horizontally arranged cross passage, namely in a horseshoe shape. For surrounding rock with good self-stability, arch roof with vertical walls may be adopted. In sections with particularly poor surrounding rock and abundant groundwater, arch roof with curve-side walls may be adopted. For cross-section size, the clear section requirements during construction need to be considered in addition to the functional requirements.

12.3.3 According to survey, the dip angle of inclined shaft is mostly less than 25° (46.6%), so the dip angle of inclined shaft is small. In addition, the construction speed of inclined shaft itself is fast, it is easy for workers to go up and down safely, and the transportation efficiency is high. In the case of hoisting by rail skip, the inclined shaft has a large dip angle which is up to 35° (70%) and a short length. In the case of hoisting by miner's truck, the dip angle is greater than 25° (46.6%), which is very likely to cause falling of muck during transportation and derailment. Belt conveyor is rarely used in highway tunnel, so there is a lack of experience of its use. Considering that the muck removed from tunnel is similar to the mineral or gangue hoisted by metallurgic industry, reference to relevant domestic and overseas data are made, thus it is stipulated that the dip angle should not be greater than 15° (26.7%) in the case of hoisting by belt conveyor. The dip angle of inclined shaft is generally not greater than 7° (12.3%) in the case of trackless haulage.

12.3.4 To select the angle of intersection of inclined shaft and tunnel centerline, the structural stress and muck removal and transportation functions are mainly considered. The size of this intersection angle has great influence on the stability of surrounding rock. Small intersection angle is adverse to the stability of surrounding rock and the construction is complicated. Large angle is favorable to the stability of surrounding rock and the construction is simple.

12.3.5 Setting vertical curve at inflection point at shaft top and bottom is intended to ease the sharp change in gradient at the inflection point, so that the vehicles can pass through this point smoothly. In the case of track haulage, the gradient of longitudinal cross-section of shaft should not change. If a sag curve is adopted, there will be a bowstring shape between steel rope and rail surface, and the rope will swing substantially or even hit against the roof, thus accelerating the wear of steel rope and being easy to cause car derailment. If a crest curve is adopted, the sight condition will be poor, thus increasing the difficulty in liaison and causing unsafety.

12.3.6 Provision of refuge in inclined shaft adopting rail hoisting is intended to provide constructors going up and down with shelter space during construction. If dip angle is greater than 15°, landings shall be provided at a certain interval based on the length of inclined shaft. In the case of trackless haulage on a single-lane cross-section with long length, the provision of passing bays is to make it easy for one vehicle to give another the right of way.

12.3.7 Lining structure may be in the form of shotcrete and rockbolt lining, monolithic lining or composite lining.

12.3.8 Like the ordinary tunnel, monolithic lining or composite lining should be adopted at inclined shaft top and in zones of poor geology. The connection of shaft bottom and horizontal cross passage may bear horizontal force due to the change in structural cross-section, so monolithic or composite lining should be adopted.

12.3.9 Deformation joints shall be placed in accordance with provision of 8.3.4 herein.

12.3.11 For inclined shafts with dip angles greater than 15° (12.3%), such accidents as rope breakage, unhooking-induced car sliding (falling of guide) or overwinding accident, derailment and overturn happened during hoisting when rail haulage was adopted. Therefore, appropriate safety measures must be taken during construction of inclined shaft. In addition to stop device, such measures also include car stop at shaft top, claw hooks on vehicles, connecting bolt for preventing unhook, and devices for preventing overwinding or overspeed. In general, one stop device is provided at the shaft top and another at the shaft bottom. The shaft body may be provided with 1-2 additional stop devices based on the length of inclined shaft.

12.4 Parallel adit and cross passage

12.4.1 Parallel adits are provided to play a certain role in coping with construction ventilation, drainage and transportation, reducing construction disturbance and increasing space for main tunnel excavation. Advanced construction of parallel adits also plays a role in geological exploration. During operation, the parallel adits can serve as rescue and evacuation corridors or operational ventilating ducts. But the cost of parallel adits is high, so it may be provided only if necessary.

12.4.2 Locating the parallel adit of a single tunnel on the side of groundwater recharge source can reduce the hazard of groundwater to main tunnel. For twin tunnels, parallel adit may be located between two tunnels or on either side.

12.4.3 Like the layout of a separated tunnel, the clear distance between main tunnel and parallel adit shall be arranged in such a way that both will not influence adversely with each other. Keeping the main tunnel and parallel adit close is more beneficial to the utilization of the parallel adit and the reduction of quantities of cross passage works.

12.4.4 Making the invert elevation of parallel adit lower than the tunnel invert elevation may enable the longitudinal slope of cross passage to incline toward the parallel adit, thus facilitating drainage of water from main tunnel towards parallel adit.

12.4.6 Where there are available topographical conditions for a tunnel along river and mountain, a cross passage may be arranged to connect to the ground surface, so as to increase spaces for tunnel construction and facilitate ventilation. The longitudinal slope of the cross passage should be downward slope with slope gradient not less than 0.3% and inclines towards the outside of cross passage to facilitate drainage. If it is an upward slope, water interception, extraction or drainage measures may be taken as needed, including the interception drain or water retaining facility at channel entrance, and the pipe trench, sump or pumping facility connected to drainage of main tunnel. During operation, the cross passage may serve as a cross rescue and evacuation corridor.

12.5 Duct and Tunnel ventilation fan (TVF) room

12.5.1 To reduce air pressure loss, the intrados of duct shall be smooth. When different cross-sections are connected, variable cross-section for continuous and smooth transition is required for duct ventilation. But this will lead to inconvenience in preparation of formwork. When different cross-sections are connected, air current vortex or convolution at the sudden change position of cross-section will occur, increasing the air current loss. Therefore, transition wall needs to be provided for smooth transition of air flow. Duct partitions are used for partitioning the air currents in different directions passing through the same cavern. When concrete partitions are used for separation, its integration with lining is to ensure the duct is sealed and to prevent air series flow.

12.5.3 As the crane rail is required to be installed level, the longitudinal gradient of cavern wall top and crown should also be level. Therefore, the longitudinal gradient at bottom should be level.

12.5.5 To meet the side drainage requirements of cavern, the drainage gradient of side drain shall not be less than 0.3%.

12.6 Intersection

12.6.1 Intersection means the intersection of cross passage and main tunnel, the intersection of cross passage and tunnel ventilation fan (TVF) room and the intersection of horizontal duct and main tunnel or inclined shaft. Intersections shall avoid poor ground conditions.

12.6.2 Since intersection includes the vertical intersection of identical cross-sections, the vertical intersection of different cross-sections, and the intersection of circular cross-section and horseshoe-shaped cross-section, the common practice of using composite or monolithic lining can better ensure the structural stability at intersection. Shotcrete and rockbolt lining can be used when the two intersecting caverns have small cross-sections, the surrounding rock is stable and there are no special requirements for use. When two caverns intersect at the crown, the intersection shall be reinforced. When one cavern and the other smaller cavern intersect at the side wall, it is possible not to consider them as intersecting caverns. When the lining at intersections is of monolithic or composite type, settlement joints shall be provided based on the length of continuous lining.

13 Auxiliary Engineering Measures

13.1 General requirements

13.1.1 Auxiliary engineering measures can be divided into surrounding rock stabilization measures and inflow treatment measures. Section with poor self-stability means the section where it is difficult to maintain the stability of surrounding rock with rockbolt, shotcrete and steel support and which is susceptible to tunnel face instability, tunnel collapse and roof caving. For strata in such sections, surrounding rock stabilization measures may be taken to make the surrounding rock more stable. In surrounding rock section with inflow and mud burst and in sections with high groundwater inflow that need to be treated, inflow treatment measures may be taken to minimize the hazard of groundwater to tunnel construction and operation or the loss of groundwater. Surrounding rock stabilization measures can be subdivided into surrounding rock advance strengthening measures and surrounding rock support measures.

13.1.2 One or multiple auxiliary engineering measures shall be taken depending on surrounding rock conditions, tunnel structure design, construction conditions, schedule requirements, construction machinery, duration and economical efficiency.

13.2 Surrounding rock stabilization measures

13.2.1 Pipe roofing (also called advance large pipe roofing) is a steel pipe roof composed of a series of steel pipes (Tremie) which are arranged beyond the tunnel excavation profile in tunnel axis direction before tunnel excavation. With the excavation of tunnel, the steel pipe roof can be connected to the steel rib support constructed in time to form a vertical and transverse support system (Fig. 13-1). Pipe roofing is constructed before tunnel excavation to vertically support the rock above the arch ahead of tunnel face. During tunnel excavation, pipe roofing and steel rib support are combined to significantly prevent subsidence of surrounding rock and collapse of tunnel face and maintain the stability of tunnel face. The advance large pipe roofing has very strong support capacity and settlement control capacity and can be adopted in loose rock strata, fractured formation, in shallow sections with strict control requirements of ground subsidence, and in collapse susceptible sections. Due to limited technological conditions, the advance large pipe roofing is used mostly at portals and rarely used inside tunnels as there are other alternative forepoling measures.

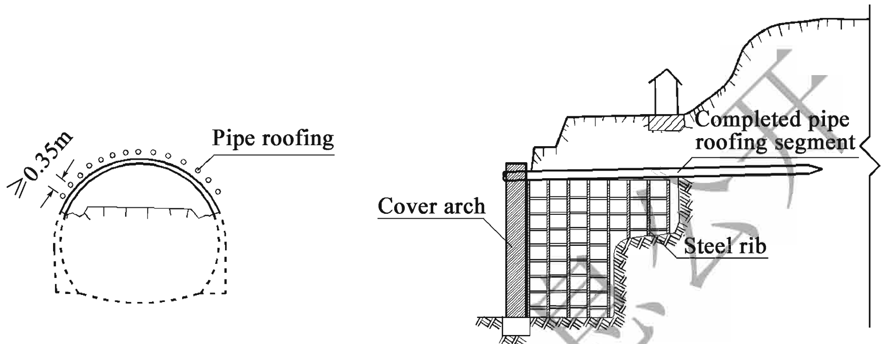


Fig. 13-1 Forepoling by pipe roofing

1 The layout of steel pipes are similar to the shapes of tunnel excavation profiles, with a distance of 100 ~ 200mm from the steel pipe center to the excavation profile, as shown in Fig. 13-2. The pipe roofing shall have an external dip angle of 0.5° ~ 2° at the arch to prevent longitudinal steel pipes from encroaching into the tunnel excavation profile.

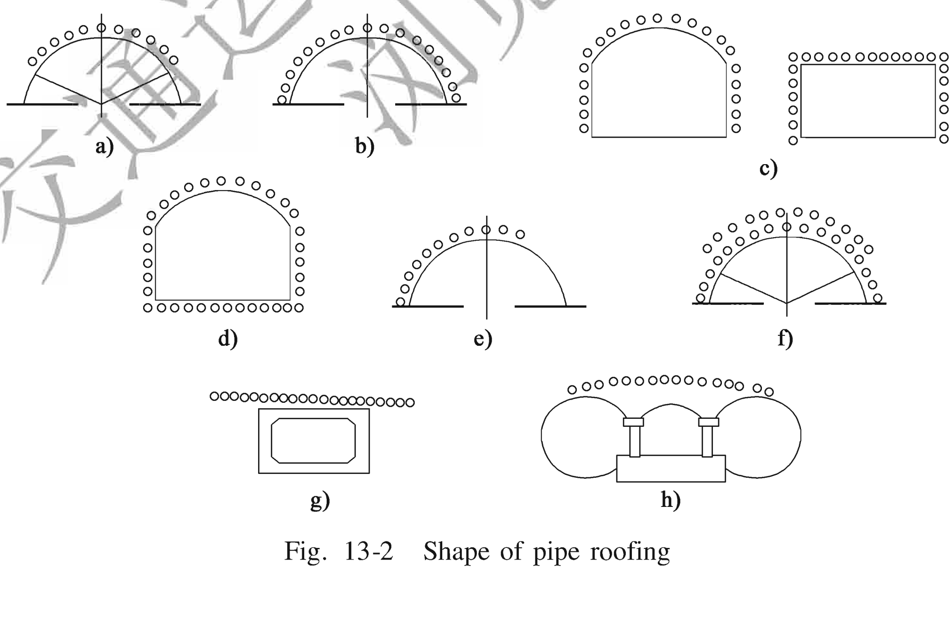


Fig. 13-2 Shape of pipe roofing

2 The circumferential spacing of steel pipes depends on the geological conditions in the pipe roofing support section and is generally within a range of 350 ~ 500mm to ensure no muck falling between two pipes. Small spacing shall be taken in surrounding rock of water-

logged sandy soil formation, loose gravel layer, backfill formation and formation where the grain size of broken surrounding rock is small.

3 The steel pipe for one continuous support is generally 10 ~ 45m long, including about 8 ~ 40m of support length. The length of forepoling shall be determined as needed. When the length of forepoling required is greater than 40m, other forepoling measures are generally taken for extension, or the forepoling may be constructed twice. A horizontal overlap joint not less than 3.0m long shall be provided between two pipe roofing supports, and between pipe roofing and other forepolings to ensure the far end of steel pipe can be effectively supported. If the pipe roofing is less than 10m long, it is uneconomical to use pipe roofing as the forepoling. If the length of a section that needs to be provided with forepoling is within a range of 10 ~ 40m, the steel pipes shall be driven into good ground for not less than 3.0m so that there is still enough length of forepoling at the far end of pipe roofing after excavation.

4 The length of steel pipe segment shall be determined according to pipe roofing technology. Each joint of steel pipe shall be staggered by not less than 500mm with adjacent steel pipe joints, and the steel pipe joint shall not exceed 50% in the same cross-section.

5 To ensure the continuity and strength of steel pipe, reinforcing steel cage or rebar bundle shall be threaded through the steel pipes before injection of mortar (Fig. 13-3). Bad connection between steel pipe segments is likely to cause the breakage of pipes and the failure of pipe roofing. Insertion of reinforcing steel cage or rebar bundle into steel pipes can ensure the continuity of steel pipes as a whole. The pipes are filled fully with mortar of certain strength to maintain their strength and stiffness, so that they will not bend or dent under stress and consequently ensure the support capacity of the entire pipe roofing system. Grouting is generally designed as limited grouting. The water-cement ratio in the grout is within a range of 1:0.5 ~ 1:1.0. The quantities of single-pipe grouting works may be estimated as per 13.2.2 below. The initial grouting pressure is 0.5MPa ~ 1.0MPa.

6 Small grouting holes may be drilled in the steel pipe wall (as shown in Fig. 13-3) so that some grout can permeate into the surrounding rock to strengthen the surrounding rock and improve its self-stability. The tail end of steel pipe is a grout-stop segment with no drilled holes. The grout-stop segment is driven into the rock for 1.0 ~ 2.0m. Both the grout- stop segment and the exposed segment are not drilled.

Steel pipe grouting is mainly intended to increase the stiffness of steel pipes and their support capacity in longitudinal direction of tunnel by filling the pipes full of mortar. The original specifications set out that grouting holes, each with a diameter of 10 ~ 16mm, should be drilled in the steel pipe wall at an interval of 150 ~ 200mm. The interval was smaller. To reduce the damage to steel pipes caused by drilling, the interval is adjusted in this revision.

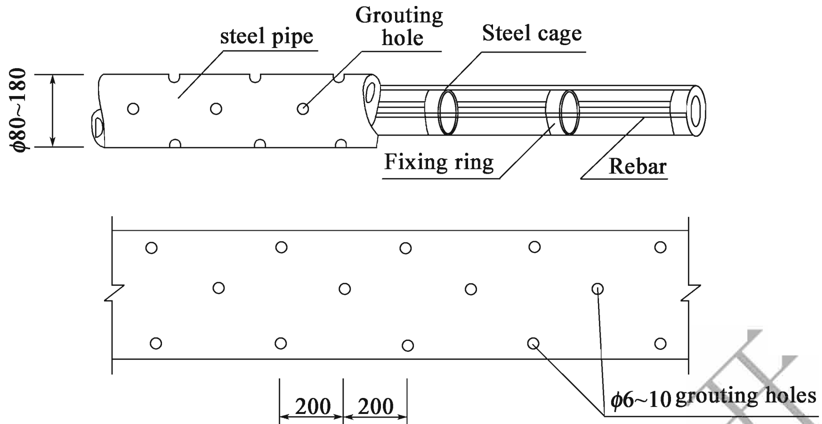


Fig. 13-3 Structure of steel pipes for pipe roofing (unit: mm)

7 The end of pipe roof shall be supported to play the role of pipe roofing in forepoling. Cover arch is provided to effectively support the end (initial segment) of roof steel pipe. Cover arch shall be rigid support structure. Steel guide tube is embedded in the cover arch to ensure accurate positioning of steel pipes and drilling orientation.

13.2.2 Advanced tremies are the small grout pipes driven and densely arranged with an external dip angle of 5° ~ 12° forward longitudinally along the excavation profile at the tunnel arch (as shown in Fig. 13-4). The exposed end of the tremies needs to be supported by the steel rib support close to the tunnel face. The tremies and steel rib supports constitute a longitudinal and transverse support system. Mortar is injected via the tremies and permeates into the surrounding rock ahead to consolidate and support the surrounding rock within a certain range. Advanced tremies also have the function of pipe roofing and stronger support capacity than advance rockbolts. Compared to pipe roofing, they are simpler and easier to construct and more flexible and economical, but their support capacity is weaker. The overlap length for each round of tremies may be increased by reducing the longitudinal round spacing of tremies, so that there are two layers of functioning tremies.

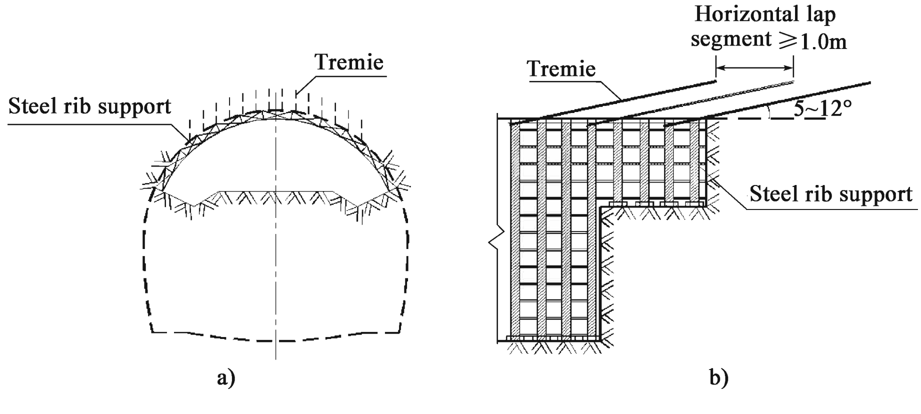


Fig. 13-4 Forepoling by Tremie

2 Grouting holes are drilled through the tremies and arranged in quincunx form. The front end of tremies is in cone shape. A grout-stop segment not less than 500mm in length shall be reserved at the tail (as indicated in Fig. 13-5).

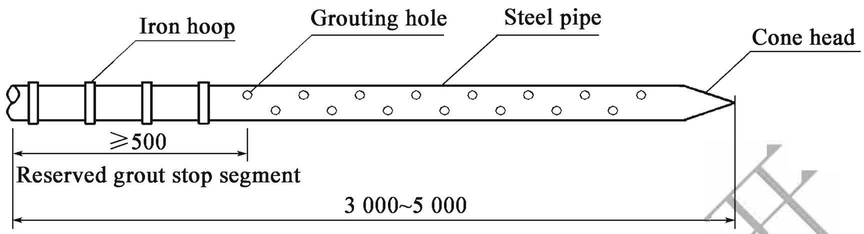


Fig. 13-5 Tremie structure (unit: m)

5 Water-cement ratio for cement mortar to be injected is generally within a range of 1:0.5 ~ 1:1.0. When the surrounding rock is broken and its grout stop effect is not good, cement - sodium silicate grout may be adopted, with the grout setting time controlled within a few minutes and grouting pressure kept within a range of 0.5 ~ 1.0MPa.

The grout diffusion radius (R) may be determined based on the density of tremie arrangement and may be calculated as per Eq. (13-1) considering the overlap within the grout diffusion range:

$$R = (0.6 \sim 0.7) L \tag{13-1}$$

Where,

L——Spacing between centers of Tremie (m).

The weight of a single tremie for grouting (Q) may be calculated as per Eq. (13-2):

$$Q = \pi R^2 l n \tag{13-2}$$

Where,

R——Grout diffusion radius (m);

l——Length of tremie (m);

n——Void content in surrounding rock (%).

In the case of broken rock mass, the rock mass between thetremies may collapse, so two layers of tremies may be arranged in an alternate way. The external insertion angle of the inner layer in tunnel shall be 5° ~ 12° and the external insertion angle of the outer layer shall be 10° ~ 30°. When two layers of Tremie are used at the portal, the spacing between two layers should not be greater than 300mm.

13.2.3 The arrangement and function principles of advance rockbolts are identical to those of advanced tremies. But the support capacity of the former is weaker than that of the latter. Self-propelled advance rockbolts may be used when the loose, broken surrounding rock is unfavorable for drilling.

13.2.4 For consolidation by advance drilling and grouting, the grout materials with filling and cementitious performance are injected by the matched grouting machine and equipment into the stratum that needs to be consolidated. Then, through hardening of the grout, they fill and plug the gaps in stratum to improve the density of surrounding rock in grouting area or reduce the water permeability and the leakage of water during tunnel excavation, as well as to solidify the weak and loose rock mass, thus improving the strength and self-stability of surrounding rock. Advance drilling and grouting is a method to consolidate the stratum.

In advance drilling and grouting, the grouting holes may be arranged in an umbrella-shaped, radial form from the tunnel face towards the excavation direction. They may be arranged in layers on the front of tunnel face. Depending on the tunnel excavation methods, full-face one-time hole arrangement or half-face multiple-times hole arrangement may be adopted. Drill holes are arranged in one circle or several circles, with long holes and short holes combined, as shown in Fig. 13-6.

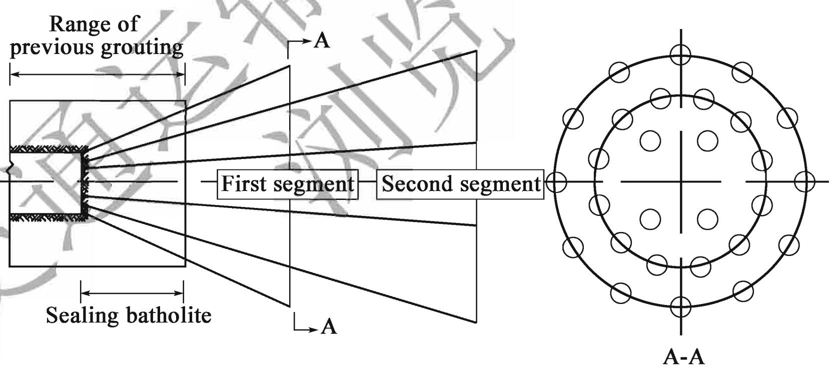


Fig. 13-6 Arrangement of drill holes for advance drilling and grouting

1 The consolidation extent may be the entire excavation and its periphery or perhaps one side, the arch or other local areas, as shown in Fig. 13-7.

2 The arrangement of grouting holes depends on the spacing between hole inverts, which may be 1.4 ~ 1.7 times the grout diffusion radius. The grout diffusion radius may be controlled within a range of 1.0 ~ 2.0m.

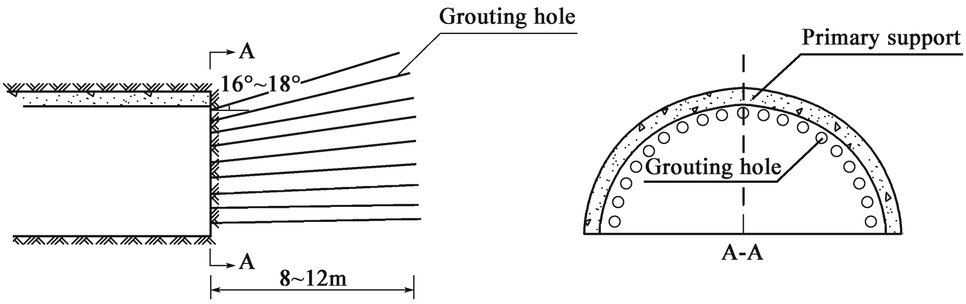


Fig. 13-7 Pre-grouting of periphery holes

13.2.5 The spacing between holes of advance horizontal jet grouting piles shall be determined according to the specific geological conditions. Table 13-1 presents the diameters of advance horizontal jet grouting piles for reference. Effective diameters of directional jet grouting piles and swing jet grouting piles shall be 1.0 ~ 1.6 times the diameter of jet grouting piles.

Table 13-1 Design diameter of jet grouting piles (m)

Soil property		Single tube method	Double tube method	Triple tube method
Cohesive soil	0 < N < 5	0.5 ~ 0.8	0.8 ~ 1.2	1.2 ~ 1.8
	6 < N < 10	0.4 ~ 0.7	0.7 ~ 1.1	1.0 ~ 1.6
	11 < N < 20	0.3 ~ 0.6	0.6 ~ 0.9	0.7 ~ 1.2
Sandy soil	0 < N < 10	0.6 ~ 1.0	1.0 ~ 1.4	1.5 ~ 2.0
	11 < N < 20	0.5 ~ 0.9	0.9 ~ 1.3	1.2 ~ 1.8
	21 < N < 30	0.4 ~ 0.8	0.8 ~ 1.2	0.9 ~ 1.5

Note: N means the number of blows in standard penetration test.

Cement grout is generally used as the grouting material for horizontal jet grouting piles. Quick-setting early-strength cement grout should be used in areas with abundant groundwater. Cement shall be of Grade 32.5 or 42.5 Portland cement, with water-cement ratio of 1:1 ~ 1.5:1 in grout.

The quantity of grouting can be calculated by volumetric method or jet measurement method. By volumetric method, the quantity is calculated as per Eq. (13-3).

$$Q = \frac{\pi}{4} D_e^2 k_1 h_1 (1 + \beta) + \frac{\pi}{4} D_0^2 k_2 h_2 \tag{13-3}$$

Where,

Q——Required grout quantity (m^3);

D_e——Diameter of jet grouting pipe (m);

D_0——Diameter of injection pipe (m);

k_1——Filling rate, taken at 0.75 ~ 0.9;

h_1——Jet grouting length (m);

k_2——Soil filling rate in the scope without jet grouting, taken at 0.5 ~ 0.75;

h_2——Length without jet grouting (m);

β——Loss coefficient, taken as 0.1 ~ 0.2.

By jet measurement method, the grout quantity is calculated based on the quantity of grout injected per unit time and the injection duration, as shown in Eq. (13-4).

$$Q = \frac{H}{v} q (1 + \beta) \tag{13-4}$$

Where,

Q——Quantity of grout used (m^3);

v——Lifting speed (m/min);

H——Jet length (m);

q——Quantity of grout injected per unit time (m^3/min);

β——Loss coefficient (0.1 ~ 0.2).

Horizontal jet grouting piles are generally used in saturated soft soil areas. Advanced horizontal jet grouting piles should not be used in soft soil stratum where the groundwater flow velocity is high and grout cannot solidify around the injection pipe.

13.2.6 Glass fiber rockbolts are mainly used to consolidate the soil mass ahead of excavation. Advance consolidation with glass fiber rockbolts is a supporting technology of the ADECO-RS approach. The ADECO-RS approach is a systematic tunnel design and construction technique for full-face mechanical excavation of tunnels. Glass fiber rockbolts are used for advanced consolidation of tunnel face so that the tunnel face can achieve certain self-stability. Full-face excavation is made efficiently by mechanical system. The ADECO-RS approach for the design and construction of tunnels has been included in codes of the Italian highway and railway fields and widely used. It has also been adopted in the construction of most large-scale projects in European countries.

The glass fiber rockbolts are grouted and anchored in full length to consolidate the tunnel face and surrounding rock. The glass fiber rockbolts have high strength and light weight. Their tensile strength may be 1.5 times the steel rockbolts and their weight is 1/4 ~ 1/5 of the steel rockbolt of the same specification. They are safe and resistant to static electricity, flame, corrosion, acid and alkali, and low temperature. Due to their low shear strength, they can be removed directly by construction machinery.

Advance glass fiber rockbolts are used for advanced consolidation of core ahead of tunnel face, as shown in Figs. 13-8 and 13-9.

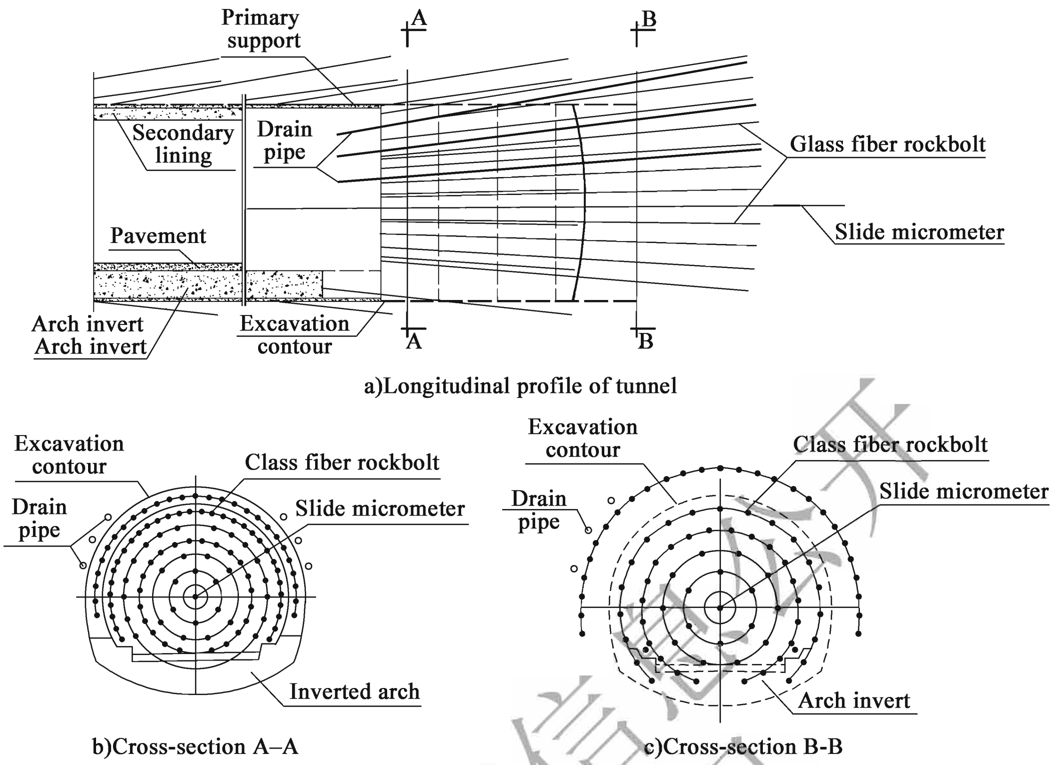


Fig. 13-8 Schematic diagram of consolidation by advance glass fiber rockbolt

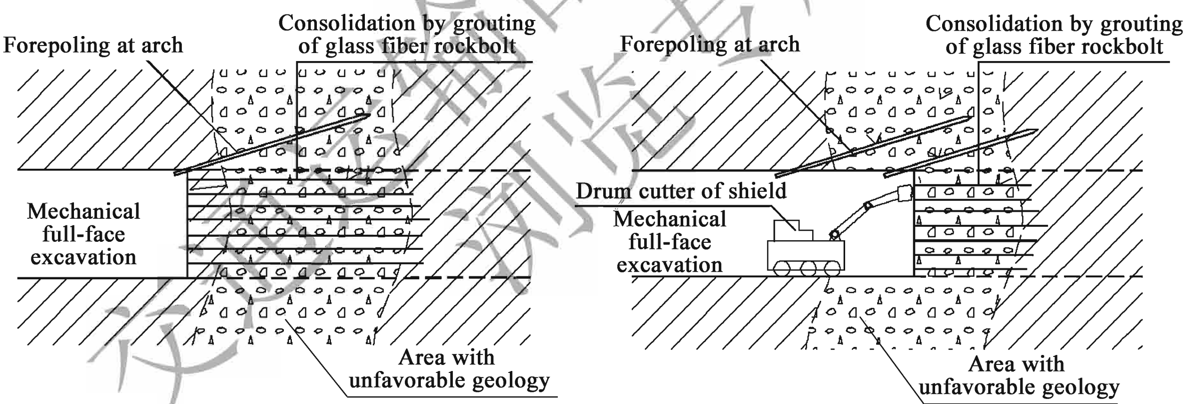


Fig. 13-9 Schematic diagram of consolidation by and excavation for advance glass fiber rockbolt

13.2.7 Surface mortar rockbolts are used for stratum consolidation on the surface for tunnels with a depth generally not more than 25m (as shown in Fig. 13-10). To ensure the consolidation effect, excavation of tunnel below cannot be made until the anchoring mortar has exceeded 70% of strength.

Rockbolts arranged transversely shall not encroach on tunnel excavation profile and shall be kept 0.5m away from the tunnel excavation line, as shown in Fig. 13-11.

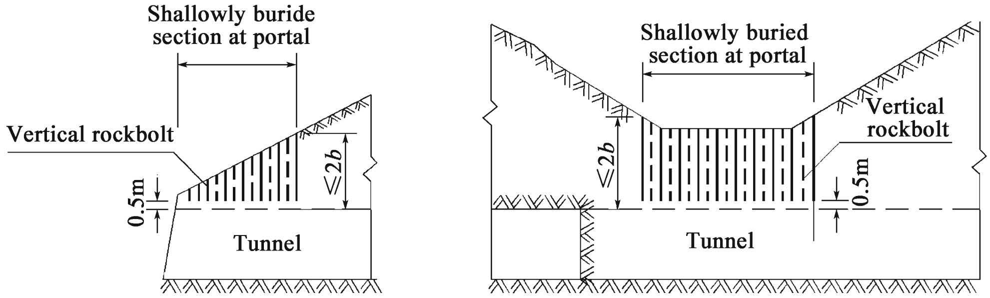


Fig. 13-10　Longitudinal arrangement of surface mortar rockbolt

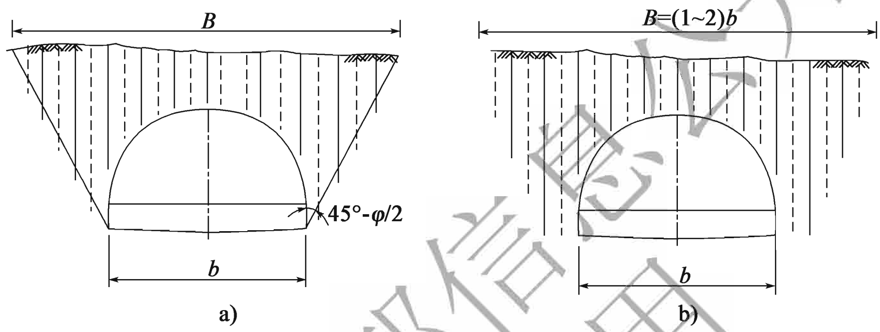


Fig. 13-11　Transverse arrangement of surface mortar rockbolt

The width of consolidation by surface rockbolts are generally considered as 1 ~ 2 times the tunnel width or determined according to the following methods:

(1)　Fracture plane estimation method

For calculation of consolidation width by fracture plane method, it is assumed that the rock mass outside either side wall slides along the facture plane at an angle of 45　-ϕ/2 from the vertical plane after excavation in soft and weak surrounding rock. By extending the fracture planes upward until they intersect the ground surface, the distance between the two intersections is regarded as the consolidation width B (Fig. 13-12) and its half width $B/2$ is as follows:

$$B/2 = \frac{b}{2} + (h + H)\,\mathrm{tg}(45° - \frac{\varphi}{2}) \tag{13-5}$$

Where,

$b/2$——Half of the tunnel excavation width (m);

h——Depth of tunnel (m);

H——Tunnel excavation height (m);

φ——Internal friction angle in rock mass (°).

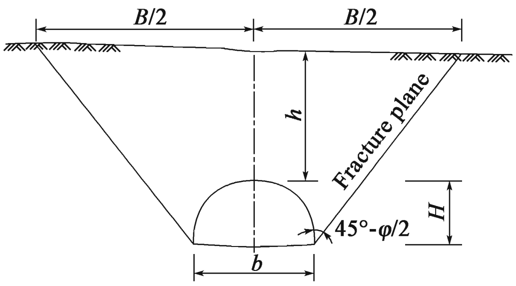


Fig. 13-12　Consolidation width B

(2)　Determination of longitudinal consolidation length based on burial depth

Longitudinal consolidation length is generally the length of the shallowly-buried section or the length when the depth $h \leqslant 2b$ (b means the tunnel excavation width).

13.2.8　Consolidation by surface grouting means the advance consolidation of surrounding rock by injecting grout into holes drilled downward from ground surface. Compared with surface consolidation by surface mortar rockbolts, all requirements are the same except for the arrangement of grouting holes (Fig. 13-13).

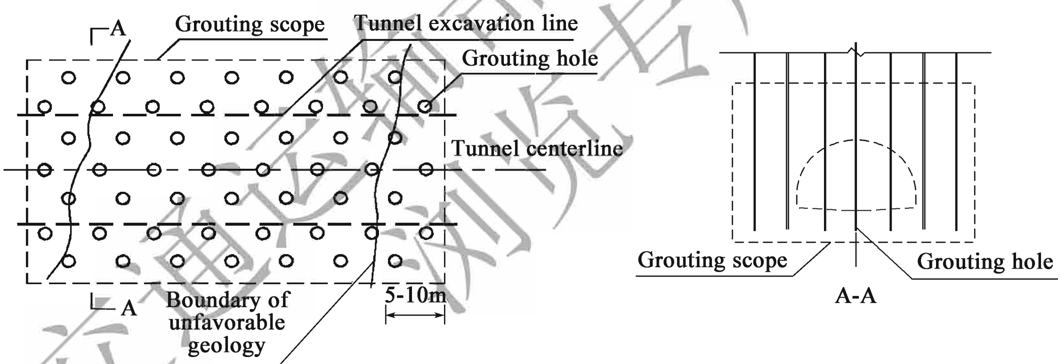


Fig. 13-13　Arrangement of grouting hole

13.2.9　Rockbolts at toe of walls (pipes) are used at the arch foot or elephant foots from the steel arch shoulders to the toes of side walls and at joints of steel rib support, i.e. at the toes of side walls in full-face excavation or at the arch foot or elephant foots of the upper half cross-section (heading) and the toes of side walls of the bench excavation by sequential excavation method, with a main purpose of controlling the settlement deformation of primary support. Rockbolts at toe of walls shall be used in the case of good geological conditions. Locked anchor pipes shall be used in the case of poor geological conditions.

13.2.10　The radial lengths of and the interval between tremies shall be determined according to the size of clear section and the surrounding rock consolidation scope.

13.2.11 Temporary closure and support include temporary closure of tunnel face, temporary arch invert of primary support, temporary member support, fan-shaped support at arch, #-shaped truss support, and wooden crib support. Temporary closure and support are generally used in tunnels with very poor geological conditions and large cross-sections and where change in working procedures is necessary. They are also used to control the instability of tunnel face, cracking of support structures and continual development of deformation and to treat the collapsed tunnel.

(1) Tunnel face may be supported by rockbolt and shotcrete or closed by sand bags in the case of extrusion deformation and mud outburst on tunnel face, or for collapse.

(2) Temporary arch invert of structural steels or diagonal bracing of structural steel or square timber may be provided in the case of large deformation of surrounding rock or for excavation of collapse body.

(3) Arched steel supports or fan-shaped structural steel rib support may be used for support in sections where the primary support cracks severely and the arch wall lining needs to be removed and replaced.

(4) #-shaped truss support or wooden crib support may be used when the arch settlement is obvious or the requirements for surface subsidence are strict.

(5) Cast-in-situ concrete retaining wall or sandbags may be used for closure if high-grouting is required ahead of tunnel face.

(6) Collapse may be locked with arched steel rib support, fan-shaped support at arch, #-shaped truss support and wooden crib support.

13.3 Inflow treatment measures

13.3.1 According to a lot of practice in recent years, tunnel inflow plugging is technically difficult to achieve the ideal results with the current construction technologies. Inflow treatment solely by drainage will cause damage to groundwater environment at tunnel site. Therefore, the preliminary site investigation and route selection, design and construction shall follow the principle of "plugging as the primary aim, combining plugging with drainage and being environmentally friendly". One or more ways shall be selected based on such factors as geological and topographical conditions on site, environmental protection requirements, construction technology and construction costs, in order to produce safe and environment-friendly control effects during construction and operation.

13.3.2 Pre-grouting into surrounding rock for water plugging is used for surrounding rock in the section not excavated ahead of tunnel face. The grouting ring thickness is determined through overall consideration of such factors as the inflow volume, surrounding rock geological conditions and groundwater pressure. The length of one grouting operation is determined based on such factors as the geological conditions of surrounding rock ahead of tunnel face, groundwater pressure, grout-stopping wall thickness and construction machinery level. The spacing of grouting hole inverts from center to center is determined based on overlap of grout diffusion extent of holes. Grouting quantity and grout diffusion radius are very hard to determine accurately. In general, they are selected preliminarily through engineering analogy and determined through experimental verification on site.

13.3.3 Radial grouting into surrounding rock is a way to plug water by grouting into radially drilled holes along the tunnel excavation profile after tunnel excavation and completion of primary support. After tunnel excavation, the amount, location and form of water seepage from surrounding rock are exposed, so the target of water plugging by grouting is clear. In addition, the process is simple and may produce good results at a lower cost than pre-grouting into surrounding rock. Full-face radial grouting, local radial grouting or supplementary grouting may be selected depending on site conditions.

13.3.4 Advance borehole drilled for drainage is a measure taken to prevent outburst of confined water and also an effective method of advanced water exploration. The groundwater flow direction and inflow, as well as the locations, directions, number, depth per drill and diameters of boreholes may be determined through detailed site investigation and analysis of engineering geology and hydrogeology. During excavation and advance of the tunnel face, the advance borehole inverts are kept 1-2 round lengths ahead of the tunnel face, in order to ensure the thickness of rock mass, which is not drilled and drained, in surrounding rock section ahead of the tunnel face is enough .

13.3.5 Drainage tunnel is generally used for drainage where high-pressure groundwater or inflow with sufficient recharge source or seasonal inflow exists ahead of the tunnel face, poses a serious threat to construction and operation and, if drained, does not affect the surrounding rock stability and has little effect on the water environment around the tunnel.

13.3.6 Well point dewatering is a dewatering measure generally taken during construction to reduce and eliminate the impact of high groundwater level on construction. It is a method to lower the groundwater by installing a certain number of filter pipes (wells) from the ground surface along two sides of the tunnel and drawing water with pumping equipment. Well point for dewatering includes such types as light well point, ejector well point, electro-osmosis well point, tube well point and deep well point. Well point type, dewatering method, equipment and well point arrangement shall be determined according to such factors as permeability of stratum, dewatering scope and depth.

14 Design of Tunnels in Special Geology

14.1 General requirements

14.1.1 Particular design and special construction methods are required for construction of tunnels in areas with special geology because the geologic origin of stratum is complicated in these areas and may bring great harm to tunnel construction which is difficult to cope with by conventional method only.

14.1.2 The geological data available and the countermeasures developed before construction in areas with special geology may not fully tally with the actual conditions on site. Therefore, during construction, it is necessary to often observe the change in strata and change in stress and structural deformation, to understand the stress status of surrounding rock support and lining based on the information obtained through field monitoring and measurement, to timely adjust engineering measures and parameters, and to find and eliminate dangerous situation, thus preventing accidents.

14.2 Swelling surrounding rock

14.2.1 Tunnel lining in swelling surrounding rock may bear great swelling pressure from all directions. To better adapt to this stress condition, the cross-section of tunnel shall be in circular shape or in oval (nearly circular) shape.

14.2.2 For tunnel support in swelling surrounding rock, firstly, flexible support shall be installed earlier to form a ring in time, so that the surrounding rock will deform under control. Secondly, support shall be applied in layers to gradually increase its stiffness, so as to increase its control over surrounding rock deformation that has occurred. Lastly, secondary lining shall be constructed at the right moment. If secondary lining is constructed too early, it will bear great swelling pressure of surrounding rock and may be damaged by the pressure. If it is constructed too late, the deformation will exceed the deformation allowance and encroach into its space.

14.2.3 If large deformation is caused by the swelling pressure, the shotcrete layer will spall, fall and break or the steel rib support will distort. To adapt to the large deformation, shotcrete and rockbolt lining should be constructed in two layers to gradually increase its support stiffness; alternatively, retractable steel rib support, long rockbolts, or long and short rockbolts may be adopted to resist and control large deformation.

Retractable steel rib support is made of U-structural steel. Each steel rib support may be provided with 5 ~ 7 retractable joints. Each joint may retract for some 50 ~ 100mm.

14.2.4 Swelling surrounding rock is characterized by large deformation, so its deformation allowance shall be properly increased compared to the ordinary surrounding rock, thus reducing surrounding rock swelling pressure on structures.

14.2.5 In swelling surrounding rock, shotcrete-and-rockbolt support is advantageous because it can be constructed in time and in tight contact with surrounding rock and deform jointly with surrounding rock, thus effectively controlling the surrounding rock deformation. The secondary lining mainly bears the subsequent increasing swelling pressure of surrounding rock. To ensure the secondary lining has enough bearing capacity, reinforced concrete structure shall be used. Timely construction of arch invert can bring the overall bearing capacity of lining into play in advance.

14.2.6 In swelling surrounding rock, water has a great effect on its strength and volume change. Therefore, surface water interception and drainage works shall be carried out to reduce the surface water seepage into tunnel, and water accumulated in tunnel shall be drained in time.

14.3 Karst

14.3.1 The rock solubility and facture as well as the water erosion and flow condition are the determining factors as to the development of karst. Soluble rock may be classified into three categories, i. e. carbonate rock (limestone, dolomite and marlstone), sulfate rock (gypsum, mirabilite) and halide salt rock (rock salt).

The main influence of karst on tunnel works comes from caves, groundwater and cave infill, which may lead to collapse of tunnel and surface sinkholes above tunnel crown. In general, depending on the location of karst with respect to the tunnel and the constructions, such measures as crossing, cave consolidation, karst water drainage, interception and diversion, infill removal or infill consolidation with grouting, surface collapse backfilling and closure, as well as surface water drainage or a combination of such measures may be designed, as shown in Fig. 14-1.

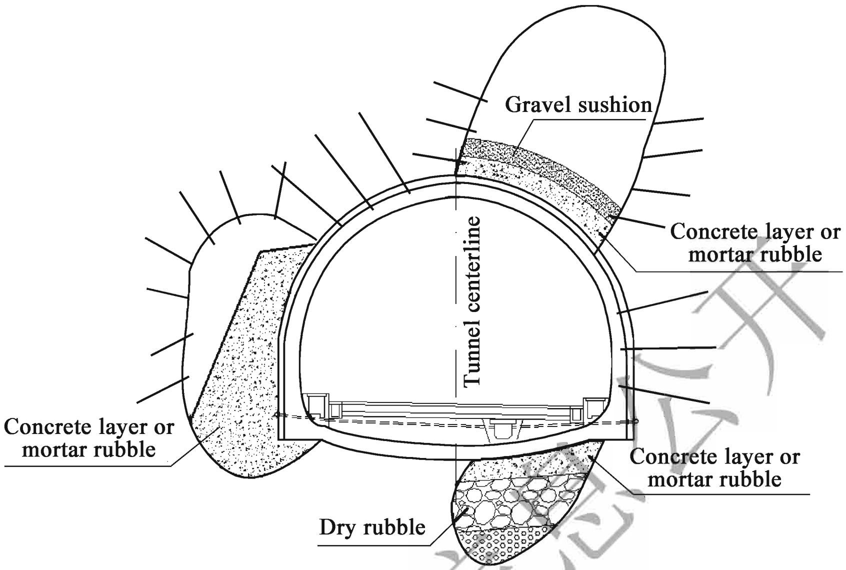


Fig. 14-1 Schematic diagram of composite backfill structure

14.3.2 As to the method of crossing, the girder bridge and arch bridge are usually used for crossing; alternatively, a roundabout way may also be adopted for avoiding karst caves.

14.3.3 When the karst cavity above tunnel is large, the tunnel is actually of cut-and-cover structure to pass under it and treated in accordance with the requirements of backfilling above the tunnel arch crown and on both sides of tunnel. Crown backfilling (or provision elephant foot) is intended to avoid falling of objects from karst cave, which may bring impact load hazard to crown. Both sides of tunnel are backfilled with concrete or stone pitching to provide tunnel side wall with enough resistance, as shown in Fig. 14-1.

14.3.4 Tunnel invert may be replaced and backfilled with concrete, stone pitching or dry rubble.

14.3.5 According to the lessons learned from tunnel karst control in recent years, blocking karst water drainage channel can cause damage easily to tunnel support structure and threat to tunnel operation safety. Therefore, the original groundwater runoff conditions shall be maintained and a buried culvert shall be constructed separately for drainage, thus solving the karst cave channel problem. The karst channel at the invert of tunnel may be blocked during construction, so its protection and clearing needs to be strengthened. The exposed karst channel at crown arch needs to be restored.

14.4 Goaf

14.4.1 When a tunnel passes through a goaf, the tunnel structure and goaf control measures shall be determined based on the extent of mining and mining methods of the golf, its distribution, size and purpose, as well as the volume and content of hazardous gas in it.

14.4.2 The effects of a goaf on tunnel works are mainly manifested by damages due to cavity, water and gas. A backfilled and collapsed goaf can be strengthened by grouting. A goaf in use may have its structure strengthened. An exposed goaf may be treated in the same way as karst cave or the structures may be co-constructed. Groundwater in goaf needs to be drained. Toxic gases in goaf need to be discharged and confined.

14.4.3 The tunnel within the zone of influence from goaf should be supported by strong structures which should also be capable of confining gas to prevent it from overflowing. If an unmined zone has an influence on tunnel, the tunnel structure should also be capable of confining gas.

14.5 Running sand

14.5.1 Sandy soil or silty clay losing its cohesion under the action of water forms the running sand, which is mostly in paste form and causes great harm to tunnel construction. Wherever there is running sand, the surrounding rock will lose its stability and collapse and the support structure will deform or even be damaged. Therefore, suitable engineering measures shall be taken.

14.5.2

1 Groundwater is the main cause of running sand hazard, so lowering the groundwater level is favorable to tunnel safety. Measures of groundwater level lowering include well point dewatering and advance borehole for drainage.

2 Overflow of running sand will affect construction and lead to cavitation of tunnel surrounding rock, thus affecting the stability of surrounding rock. Plugging measures shall be taken to prevent running sand from overflowing. The common plugging measures include sandbags or fast-setting cement.

3 Surrounding rock near the running sand may be strengthened by such measures as shotcreting, sandbags, concrete wall, mortar or dry rubbles to maintain rock mass

stability and to prevent expansion of the running sand area.

4 Advance consolidation measures include forepoling, advanced horizontal jet grouting pile, surface grouting, surface mortar rockbolt and dense rows of advance tremies, and if necessary, advance steel pipe curtain may be used.

5 In order to improve the bearing capacity of structural support, steel rib support used for primary support shall be closed as soon as possible and temporary arch invert shall be provided. In order to expand the support contact area, floor beam are added to the support footing.

6 For wooden supports or steel truss supports, their bases have large contact area, thus having greater settlement control capacity.

14.6 Gas and other toxic gases

14.6.1 Among the tunnel gas control measures, extraction means gas discharge and emission in advance by advance boreholes. Isolation and sealing mean isolation of gas from support structure by gas-proof board (similar to tunnel waterproofing membrane) and impermeable secondary cast-in-situ concrete lining. Reinforcement means use of traditional surrounding rock grouting method to fill the gaps and plug the gas leak passage, thus reducing the permeability of surrounding rock and providing stability strength. Tunnel gas prevention and control design includes not only the structural design but also advance observation of gas, coal uncovering, excavation, support structure, gas prevention and emission, ventilation, monitoring and detection. Therefore, reinforcement measures and advance observation, ventilation as well as monitoring need to be combined for gas prevention and control design.

14.6.2 Gas in rock strata mainly permeates the tunnel through the small cracks in concrete lining and the construction joints. To reduce the amount of gas permeating tunnel during construction and operation, the lining shall have a composite closure structure with arch invert, with increased anti-permeation performance of secondary cast-in-situ concrete lining.

14.6.3 General anti-permeation measures for construction joints in tunnel include hydrophilic strip, grout pipe, buried waterstop strips and back-bonded waterstop strips. Where a two-pass lining system of cast-in-situ concrete is employed, construction joints in the two layers of lining shall be staggered to make the construction joints more impermeable.

14.6.4 The minimum thicknesses of shotcrete and secondary cast-in-situ concrete are specified to ensure the isolation effects of lining.

14.6.5 When the reserved caverns are excavated, the thicknesses of lining structures behind and around the caverns shall comply with 14.6.5 above of the Specifications. Embedded parts shall not penetrate the structure.

14.6.6 Other toxic gases mean hydrogen sulfide, petroleum asphalt gas, etc.

14.7 Loess

14.7.1 Loess is a quaternary accumulation of continental sediments. It is a special soil with needle-like pores and vertical joints formed under semi-arid climatic conditions. According to the age of formation, it can be classified into old loess and new loess. The old loess includes the Wucheng loess and the Lishi loess. The new loess includes the Malan loess and the recently deposited loess. There is a big difference between new and old loess in terms of physical and mechanical properties and surrounding rock stability, so the lining structure shall be determined based on its soil classification and physical & mechanical properties. The different construction methods will directly affect the stability, loading and loading distribution of loess, so the tunnel lining structure shall be determined through considering the combination of multiple unfavorable loads to enable the lining to adapt to various load conditions that may occur during construction and operation.

14.7.2 According to a large number of field test studies and measurement data, the vertical pressure borne by the lining of a tunnel in loess is uneven, the side pressure is large, and the side pressure coefficient can reach 0.5-0.8. Composite (or two pass cast-in-situ) lining with arch invert or curved wall is conducive to the stability of surrounding rock.

Currently, there are two types of lining structures for tunnels in loess, i.e. two pass cast-in-situ lining structure and composite lining structure. If loess surrounding rock is exposed for a long time after excavation, there will be groundwater seepage, which will accelerate the relaxation of the surrounding rock mass and cause collapse. The exposure time of surrounding rock for the two pass cast-in-situ lining is longer than that for the composite lining. For composite lining, the surrounding rock is timely closed by shotcreting after excavation, and the shotcrete, steel rib support and reinforcement mesh quickly form a closed support structure. The closing time is short and the surrounding rock deformation can be controlled effectively. Therefore, a composite lining structure is generally used. Two pass cast-in-situ lining may be used as the secondary lining in the composite lining.

In the loess strata, rockbolts improve the modulus of elasticity, cohesion and internal friction angle of the surrounding rock in the anchorage zone and have certain effects on controlling the tunnel displacement. However, at the portals, in the shallow overburden tunnel section with high water content and within the zone of fracture angle of the stratum, systematic rock bolting is generally not provided. Instead, such measures as strengthened steel rib support, shotcrete support and reinforcement mesh support or increasing rockbolts at toe of walls of steel rib support can be taken.

14.7.3 Foundation consolidation measures include steel tube piles, compaction piles, jet grouting piles and root piles. Consolidation measures should be adapted to the tunnel space and construction time requirements.

14.7.4 Due to the porosity and collapsibility of loess, it is water-softening and its shear strength and compressive strength are significantly reduced with the increase of water content. Water is extremely harmful to the integrity and stability of the loess stratum, and the reaction is sensitive. In order to reduce surface water seepage, the surface gully, sink holes and cracks near the tunnel shall be backfilled and paved, and the surface water drainage facilities shall be provided.

14.7.5 When the groundwater volume is large and the groundwater level is higher than the tunnel excavation range, well point dewatering can be used in the surface or the tunnel to reduce the groundwater level to 1.5m below the arch invert. Measures such as plugging and drainage are taken against the water seepage and water streams in the tunnel to prevent the arch foot or elephant footing and tunnel invert from being soaked.

14.7.6 The collapse deformation of loess is a significant additional subsidence caused by the rapid failure of soil structure when the loess is wetted by water under certain pressure. Depending on its collapsibility, loess can be classified into two types, namely self-weight collapsibility and non self-weight collapsibility. The self-weight collapsible loess means the loess which collapses under its own weight, and non self-weight collapsible loess means the loess which collapses under certain external load.

There are many measures to eliminate the collapsibility of loess, which shall be determined according to the specific engineering environment and the mechanical equipment used. If the collapsible loess layer below the tunnel arch invert is thin, it is usually replaced with3:7 lime soil. When the collapsible loess is deep and replacement is not feasible, the compaction piles, jet grouting piles, steel tube piles or root piles are often used to eliminate collapsibility.

14.7.7 Loess tunnel portal design shall be as specified below:

1 The design and construction of tunnel portal on loess foundation are basically the same as

those in other areas. Attention shall be paid to the interception, diversion and drainage of surface water. The toe of portal side and heading slopes and foundations susceptible to scouring shall be paved to avoid scouring damage from water. At the junction of side and heading slopes, excavation shall be done by chamfer method to reduce concentrated scouring by rainwater.

2 For tunnel portal wall on collapsible loess ground, the ground stabilization measures shall be taken for end wall, wing wall and cut-and-cover tunnel based on the physical and mechanical properties of loess, so as to eliminate the collapse settlement of ground. Alternatively, the foundation shall be founded on the non-collapsible loess layer.

14.8 High in-situ stress zone

14.8.1 In high in-situ stress zone, the tunnel axis direction and the maximum principal stress direction are arranged to intersect at a small angle in order to reduce the effect of in-situ stress on tunnel. When the maximum principal stress direction is perpendicular to the cavern cross-section, the tunnel lining cross-section structure bears the maximum pressure. In general areas, the tunnels mostly have curve-sided circular arch cross-section or horseshoe-shaped cross-section, with vertical stress as the maximum principal stress. In high in-situ stress area, stress comes from all directions, so circular cross-section shall be adopted to ensure uniform stress around cavern. In addition, smooth and round cavern cross-section can avoid stress concentration caused by sharp corner.

14.8.2 For tunnels in high in-situ stress zones, the main characteristics of surrounding rock instability are as follows: rock burst or stripping in hard rock and large deformations in soft rock, making the cavern clear dimension smaller. At present, there are many classification methods for rock burst and large deformation, but without a unified standard. For rock burst classification, there are strength, stiffness, energy, instability, fracture and shock wave initiation theories. For large deformation classification, one method is engineering analogy and judgement based on experience, and the other method is numerical analysis, including shear strength to compressive strength ratio, stress ratio and critical depth methods.

1 In general, rock burst will occur under the following 5 conditions:

1) Rock strength $R_b \geqslant 50$MPa.

2) In rock strata, the original initial stress $\sigma_0 \geqslant (0.15 \sim 0.2)\ R_b$ and the maximum principal stress is generally greater than 20MPa.

3) Surrounding rock class: Classes I, II and III.

4) Rock is dry and brittle, basically without developed joints.

5) Elastic strain energy is released during excavation.

2 In general, large deformations will occur under the following 5 conditions:

1) Surrounding rock is soft, with low uniaxial compressive strength, small internal friction angle and cohesion as well as obvious plastic and rheological properties and belongs to Class IV, V or VI surrounding rock.

2) In high in-situ stress ($R_c/\sigma_{max} < 7$) zones, the in-situ stress is far greater than the surrounding rock strength.

3) The lateral pressure coefficient (λ) is greater than 1.

4) Surrounding rock has high water content.

5) Support structure has insufficient stiffness and strength and is installed late and not closed in time.

The rock burst and large deformation classification standards are presented based on the conditions under which they occur and the experience in and scientific research results on tunnel construction in high in-situ stress zones in recent years.

14.8.3 Rock burst is the rock mass burst, stripping, ejection (projection), blast, shock, sounding and even vibration on tunnel face or tunnel wall under high in-situ stress conditions. The strain energy accumulated in original rock mass in a state of triaxial stress converts instantly into impact kinetic energy when the surrounding rock loses its stability after a free face is formed through excavation and exposure, so rock burst is a geological disaster which is destructive and may lead to personal injuries and equipment damage. Rock burst mostly occurs in sections with hard and intact rock mass, little or no groundwater. For a possible rock burst, the principle of "prioritizing prevention and combing prevention with control" shall be followed. Field in-situ stress test shall be conducted during construction to predict the possibility of rock burst. For Class I slight rock burst, full-face excavation may be adopted. For Class II moderate rock burst, heading-bench/invert parallel excavation sequence, full-face excavation or partial excavation may be adopted. For Classes III and IV severe rock burst sections, partial or advance heading excavation may be adopted by limiting excavation scale, slowing down construction and taking comprehensive measures such as short excavation round, dense surrounding holes, multiple rounds, timely support and change of surrounding rock stress conditions by advance stress relief.

14.8.4 For tunnels in high in-situ stress areas, large deformation means the phenomenon of progressive plastic deformation or obvious cavern clearance reduction, with obvious time effect, of tunnel surrounding rock which loses its self-supporting capacity wholly or partially and whose strain energy is released slowly under the action of high or extremely high in-situ stress. Large deformation of soft rock is caused by the excessive distortional pressure on support structure. Support structure can control surrounding rock deformation and maintain surrounding rock stability only when it fits the large deformation characteristic of soft rock. Therefore, forepoling (for consolidation) is recommended to strengthen and improve the surrounding rock properties and enhance the resistance of surrounding rock to deformation. Deformation shall be allowed during provision of active (flexible) support, so as to allow deformation of surrounding rock in a controlled way, generate a reasonable plastic zone and release certain energy, thus keeping the strength of surrounding rock in such a way that it does not decrease rapidly to produce a broken zone. The long-term stability of support system shall be ensured by strengthening control during construction. In the case of partial deformation of surrounding rock, passive (rigid) support shall be provided to increase the strength and stiffness of secondary lining structure and to prevent later deformation of surrounding rock.

14.9 Permafrost

14.9.1 Frost heaving or freezing-thawing hazard are very likely to occur in tunnels built in permafrost areas. For layout of a tunnel, its location shall be selected cautiously based on thorough investigation.

14.9.3 The design of portals in permafrost areas shall focus on waterproof and drainage, thermal insulation and protection of permafrost environment. The ratios of side and heading slopes at portals shall be adopted in such a way that the disturbance to original slope surface and the damage to vegetation are minimized. Excavated side and heading slope surfaces shall receive thermal protection treatment, thus minimizing the disturbance to the original heat balance.

14.9.4 Tunnel structure in a permafrost area is currently of two types, namely the shotcrete-and-rockbolt composite lining structure that is mostly applicable to a tunnel in frozen rock and the two pass cast-in-situ concrete lining structure which is mostly applicable to tunnel in frozen soil. Considering the complex mechanical properties of frozen soil (rock) and the formation of freezing-thawing circle due to change of surrounding rock temperature field and frozen soil environment caused by tunnel construction, the frost heaving or freezing-thawing hazard are likely to occur, producing frost heaving action on lining structure. Composite lining with curved wall and arch invert has good adaptability. Appropriate enlargement of clear section is intended to reserve a certain reinforcement space.

14.9.5 Cast-in-situ concrete which is placed at low temperature and has low hydration heat is used to reduce the disturbance to surrounding rock temperature field.

15 Tunnel Subgrade and Pavement

15.1 General requirements

15.1.1 ~ 15.1.3 For tunnels passing through underground strata, their embedment conditions and operation environment differ greatly from those outside them. Compared to road sections outside tunnels, the subgrade and pavement in tunnels have the following particularity:

1) Tunnel subgrade (floor) is in mountain and the groundwater has a greater influence on the subgrade and pavement;

2) Tunnels are tubular structures with narrow space, so automobile emissions accumulate inside them. The adhesion of these exhaust emissions, fume and dust to pavement surface in tunnel is greater than that outside tunnel. As oil stains and dust stuck onto pavement deteriorate the skid resistance of pavement and cannot be washed away by natural rainfall, their long-term action will affect the skid resistance of pavement;

3) In case of a fire in the tunnel, the temperature will have a greater influence on pavement in tunnel than outside tunnel;

4) As subgrade and pavement in tunnel are restricted by site conditions, the construction conditions are poor and they are difficult to maintain;

5) Driving safety is greatly affected in rainy days as water brought into portal section by vehicles reduces the skid resistance of pavement; and

6) Driving conditions in tunnels are generally adverse to traffic due to poor lighting and visual environment.

The particularity mentioned above makes the traffic volume, running speed, horizontal and vertical alignment parameters and weather conditions have a greater influence on traffic safety in tunnel than in general road sections. Considering that the resistance to corrosion by groundwater and to softening of tunnel pavement structures is higher than that outside tunnel, rigid pavement system is widely used in domestic tunnels since it has good water stability and good adaptability to environment. As semi-rigid pavement with low modulus of elasticity will shorten the service life of pavement, the semi-rigid and flexible pavement systems are rarely used.

Rigid pavement system includes pavement with cement concrete surface course (including steel fiber reinforced concrete pavement surface and continuously reinforced concrete pavement) and composite pavement consisting of upper asphalt mixture surface course and lower cement concrete (including steel fiber reinforced concrete pavement and continuously reinforced concrete pavement) surface course. Tunnel pavement design shall conform to relevant provisions of the current*Specifications for Design of Highway Cement Concrete Pavement* (JTG D40) and *Specifications for Design of Highway Asphalt Pavement* (JTG D50).

15.1.4 Improved drainage below the tunnel pavement can help to reduce hazards and significantly prolong the service life of pavement structures. In particular, the asphalt surface course is sensitive to water and has poor water stability, so the drainage is vital to a water-free environment.

15.2 Tunnel subgrade

15.2.1 For a tunnel provided with arch invert, the lining structure is enclosed, so the arch invert is required to be backfilled with concrete or rubble concrete to make the subgrade achieve better stability, density and homogeneity.

15.2.2 For a tunnel without arch invert, natural stone foundation is used as the tunnel subgrade. Since such subgrade is largely affected by groundwater, certain requirements are imposed on water stability and softening degree. A stable stone foundation means that the foundation is composed of massive ~ intact hard rock without significant softening. Medium hard rock or relative soft rock is used as natural foundation. A mountain tunnel is generally excavated by blasting, and blasting at tunnel bottom would have certain impacts on the integrity of surrounding rock.

15.3 Tunnel pavement

15.3.1 With regard to tunnel pavement, almost all tunnels in Europe have asphalt pavement, while Japan uses cement concrete pavement. Before 2000, most highway tunnels in China had cement concrete pavement. Since the issuance and implementation of the *Specifications for Design of Highway Tunnel* (JTG D70-2004), composite pavement has been used in more and more expressway tunnels and Class-1 highway tunnels. Composite pavement is applied more and more because it can significantly improve the traffic safety as well as reduce accident rate. Therefore, the Specifications recommend composite pavement consisting of upper asphalt mixture surface course and lower cement concrete course as the Class-1 highway and expressway tunnel pavement. However, in view of the vast territory of China, big regional difference, the unbalanced development, the different traffic volume and transportation conditions and the different requirements, composite pavement or cement concrete pavement may be used in tunnels of other classes of highways, depending on the traffic conditions, local characteristics, material supply and economical analysis.

15.3.2 As over-excavation and under-excavation of rock subgrade exist, the pavement of tunnel without arch invert shall be provided with base course which can double as leveling course. In the case of large over-excavation of the invert or construction needs, a separate leveling course can be provided. For a tunnel with arch invert, the arch invert filling can play a role in leveling and as rigid base course, so it is possible not to lay base course.

15.3.3 The frequency of groundwater action on pavement base course in tunnel is higher than that outside tunnel, thus rigid base course with good water stability should be used. Plain concrete materials with high strength and good stability are recommended.

If a leveling course is provided, its masonry quantities, which should not be less than 15cm can be determined based on the permissible average over-excavation depth of tunnel floor. It is found in the survey for this revision that the leveling course of 100 ~ 150mm thick is not thick enough.

15.3.4

1 As indicated by years of engineering practice in some provinces, the jointed cement concrete surface courses are generally used in Classes II, III and IV highway tunnels at present, and are in good service condition.

2 The cement concrete pavement of Class-1 highways and expressways is provided with jointed surface course or lower surface course of ordinary cement concrete (including continuously reinforced concrete and steel fiber reinforced concrete). The use of reinforcement or steel fiber reinforced concrete in surface course enables the reduction of reflection cracks, improvement of pavement durability and performance, and reduction of maintenance costs, thus they are promoted in the Specifications.

3 During design, consideration shall be given to the small temperature difference in the tunnel, so the spacing between construction joints and between expansion joints of surface slabs in tunnel can be greater than that outside tunnel.

4 Considering the difficulty in construction and maintenance in tunnel, the thickness of cement concrete pavement slab in tunnel should be slightly greater than or at least equal to that outside tunnel. If pavement structure is designed in accordance with reliability design criteria, the variable coefficient C_V of its material properties and structural dimension parameters should be within the medium - high range of variable level.

5 See Table 15-1 for the surface texture depth requirements of cement concrete surface courses of all classes of highways as listed in Table 4.5.7 of the *Specifications for Design of Highway Cement Concrete Pavement* (JTG D40-2011).

Table 15-1 Surface texture depth requirements of cement concrete surface courses of all classes of highways (mm)

Highway class	Expressway and Class-1 highway	Classes II, III and IV highways, vehicle cross passages
General road section	0.70-1.10	0.50-1.00
Special road section	0.8-1.2	0.6-1.1

Notes:

1. Special road section—It means the flyovers, level crossings or speed change lanes for expressways and Class-1 highways and means the sharp turns, abrupt slopes, intersections or the vicinity of market towns for other classes of highways.
2. In regions receiving less than 600mm of precipitation annually, the values listed in the table can be decreased properly.

In recent years, the main problems of cement concrete pavement of highway tunnels are that the surface lacks skid resistance and the surface adhesion coefficient (friction coefficient) is low. Therefore, this revision considers the pavement surface texture in tunnel as that in special road sections and states that the pavement surface texture shall be wear resistant. This provision specifies that the surface texture depth under unfavorable conditions of the tunnel (such as sections with very heavy traffic, heavy traffic, sharp turns and continuous long and abrupt longitudinal slopes) shall be given a high value. This is to further improve the tunnel operation safety.

Research and actual measurement indicate that the longitudinal grooves mainly increase the transverse sliding or steering friction force to prevent side slide and that the transverse grooves mainly increase the longitudinal braking friction force to shorten the braking distance. Hence, transverse grooves can be used in general road sections of Class II (and below) highway tunnels. Longitudinal grooves or transverse grooves or the combination of both should be used on the tunnel pavement in large longitudinal slope section, expressways and Class-1 highways to improve skid resistance.

6 Tunnel pavement overlays are mainly targeted at expressway and Class-1 highway tunnel pavement lacking skid resistance and at the old road reconstruction for tunnels of other classes of highways.

15.3.5 According to relevant calculation and research, if rigid base course specified herein is adopted, the tensile stress at the bottom of the surface slab is low when the cement concrete pavement surface course is under the action of vehicle loads. Therefore, the main purpose of reinforcement is to control the concrete shrinkage and the generation of shrinkage cracking. The calculation method in Appendix E "Longitudinal reinforcement calculation for continuously reinforced concrete surface course" to the *Specifications for Design of Highway Cement Concrete Pavement* (JTG D40-2011) should not be used indiscriminately. Hence, this revision reduces the pavement reinforcement ratio in tunnel.

15.3.6

1 If the materials and performance indexes of asphalt pavement in tunnel are the same as those of asphalt pavement outside tunnel, the tunnel environment requirements generally can be met. Therefore, this provision specifies that the performance requirements shall conform to the current *Specifications for Design of Highway Asphalt Pavement* (JTG D50).

2 The tunnel is a semi-enclosed narrow long space which is difficult to maintain and repair, and the total thickness of asphalt surface course of composite pavement is 80-100mm. The use of asphalt surface course with such thickness in completed tunnel works has produced good results, so the value specified in the last edition remains unchanged.

3 The type of mixture should be the same as that outside tunnel in order to facilitate its paving and maintenance together with that outside tunnel. Hot mix asphalt mixture has been used in some extra-long tunnels. Selection of hot mix additives without influence on pavement performance of mixtures can significantly improve the working environment of paving.

4 Arrangement of a tack coat between the lower asphalt surface course and the concrete pavement slab can enhance the bonding between courses and avoid slipping between courses.

5 Arrangement of such measures as reinforced geomaterials or stress absorbing layer can

effectively reduce the reflection cracks occurring at deformation joints.

15.3.7 At the parts of concrete pavement slab susceptible to cracking or crack opening, a leveling course may be arranged on the lower concrete surface course or on the floor of reinforced concrete structure to reduce the reflection cracks of asphalt surface course. The main purpose of specifying a leveling course is to strengthen the bonding performance between asphalt surface course and the lower cement concrete surface course, thus strengthening the antistripping performance with the upper asphalt surface course and reducing the damage to structure caused by paving. This provision is in reference of the provisions on cement concrete bridge deck pavement in the *Specifications for Design of Highway Asphalt Pavement* (JTG D50).

15.3.8 If the type of pavement surface course in tunnel is inconsistent with that outside tunnel, the skid resistance of pavement will not be the same, thus affecting the traffic safety.

1 The total length of tunnel lighting entrance section and transition section shall be calculated based on the *Guidelines for Design of Lighting of Highway Tunnels* (JTG/T D70/2-01-2014), and the requirement shall be higher than that on horizontal and vertical alignment for 3s at portal set out in the *Technical Standard of Highway Engineering* (JTG B01-2014).

2 The provision is the same as the provision on horizontal and vertical alignment at portal specified in the *Technical Standard of Highway Engineering* (JTG B01-2014).

15.3.9 When concrete pavement and asphalt pavement is connected in a tunnel, the transition section can be arranged as indicated in Fig. 15-1.

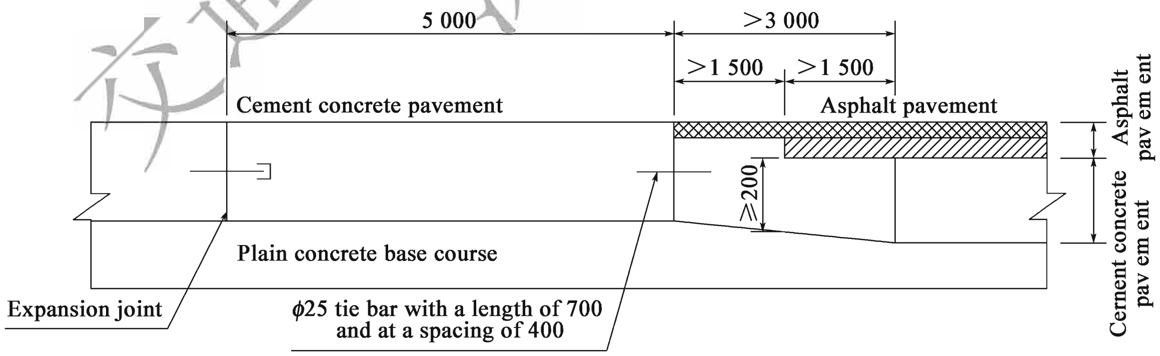


Fig. 15-1 Transition section for connection of concrete pavement and asphalt pavement (unit: mm)

16 Seismic Design

16.1 Seismic design classification and standard

16.1.1 The seismic fortification classification and standard of highway tunnels are developed based on the *Standard for Classification of Seismic Protection of Building Constructions* (GB 50223), the *Specification of Seismic Design for Highway Engineering* (JTG B02) and their own characteristics of highway tunnels.

The seismic fortification classification and standard in the *Specification of Seismic Design for Highway Engineering* (JTG B02-2013) are generally reasonable. However, with the rapid development of China's highway tunnel construction, especially the large-scale construction of extra-long tunnels, it is necessary to make proper adjustment to the classification of seismic fortification for tunnels based on their own importance, so as to make this classification more workable in seismic design. Seismic fortification of tunnels is classified into four categories, i.e. A, B, C and D. Category A tunnel means large underwater tunnel, which is not included herein. The scope of application of Categories B, C and D tunnels listed herein is specified in reference to the safety classification standards of highway tunnel structures.

16.1.2, 16.1.3 In accordance with the overall seismic fortification objective of "no damage under minor earthquakes, repairable damage under moderate earthquakes and no collapse under major earthquakes", in view of the continuity of and consistency with the current specification for seismic design about seismic performance objectives, and in reference of the new objectives of seismic design for tunnels at home and abroad, the Specifications hereby specify that the seismic fortification objective of Category B tunnels is no damage under E1 earthquake effect (with a return period of 75 years), limited damage under E2 earthquake effect (with a return period of about 1,000 years) and enabling normal traffic flow after earthquake; that the seismic fortification objective of Category C tunnels is no damage under E1 earthquake effect (with a return period of 50 years) and no partial or whole collapse under E2 earthquake effect (with a return period of about 475 years), and that the seismic fortification objective of Category D tunnels is no damage under E1 earthquake effect (with a return period of 30 years).

The seismic fortification standards specified herein are basically maintained at the same level as the*Specification of Seismic Design for Highway Engineering* (JTG B02-2013). But the seismic design method has changed a lot. The method of two-level seismic fortification and two-stage seismic design is adopted. The first stage of seismic design is elastic seismic design and the second stage of seismic design is plastic seismic design. Through the first stage of seismic design, namely the seismic design corresponding to E1 earthquake effect, the seismic fortification level can equal to that of the *Specification of Seismic Design for Highway Engineering* (JTG B02-2013). The second stage of seismic design, i. e. seismic design corresponding to E2 earthquake effect, is aimed at ensuring that the structures have enough plastic deformation capacity, and checking calculation is made to ensure that the structures will not collapse. Through the design of seismic measures, the structures are guaranteed to have enough earthquake resistance capacity.

According to a large number of results of investigation into earthquake damage to tunnels, the seismic performance of underground works is superior to that of ground structures and the tunnel structures are rarely damaged in regions with actual seismic intensity of VI ~ VI degrees. Therefore, the highway tunnels in regions with the basic seismic peak ground acceleration of not more than 0. 10g may be designed with seismic measures only.

16.2 Earthquake effect

16.2.1 In general, special seismic safety evaluation for engineering sites is unnecessary in seismic design of highway tunnels, and other earthquake effects are explicitly stipulated in these Specifications.

16.2.2 The provisions of the *Seismic Ground Motion Parameters Zonation Map of China* (GB 18306) are directly quoted in Table 16.2.2 herein. The seismic fortification ground motion grading and seismic level of fortification are given to facilitate the identification of earthquake-induced liquefaction of site and the determination of structural seismic measures.

16.2.3 As can be seen in Tables 16-1 ~ 16-3, considering the importance coefficients and different comprehensive influence coefficients, the design ground motion parameters of Category B tunnels is within a range of (0. 34 ~ 0. 425) A, and the design ground motion parameter corresponding to a return period of 75 years is 0. 426A; the design ground motion parameters of Category C tunnels is within a range of (0. 26 ~ 0. 325) A, and the design ground motion parameter corresponding to a return period of 50 years is 0.34A; the design ground motion parameters of Category D tunnels is within a range of (0.20 ~ 0.25)A, and the design ground motion parameter corresponding to a return period of 30 years is 0.255A.

Table 16-1 Ground motion parameters calculated based on importance coefficient and comprehensive influence coefficient in the *Specification of Seismic Design*

Comprehensive influence coefficient	Importance coefficient			
	1.7	1.3	1.0	0.8
0.20	0.34A	0.26A	0.20A	0.16A
0.25	0.425A	0.325A	0.25A	0.20 A

Note: "A" means the design basic seismic peak ground acceleration.

Table 16-2 Return periods corresponding to different importance coefficients and comprehensive influence coefficients in Table 16-1 (Year)

Importance coefficient	Comprehensive influence coefficient			
	1.7	1.3	1.0	0.8
0.20	50	31	21	16
0.25	75	46	29	21

Table 16-3 Return period of design earthquake considering only the importance coefficient (Year)

Importance coefficient	Probability of exceedance in ··· years	Return period
1.7	0.048%	About 2,000 years
1.3	0.106%	About 1,000 years
1.0	0.210%	About 475 years
0.6	0.370%	About 270 years

This shows that, for E1 earthquake effect, the design ground motion parameters can be adjusted by introducing different importance coefficients and that it is appropriate to adopt elastic design and cancel the comprehensive influence coefficient. The importance coefficients of Categories B, C and D are 0.43, 0.34 and 0.26 respectively, and the corresponding return periods of design ground motion are about 75, 50 and 30 years respectively.

For E2 earthquake effect, the importance coefficients of Categories B and C are basically the same as those in the *Specification of Seismic Design for Highway Engineering* (JTG B02-2013), which are 1.3 and 1.0 respectively, and the return periods of design ground motion are about 1,000 and 475 years respectively. The *Specification of Seismic Design for Highway Engineering* (JTG B02-

2013) only adopts the one-stage design, in which elastic seismic design is adopted after the seismic force has been reduced by introducing the comprehensive influence coefficient. The implied meaning is that the structures are allowed to go into a plastic state, so there are corresponding requirements for the plastic deformation capacity of structures. However, the necessary plastic seismic design is not carried out in the design, thus it is uncertain whether the structural plastic deformation requirements can be met or not. This is also a major defect of JTG B02-2013. Therefore, the Specifications stipulate the seismic design stage of E2 earthquake effect and clearly define the plastic seismic design to make up for the deficiency of JTG B02-2013.

16.2.4 However, according to the Specifications and the current *Evaluation of Seismic Safety for Engineering Sites* (GB 17741), the earthquake effect levels for the tunnels that need to go through seismic safety evaluation for engineering sites shall not be lower than corresponding provisions in the Specifications, namely not be lower than the E1 and E2 earthquake effect levels listed in Table 16.2.3. As the seismic safety evaluation for engineering sties generally gives the ground motion parameters in a form of "an exceedance probability in a design reference period", the exceedance probability of earthquake effect for the tunnels going through the seismic safety evaluation for engineering sites shall not be lower than the exceedance probability corresponding to the seismic importance coefficients in Table 16.2.3. Therefore, the earthquake effect for tunnels going through the seismic safety evaluation for engineering sites may be given as per the recommended values in Table 16-4.

Table 16-4 Earthquake effect levels for tunnels subject to seismic safety evaluation for engineering sites

Seismic fortification category	E1	E2
B	63% exceedance probability in a design reference period of 75 years	10% exceedance probability in a design reference period of 100 years
C	63% exceedance probability in a design reference period of 50 years	10% exceedance probability in a design reference period of 50 years
D	63% exceedance probability in a design reference period of 30 years	—

16.3 Seismic design check

16.3.1 Seismic design check is to determine the appropriate performance of checking calculation objects after the seismic performance requirements have been determined. At present, the main performance selected includes stress level (strength), serviceability function (deformation amount, crack width, joint opening, etc.) and stability of soil around tunnel. The specific permissible values shall be determined by considering the structure importance, earthquake effect level, structure type and surrounding rock conditions.

The load combination modes under earthquake effect are specified herein and in the *Specifications for Design of Highway Underwater Tunnel*.

16.3.2 For seismic design check under E1 earthquake effect, the structural performance shall be within an elastic range and the structural stress level shall be within the elastic limit, with relevant provisions given in the current seismic design specifications. Hence, drill-and-blast tunnels shall directly follow the provisions on structural strength safety factors in the *Specification of Seismic Design for Highway Engineering* (JTG B02-2013) and cut-and-cover tunnels can refer to the provisions on structural strength checking calculation in the *Code for Seismic Design of Buildings* (GB 50111).

16.3.3 For drill-and-blast tunnel structures, comprehensive and systematic seismic response analysis of arched (horseshoe-shaped) highway tunnel was carried out in the preparation of the Specifications. A lot of numerical calculations were made for stiffness, depth and structural stiffness of different rock and soil mass based on the elastoplastic damage of concrete. The earthquake resistance capacity curve of tunnel structure based on damage degree was obtained, and its correspondence with earthquake damage was studied. Based on the study of its structural response sensitivity and performance index characteristic, the maximum deformation rate (i.e. the maximum convergence) of the tunnel was selected as the seismic performance index. The typical earthquake resistance capacity curve was studied and the performance was divided into three levels, namely intact structure, minor damage and severe damage. The impacts of such links as design, construction and maintenance are taken into overall consideration. Through statistical analysis, the final recommended performance index thresholds are as follows: The maximum convergence is 5‰ in the case of minor damage and 15‰ in the case of severe damage.

16.3.4 According to the engineering geological conditions in which the tunnel is located and the importance and structural characteristics of the tunnel, the current seismic calculation of tunnel is made in three main aspects, i.e. transverse section, longitudinal section and 3D space model. The main methods of seismic calculation for transverse section are static method, response displacement method and dynamic analysis method (time history). The main methods of seismic calculation in longitudinal direction are response displacement method and dynamic analysis method (time history).

In static method, the considered earthquake effect on structures is composed of three main parts, i.e. inertial force generated by dead weight, earthquake effect on soil column on tunnel roof, and increment of seismic lateral soil pressure. This method is applicable to drill-and-blast highway tunnels with large part and wide area in rocks.

Response displacement method adopts the displacement difference between strata around tunnel, the shear force on the perimeter of tunnel structure and the structural inertial force in the case of an earthquake as the seismic loads. This method applies to tunnels whose seismic response is mainly controlled by the relative displacement of strata, such as the shield-driven or cut-and-cover tunnels commonly used in cities and the shield-driven or immersed tube tunnels commonly used across rivers, lakes and seas.

Dynamic analysis method (time history) has high accuracy. It can consider the non-linear characteristics of tunnel surrounding rock and structure, as well as the seismic responsiveness of tunnel in each direction. This method is applicable to any type of tunnel, but it is time-consuming and strenuous, and the calculation and result analysis put high demands on calculation personnel.

16.4 Seismic measures

16.4.1 In general, seismic resistance of tunnel is favorable in hard and intact rock mass and unfavorable in adverse geological section. The seismic resistance of deep tunnel is favorable, while that of shallow tunnel is unfavorable. Compared to tunnel body, the seismic resistance of portal and side & heading slopes is unfavorable. Especially at unfavorable geologies like rock pile, landslide mass, debris flow gully, collapse and surrounding falling rocks, and at low-lying place with difficulty in drainage or under the unstable cliff, a strong earthquake will lead to deformation of mountain.

16.4.2 This provision mentions the excavation heights of portals, cut slopes and heading slopes in seismic regions. In sections with poor rock mass integrity and soil properties, due to long-term weathering and erosion, collapse and rockfall are very likely to happen and block the portals in an earthquake, thus endangering the traffic safety. Therefore, it is required that the excavation heights of portals should be strictly controlled and that cut-and-cover tunnels or other effective protection measures should be implemented in portal sections with unfavorable terrain to ensure safety.

16.4.5 The integrity of structure is one of the important factors affecting its earthquake resistance capacity. The construction joint between end wall at tunnel portal and lining ring frame, as well as between end wall and retaining wall or wing wall, and the wing wall structures in cantilever form of cut-and-cover tunnel are weak links of seismic resistance, so seismic measures of connection reinforcement shall be taken.

16.4.6 The opening section, shallowly-embedded section or section under eccentric load of a tunnel shall be the key points of seismic fortification. The lining structures in these sections shall be reinforced in consideration of the class of the surrounding rock. For the length of tunnel reinforcement, the length of seismic fortification section is calculated in accordance with the minimum cover thickness of soil at tunnel shoulder and the variation of longitudinal slope at the openings of tunnel, as well as the clear section width and surrounding rock class. In practice, the topographical and geological conditions of tunnel are very complicated, so its fortification length shall be taken with proper allowance made for unforeseen circumstances based on specific construction conditions.

16.4.7

4 The expanded size of transverse section after an earthquake can ensure the clearance area of the section, thus providing surplus space for subsequent repair. Over-excavation volume is determined through overall consideration of such factors as the seismic intensity, surrounding rock conditions and clear section.

Seismic joint can double up as construction joint and settlement joint. It is wider than general settlement joint or construction joint, so it can effectively cater for longitudinal dislocation of tunnel after an earthquake. In addition, the seismic joint shall be filled and compacted. Tunnel shall be provided with proper waterproof measures, such as back bonded waterstops and embedded rubber water stop with steel edges. The physical and mechanical properties of waterstops shall meet the dislocation and deformation requirements of tunnel structure.

16.5 In-tunnel facilities

16.5.1 ~ 16.5.3 In-tunnel facilities include in-tunnel auxiliary structures, ventilation and lighting facilities, and traffic engineering facilities. For seismic design of highway tunnel, the in-tunnel facilities need to be kept stable against earthquake and prevented from damage and the resulting impacts on structures, pedestrians and traffic safety.

17 Revamping Design

17.1 General requirements

17.1.1 Highway tunnel reconstruction and extension means the expansion or reconstruction of existing tunnels or construction of additional tunnels by utilizing the alignments and corridors of the existing tunnels to raise the highway class, improve the traffic conditions and increase traffic capacity, including expansion of the existing two-tube four-lane highway tunnel into a two-tube six-lane or eight-lane highway tunnel, and reconstruction of the existing single-tube two-way traffic tunnel into a two-tube one-way traffic tunnel.

Reconstruction: lining structure reinforcement, roadway adjustment, pavement re-surfacing, drainage ditch reconstruction and cable trench reconstruction for and addition of cross passages to existing tunnels, as well as local improvement of technical indexes and safety performance, improvement of service functions, etc.

Extension: expanding excavation of existing cross section, increase of section clearance, and removal of existing lining for reconstruction.

New construction: construction of additional tunnels in parallel to existing tunnels.

17.1.2 Prior to tunnel reconstruction and expansion design, it is necessary to investigate the current design, construction and operation status of the existing tunnel, including: the design drawings, relevant geological data in design and construction phases, design changes and as-built drawings of the existing tunnel; the portal conditions and surrounding buildings; tunnel drainage facilities and drain capacity; structural and defects inspection, maintenance and strengthening data of tunnel structure, as well as the current status of structure distress and the operational status of tunnel.

17.2 Tunnel reconstruction and expansion scheme design

17.2.1 Tunnel reconstruction and expansion are based on partial or full utilization of the existing highway tunnel. In general, the normal traffic on the existing tunnel is maintained during reconstruction and expansion; and the existing tunnel structures and traffic operation conditions may be affected. Therefore, for tunnel reconstruction and expansion, the status of the existing tunnel shall be first investigated and analyzed in detail. The technical standards shall be determined reasonably. The existing tunnel shall be utilized to the greatest extent to save construction cost. For tunnel reconstruction and expansion scheme, technical and economical comparison should be made among multiple schemes to select the tunnel reconstruction and expansion scheme that can meet the traffic function and traffic safety.

17.2.2 Like new tunnel construction, the additional tunnel construction and tunnel expansion shall comply with the current technical standards.

17.2.3 Expansion of a two-tube four-lane tunnel into a two-tube six-lane tunnel in the original location is economical and can provide good operational conditions.

17.2.4 There are four ways to expand the existing two-tube four-lane tunnel into an eight-lane highway tunnel: (1) Expansion of the existing separated two-tube four-lane tunnel into a two-tube eight-lane tunnel in the original location, i.e. expansion in original location, as shown in Fig. 17-1; (2) Utilization of the existing tunnels and construction of two additional two-lane tunnels to form a four-tube two-way eight-lane tunnel, as shown in Fig. 17-2; (3) Utilization of one existing tunnel, construction of a new two-lane tunnel, and expansion of the other existing tunnel into a four-lane tunnel, as shown in Fig. 17-3; and (4) Expansion of one existing tunnel into a four-lane tunnel, construction of a new four-lane tunnel, and use of the other existing tunnel as an service tunnel or an emergency standby tunnel, as shown in Fig. 17-4. Expanding excavation in the original location can be done on a single side or on both sides, as shown in Fig. 17-1.

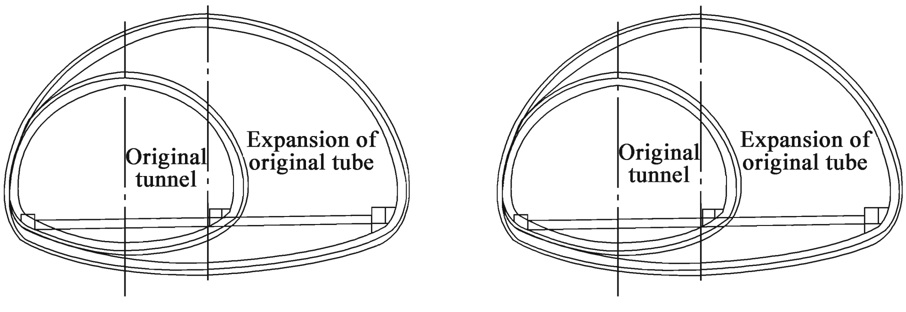


Fig. 17-1 Tunnel expansion in original location

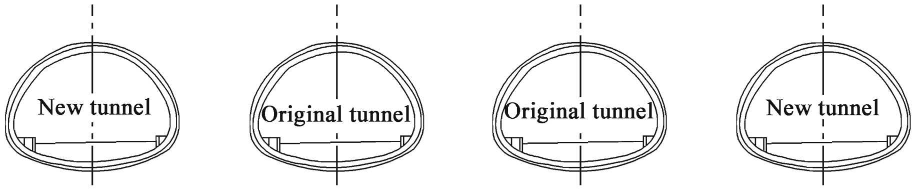


Fig. 17-2　Construction of two new two-lane tunnels

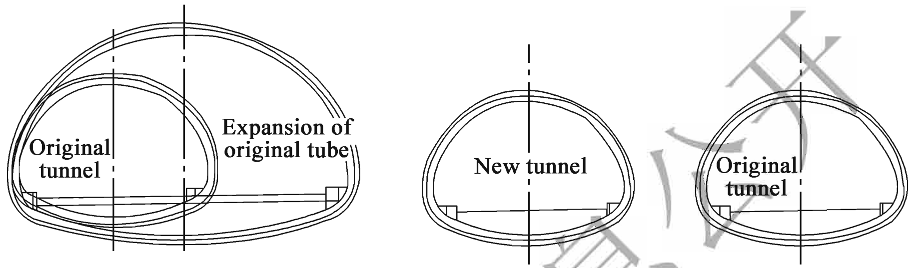


Fig. 17-3　Utilization of one existing two-lane tunnel, construction of a new two-lane tunnel and expansion of the other existing tunnel into a four-lane tunnel

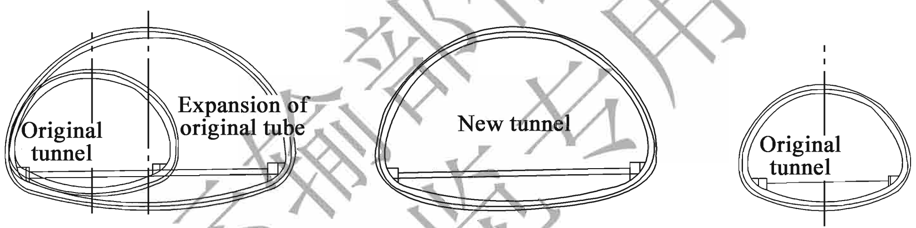


Fig. 17-4　Expansion of one existing tunnel in original location and construction of a new four-lane tunnel

17.2.5 ~ 17.2.6　Expansion of a four-lane twin-arch tunnel is complicated and its demolition is of high risks, so it shall remain unchanged whenever possible.

17.2.7　If vehicles in the same direction are separated by two tunnels, the sudden change of traffic conditions is likely to cause traffic accidents. Therefore, necessary traffic safety facilities should be provided.

17.2.9　After reconstruction and expansion, if an existing tunnel that will no longer be open to traffic should not be abandoned, it may be used as repair & maintenance service channel and emergency rescue channel. The existing tunnel used as repair & maintenance channel and emergency rescue channel shall be capable of ensuring the long-term stability of tunnel structures.

17.2.10　Completion of additional parallel tunnel construction before tunnel expansion is intended to ensure normal traffic operation during construction. Additional tunnel construction may affect the structure and traffic of the existing tunnel being open to traffic, thus the existing tunnel structures should be temporarily protected or reinforced.

17.2.11 Tunnel reconstruction and expansion have a great influence on existing traffic and the society, so it is generally required that the traffic should not be interrupted, which complicates the traffic organization. Hence, the reconstruction and expansion design shall contain construction scheme design and traffic organization design.

17.3 Tunnel expansion

17.3.1 Expansion of an existing tunnel shall make full use of the alignment occupied by the existing tunnel. In addition, to reduce investment, the alignment and elevation of tunnel after expansion in the original location shall be the same as those of the existing tunnel. This is also intended to keep the locations, elevations, distances and sizes of emergency stop zone, vehicle cross passage and pedestrian cross passage in the existing tunnel unchanged.

17.3.2 Removal and temporary support of existing structure is a very important link in tunnel expansion. Improper removal method, removal at random and delayed support may cause safety accidents, bring excessive secondary disturbance to and affect stability of surrounding rock, so detailed design should be carried out. Removal, expanded excavation method and temporary support measures of original tunnel lining have a great effect on the surrounding rock stability and structural safety of tunnel, thus it is necessary to calculate the structural stress and surrounding rock stability during expansion.

The structural calculation for tunnel expansion is complex. Structural stress and surrounding rock stability need to be analyzed and calculated under multiple working conditions.

17.3.3

2 If the collapse height of the existing tunnel is less than the equivalent height of the uniformly-distributed vertical pressure of surrounding rock calculated in accordance with the Specifications, the rock load during reconstruction and expansion can still be considered by reference to the surrounding rock class and pressure of a new tunnel. If the collapse stack height of existing tunnel is larger than the equivalent height of uniformly-distributed vertical pressure of surrounding rock calculated in accordance with Eq. (6-2) herein, the uniformly-distributed load of existing tunnel can be calculated as per the

collapse stack height.

17.3.4 Expansion of existing tunnel may be done by expanding excavation on a single side or both sides. For expanding excavation on a single side, the disturbance to surrounding rock is small and the range of removing primary support is also small. For expansion of existing tunnel, there should not be any abandoning or backfilling of the excavated space of existing tunnel.

17.4 Tunnel reconstruction

17.4.1 Investigation of existing tunnel includes:

(1) Basic information of tunnel such as construction date, location, length, horizontal and vertical alignment, cross-section geometry and geological conditions;

(2) Tunnel lining structure type, construction method and tunnel clearance, building materials and corrosion, lining strength and lining backfill conditions;

(3) Tunnel disease status quo and previous maintenance & reinforcement, waterproof and drainage status;

(4) Arrangement and use of ventilation, lighting, fire protection and monitoring equipment in tunnel, and the arrangement of equipment cavern;

(5) Tunnel portal type & dimension, foundation depth and building materials, current status of heading slope, cut slope and protection works at the portals, and water intercepting and drainage conditions at the portals;

(6) Whether there are such hazards as unstable rock, collapse, debris flow and snowdrift at the portals; and

(7) Technical status and operation of tunnel road sections.

The technical status quo and safety of the existing tunnel shall be evaluated based on the investigation results for tunnel reconstruction.

17.4.2 In general, tunnel reconstruction design shall comply with the current *Technical Standard of Highway Engineering* (JTG B01). However, in most cases, the alignment and cross-section clearance of the existing tunnel comply with the original "technical standard", the requirements of which are lower than those of the current "technical standard". If the current "technical standard" is adopted, the existing clear section needs to be enlarged, and the existing lining will be removed for expansion. This will lead to large work quantity, long construction period, increase of investment and great effects on the current traffic. To save investment and reduce disturbance to traffic, the original "technical standard" adopted in construction can be maintained.

The main purpose of tunnel reconstruction is to treat the tunnel defects and deficiencies. If it affects the tunnel structure safety and traffic safety, the cover arch scheme, which is the most commonly used structure strengthening method and an effective method to improve waterproof and drainage conditions, can be adopted. However, the reduction of clear section may lower the clearance standard. In this way, the investment can be saved and the impacts on traffic will be reduced. Nevertheless, the basic driving conditions need to be ensured, i. e. ensuring the carriageway width, lateral width and height. If the width is not enough, the maintenance access or sidewalk may be narrowed properly. If the height is not enough, the pavement elevation may be reduced; moreover, operation safety supporting facilities can be added, including arranging width, height and speed limit signs, contour light strips and refugees.

17.5 Additional tunnel construction

17.5.1 ~ 17.5.3 The structural design of additional tunnel shall be essentially the same as that of new tunnel and comply with provisions in other sections herein.

17.5.4 In order to reduce the damage to existing tunnel structures and ensure their stability, the cross passage opening location shall be kept at a certain distance from the construction or deformation joints of the existing tunnel.

17.5.5 To ensure the safety of the adjacent existing tunnels and caverns in the construction of an additional tunnel, the impact of blast vibration on the adjacent existing tunnels shall be regarded as the key content to be monitored. According to requirements of the *Safety Regulations for Blasting* (GB 6722), the permissible critical vibration velocity for safety of tunnel in operation shall be generally within a range of 100mm ~ 200mm/s. The standard values of blast vibration velocities under control in different conditions can be proposed based on surrounding rock class and the clear distance between tunnels. In general, the value shall be within a range of 80mm ~ 200mm/s in Class III surrounding rock section, within a range of 50mm/s ~ 150mm/s in Class IV and not more than 100mm/s in Class V. In actual construction of additional tunnels, the blasting vibration velocity should be determined via comprehensive test based on tunnel conditions.

18 In-tunnel Reservation, Embedment and Structures

18.1 General requirements

18.1.1Highway tunnels usually need to be provided with a certain number of equipment caverns to place various electrical, communication and fire protection equipment, as well as the embedded parts to ensure the contact, control, installation and maintenance of these equipment and caverns. The main equipment caverns include distribution cavern, transformer cavern, fire extinguisher cavern and emergency call cavern. Main embedded parts include grounding flat steel, fan hanger and cable conduits. These reserved caverns and embedded parts shall be designed in accordance with the traffic engineering and mechanical & electrical engineering requirements.

18.1.2 Reserved caverns and embedded parts (especially reserved caverns) need to be made on lining structures, which may change the stress conditions of tunnel structures and cause adverse effects. Hence, corresponding structure and construction measures should be taken to ensure the loading-bearing capacity of tunnel lining structures.

18.2 Reservation and embedment

18.2.1 The locations and sizes of equipment caverns in tunnel are generally determined according to needs of equipment for safe operation in tunnel. To facilitate construction and management, the equipment caverns should not have too many geometric shapes and size types, and should be standardized as far as possible.

(1) The size of a distribution cavern is determined according to equipment product and generally 800mmx 950mm x 400mm (Width x Height x Depth). The bottom of a distribution cavern is generally about 1.10m above the top surface of maintenance access

or sidewalk.

(2) The size of a transformer cavern is determined according to equipment product and generally 2.5m x 3m x1.8m (Width x Height x Depth). Two vertical 500mm x 600mm cable troughs are arranged below a transformer cavern to connect the cable trenches in main tunnel.

(3) The spatial size of a fire extinguisher cavern varies with the type of fire protection equipment to be placed. The common fire protection equipment includes in-tunnel fire hydrant, aqueous film-forming foam apparatus and fire extinguisher, as shown in Figs. 18-1 and 18-2.

(4) The structure of an emergency call cavern can be arranged by reference to Fig. 18-3.

18.2.2 General reserved caverns have no impacts on structures and need no special treatment. But larger reserved caverns may have harmful effects on the load-bearing capacity of structures. The larger the reserved cavern size is, the deeper its invasion into the structure is and the greater its impact on lining structure is. Therefore, the size of a reserved cavern and its depth in structure shall be minimized. In general, a reserved cavern less than 1500mm in width needs no special treatment. For a deeper reserved cavern, the remaining thickness of lining structure is generally not less than 100mm. When a reserved cavern meets the rebar in lining, the rebar needs to be cut off or the rebar bypass the cavern. In unreinforced lining section, the addition of constructional steel bar depends on the size of the reserved cavern. Arrangement of a reserved cavern at the deformation or construction joints of lining structure will have great effects on lining structure.

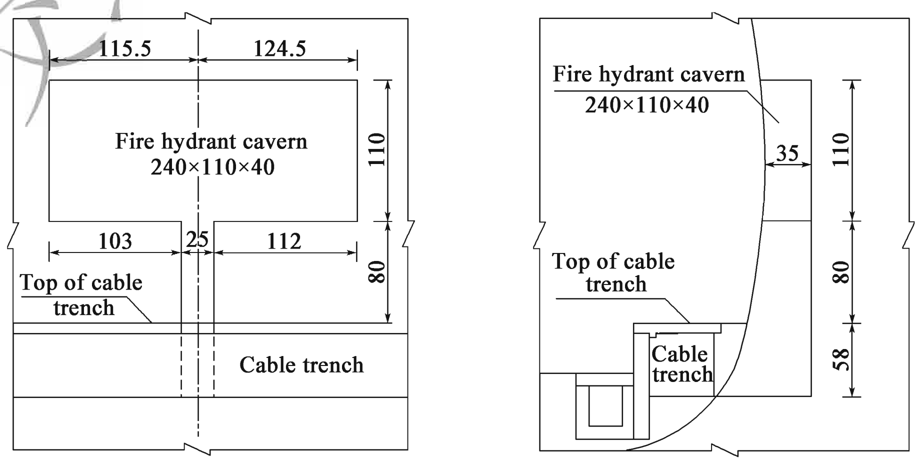


Fig. 18-1 Structure diagram of fire hydrant/extinguisher cavern (unit: cm)

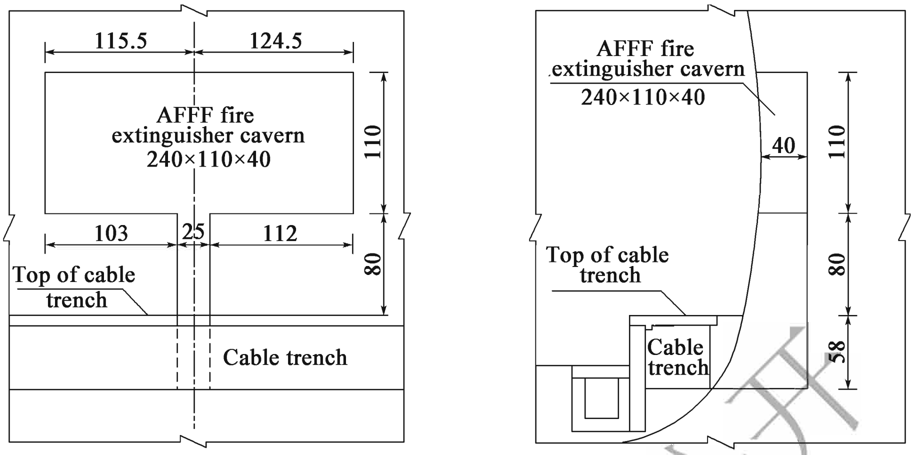


Fig. 18-2 Structure diagram of AFFF fire extinguisher cavern (unit: cm)

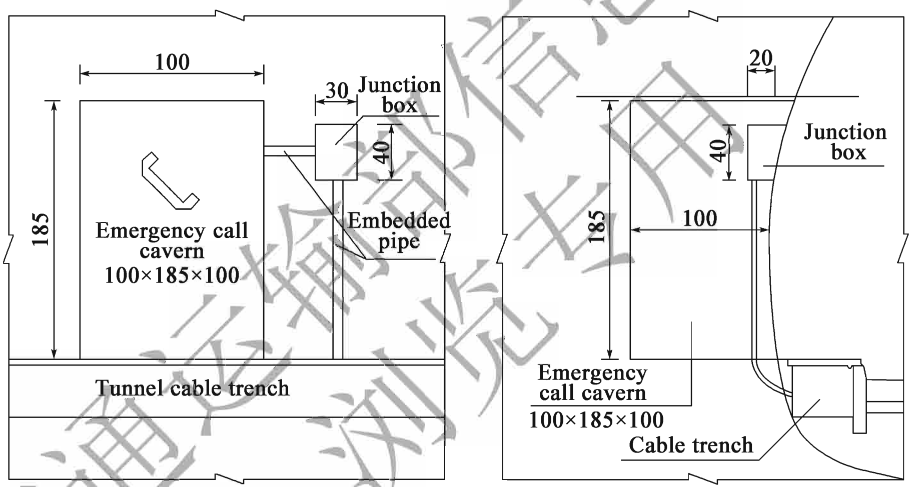


Fig. 18-3 Structure diagram of emergency call cavern (unit: cm)

18.2.3 To ensure the safety of its use and the service life of in-tunnel electrical equipment, it is required that the reserved equipment cavern should not allow water seepage. Therefore, reliable waterproof and drainage measures need to be taken in reserved caverns.

18.2.4 The embedded parts for such facilities as overhead cables and jet fans in tunnel shall not only meet the specified safety factor but also have the specified service life. They shall be protected from corrosion.

2 Embedded parts required to bear loads in tunnel include the embedded parts of overhead cables, signage and markings, camera PTZ, and jet fans.

3 Before installation of fans, the load test on the bearing capacity of embedded parts shall be conducted.

18.2.5 The wall of a pipeline embedded in lining shall not be less than 100mm away from the inner and outer edges of lining. In this sentence, the pipeline means the part of pipeline totally buried in lining and excludes the pipe outlets.

18.2.6 In general, high tension installations need to be placed in a tunnel with a length of more than 300m. Their detailed design or requirements may be presented by the electric discipline and included in civil design.

18.3 Cable trench

18.3.1 Cable trenches are mainly for arrangement of communication cables, electric power cables, fire service pipes, etc. Communication and electrical power cables are usually placed separately in the cable trenches on both sides of the tunnel.

18.3.2 In general, the size of a cable trench for placement of communication cables should not be less than 500mm × 500mm, and that for placement of electric power cables should not be less than 700mm × 600mm. For horizontal or vertical turn transition of cables in tunnel, the size of a cable pipe trench needs to meet the requirements of cable bending radius. The bending radius of cable shall not be less than 1.2m, its turning angle shall not be more than 30° and its turning length shall not be less than 0.6m. The size of a cable trench for laying fire service pipes shall be increased accordingly.

18.3.3

(1) The covers of cable trench in tunnel shall be provided with lifting hooks, holes or slots for opening or installation so as to facilitate the maintenance and management of cable trenches and the pipelines inside.

(2) Covers shall have uniform specifications to facilitate the prefabrication and production. The weight of a cover shall be such that it can be moved by an individual.

(3) The main loads borne by cable trench covers are their dead weight and the weight of pedestrians and small carts.

18.3.4 Generally, brackets are set on the side walls of cable trenches in tunnel. To prevent errant vehicles from crashing and damaging the cables, the outer walls of cable trenches should take into consideration the effect of vehicle impact and be resistant to impact.

18.3.5 Cable trenches shall use the longitudinal slope of tunnel for natural drainage. At the bottom of a cable trench, cross fall and catch basin shall be arranged, and lateral drainage holes shall be set to connect the road side drain.