JTG

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

JTG 2232—2019（EN）

Specifications for Seismic Design of Highway Tunnels

公路隧道抗震设计规范

（英文版）

Issue date: November 26, 2019 Effective date: March 1, 2020

Issued by Ministry of Transport of the People's Republic of China

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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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特此公告。

中华人民共和国交通运输部

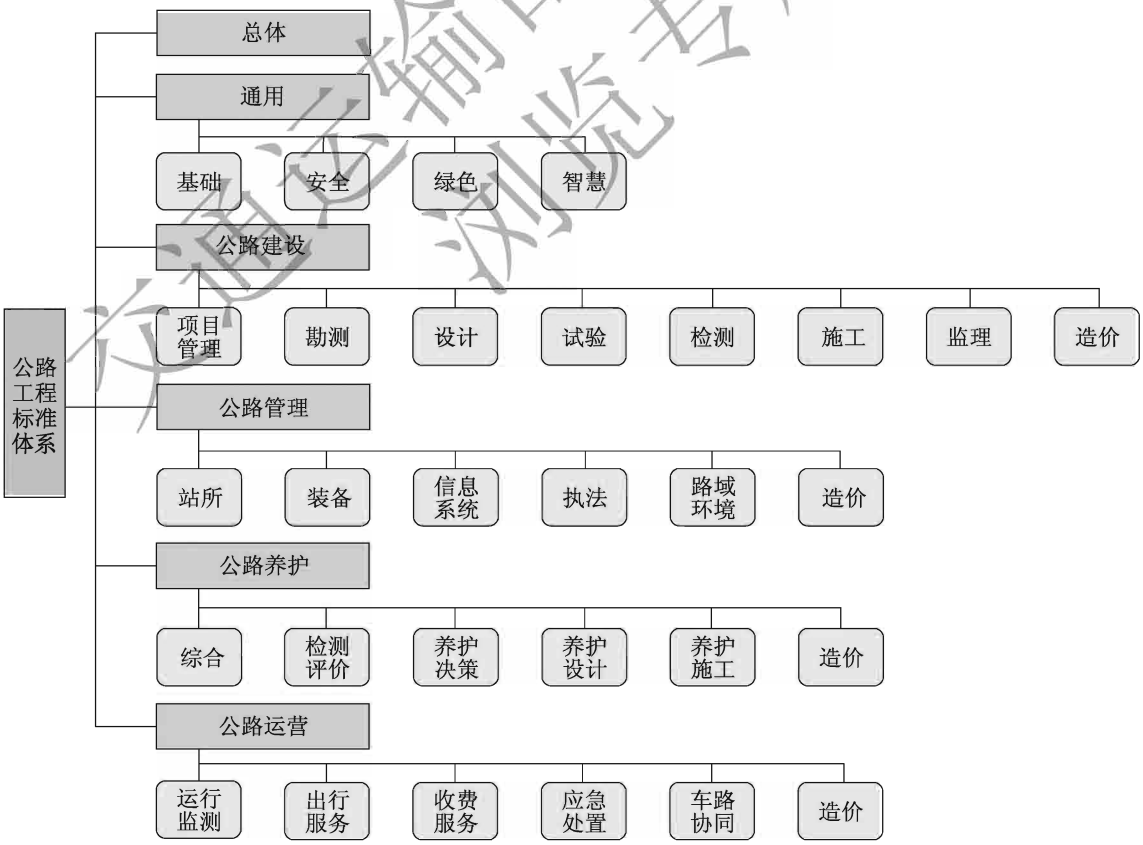
2023 年 9 月 20 日

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。



《公路隧道抗震设计规范》（简称《规范》）是中国交通行业公路隧道抗震设计的重要技术标准，主要用于不同类型公路隧道的抗震设防类别、抗震设防标准、抗震设防目标、抗震计算方法、抗震结构材料等，可供隧道建设企业、设计院、施工企业、工程监理等使用。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，越来越多的公路隧道穿越活动断裂带，隧道抗震设计需求越来越强烈。《规范》以科学合理、经济安全、利用高效为基本原则，在充分总结近年来工程实践经验和科研成果的基础上，综合考虑了中国公路隧道建设现状和隧道抗震技术发展趋势，积极采纳了新理论、新技术、新材料和新方法，并借鉴了国外公路隧道抗震设计的成功经验和先进技术，满足公路隧道抗震设计工作的需要，确定了《公路隧道抗震设计规范》（JTG 2232—2019），并于2019年由交通运输部首次发布实施。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抗震设防提供参考借鉴。

《公路隧道抗震设计规范》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、智利专家Giorgio Piaggio M.、中国专家张乾兵和禹海涛，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。

本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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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

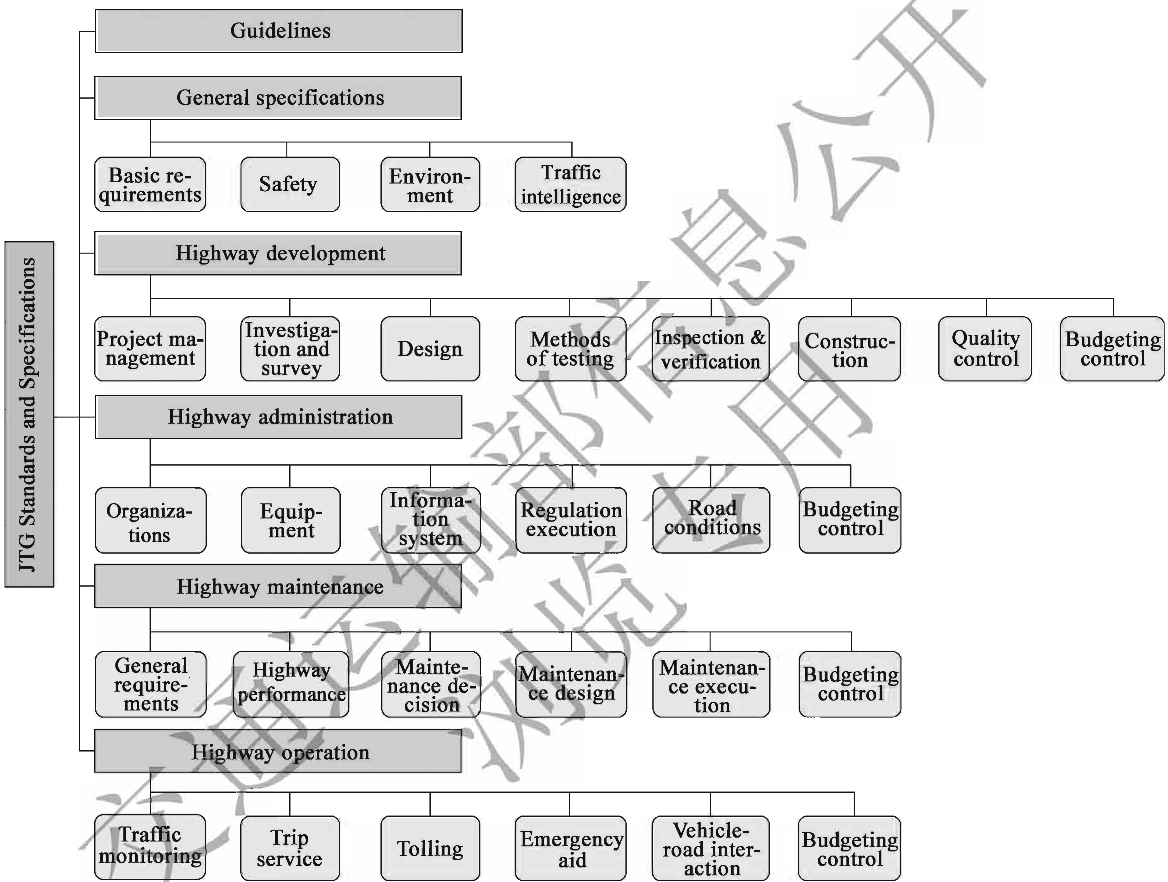
Introduction to English Version

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 Seismic Design of Highway Tunnels* (hereinafter referred to as the Specifications) are important technical standards for the seismic design of highway tunnels in China transportation industry. It is mainly used by tunnel construction enterprises, design institutes, construction enterprises and engineering supervisors for seismic fortification categories, seismic fortification standards, seismic fortification targets, seismic calculation methods and seismic structural materials of different types of highway tunnels. 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%. More and more highway tunnels cross active fault zones, and the demand of seismic design for tunnels is growing. Based on the basic principles of scientific rationality, economic safety and high utilization efficiency, the

Specifications fully summarize the engineering practice experience and scientific research achievements in recent years, comprehensively consider the present situation of highway tunnel construction in China and the development trend of tunnel seismic technology, actively adopt new theories, new technologies, new materials and new methods, refer to foreign successful experience and advanced technology in the seismic design of highway tunnels to meet the requirements for seismic design of highway tunnels, and determine the *Specifications for Seismic Design of Highway Tunnels* (JTG 2232—2019). The Specifications were first issued for implementation by the Ministry of Transport of the People's Republic of China in 2019.



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 a reference for seismic fortification 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 Seismic Design of Highway Tunnels*, 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, Chilean expert Giorgio Piaggio M., Chinese experts Zhang Qianbing and Yu Haitao, 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 this standard is consistent with the current Chinese version. In 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 the 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).

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Foreword to Chinese Version

According to the General Office of Ministry of Transport Notice on Issuing the Order for Highway Engineering Standard Development/Revision Program 2012 (TGLZ [2012] No. 184), China Merchants Chongqing Communications Technology Research & Design Institute Co., Ltd., as the principal drafter, has undertaken the task of developing the *Specifications for Seismic Design of Highway Tunnels* (JTG 2232—2019).

These Specifications consist of 13 chapters and 3 appendices, namely: 1 General Provisions; 2 Glossary and Symbol; 3 Basic Requirements; 4 Tunnel Site, Site and Ground; 5 Earthquake Effect; 6 Calculation Method; 7 Materials and Parameters; 8Seismic check; 9 Drill-and-Blast Tunnels; 10 Shield Tunnels; 11 Immersed Tunnels; 12 Cut-and-cover Tunnels; 13 Tunnel Portals; Appendix A-Static Method; Appendix B-Response Displacement Method; and Appendix C-Time History Analysis Method.

The main contents of these specifications are as follows:

(1) Identify the conditions and scope which the Specifications for Seismic Design of Highway Tunnels apply to.

(2) Establish seismic fortification category, standard and objectives for highway tunnels; adopt three types of seismic design methods, including two-level seismic fortification, two-stage design, and combined strength and deformation control.

(3) Detail the site selection for highway tunnels and provide the relationship between surrounding rock conditions and seismic category of the tunnel.

(4) Provide horizontal design seismic peak ground acceleration, characteristic period of response spectra, seismic peak displacement, and vertical design peak ground acceleration for various sites and the design method of ground motion time histories.

(5) Provide three methods suitable for seismic calculation of highway tunnels, i. e. modified static method, response displacement method and time history analysis

method.

(6) Define the requirements for material selection and properties of an earthquake resistant structure: the method of selecting physical and mechanical properties parameters for main construction and geotechnical materials.

(7) Establish a seismic check method controlled by both strength and deformation criteria.

(8) Detail the content of seismic design for drill-and-blast, shield, immersed, cut-and-cover tunnels and tunnel portals, including seismic calculation and seismic measures.

Comments and suggestions from users of this publication are welcome and should be addressed to the routine administration team (Attention: Fang Lin, No. 33, Xuefu Avenue, Nanán District, Chongqing 400067; tel.: 023-62653050; fax: 023-62653128; e-mail: fanglin@cmhk.com). The feedback will be considered in next edition.

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Institute of Geophysics, China Earthquake Administration

Sichuan Highway Planning, Survey, Design and Research Institute Ltd.

Southwest Jiaotong University

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Contents

1 General Provisions

1.0.1 The purpose of these Specifications is to provide standard for the seismic design of highway tunnels, enhances the capability to reduce damage to highway tunnels due to earthquake. In addition to this it also enables the highway transportation network to better fulfill its functions and play its role in earthquake relief.

1.0.2 These Specifications are applicable to seismic design of highway tunnels with various classes and types.

1.0.3 Seismic fortification and support categories, standards and objectives shall be based on the importance of highway tunnels and the difficulty of repairs (emergency repair).

1.0.4 For seismic design of highway tunnels that are site-specific, seismic safety evaluation, ground motion parameters, seismic fortification and support intensity established in the approved report shall be adopted. For seismic design of highway tunnels to which no seismic safety evaluation has been performed yet, then ground motion parameters given on the current Seismic Ground Motion Parameters Zonation Map of China (GB 18306) shall be employed.

1.0.5 Seismic design of tunnels in areas with seismic fortification and support intensity above IX or tunnels with special requirements shall be subjected to special study and in accordance with relevant provisions.

1.0.6 Seismic design of highway tunnels shall also comply with applicable current national and industrial standards, in addition to these Specifications herein.

2 Terms and Symbols

2.1 Terms

2.1.1 Seismic ground motion

Vibration of surface and near-surface media caused by an earthquake.

2.1.2 Seismic ground motion parameters

The physical parameters of seismic ground motion characterizing seismic design requirements, including seismic peak ground acceleration, velocity, displacement and characteristic period of response spectra.

2.1.3 Seismic ground motion parameter zonation

The territory is divided into zones with different seismic design requirements according to seismic ground motion parameters.

2.1.4 Seismic fortification and support standards

A measure of seismic design requirements, determined from seismic fortification intensity or design ground motion parameters, seismic fortification categories and performance requirements of highway tunnels.

2.1.5 Earthquake effect

The effect of earthquake on an engineering structure, which is primarily the effect of ground motion, including horizontal and vertical earthquake effects.

2.1.6 E1 earthquake effect

The effect of an earthquake at the engineering site with a short return period, corresponding to target performance level 1.

2.1.7 E2 earthquake effect

The effect of an earthquake at the engineering site with a long return period, corresponding to target performance level 2.

2.1.8 Basic ground motion parameters

The value of ground motion parameters with a return period of 475 years.

2.1.9 Seismic measures

Seismic design contents except seismic effect calculation and resistance calculation, including the details of seismic design.

2.1.10 Seismogenic fault

Faults that generated or are likely to generate destructive earthquakes.

2.1.11 Active fault

Faults that havebeen active since the late Quaternary.

2.2 Symbols

C_i—seismic importance coefficient of tunnels;

A—horizontal basic seismic peak ground acceleration at the surface of Class II site;

v_{se}—average shear wave velocity in a soil strata;

f_{aE}—adjusted ground bearing capacity;

ζ_a—adjustment coefficient of seismic bearing capacity of the ground;

f_a—corrected permissible value of ground bearing capacity;

C_e—reduction coefficient of soil liquefaction effect;

A_h—horizontal design peak ground acceleration at the ground surface;

C_s—site adjustment coefficient of seismic peak ground acceleration;

A_{hII}—horizontal design seismic peak ground acceleration at the surface of Class II site;

A—horizontal basic seismic peak ground acceleration at the surface of Class II site;

F_u—site adjustment coefficient of seismic peak displacement;

u_{maxII}—horizontal design seismic peak displacement at the surface of Class II site;

A_v—vertical design peak ground acceleration at the ground surface;

A_h—horizontal design peak ground acceleration at the ground surface;

K_v—ratio of vertical peak ground acceleration to horizontal peak ground acceleration at the ground surface;

T_g—characteristic period of site;

T—natural vibration period of structure;

S_{max}—maximum spectrum horizontal acceleration response;

γ—attenuation index of descending section of response spectrum;

ξ—damping ratio of structure;

C_d—damping adjustment coefficient;

S_H—horizontal earthquake effect;

S_V—vertical earthquake effect;

S—earthquake effect;

F_y—load combination value acting on the structure;

f_k—strength of material;

F_r—standard value of effect combination acting on the structure;

S_q—design value of earthquake effect combinations;

g—gravity acceleration;

K_a—seismic active earth pressure coefficient;

K_{psp}—seismic passive earth pressure coefficient;

K_{ca}—active earth pressure coefficient caused by cohesion of soil mass;

K_{cp}—passive earth pressure coefficient caused by cohesion of soil mass;

θ—earthquake angle;

T_S—natural period of stratum;

$(E_A)^{C}_{eq}$—equivalent compression stiffness of shield tunnels;

$(EA)^{T}_{eq}$—equivalent tensile stiffness of shield tunnels;

$(EI)_{eq}$—equivalent bending stiffness of shield tunnels;

K_J—tensile stiffness of bolts in tunnel cross section.

3 Basic Requirements

3.1 Seismic fortification category and standard

3.1.1 Seismic fortification category shall be established according to highway classes and tunnel importance, as shown in Table 3.1.1. For tunnels that are critical to economy or national defense, or conducive to earthquake relief and maintaining an unblocked lifeline, their seismic fortification categories should be appropriately raised.

Table 3.1.1 Seismic fortification category for highway tunnels

Seismic fortification category	Applicable scope
A	Underwater tunnels crossing rivers, lakes, seas and other waters which are technically complicated and difficult to repair
B	1. Expressway and class-1 highway tunnels 2. 3-lane and 4-lane tunnels 3. Twin-arch tunnel, cut-and-cover tunnel and shed tunnel 4. Tunnel ventilation fan (TVF) room
C	1. Class-2 and Class-3 highway tunnels 2. Ventilation inclined shaft, vertical shaft, air shaft and parallel heading
D	1. Class-4 highway tunnels 2. Auxiliary caverns

3.1.2 The seismic performance requirements of tunnel structures shall be divided into the following three levels according to the seismic design targets.

1 Performance Requirements 1: After an earthquake, the lining structure stress is below the elastic limit and in an elastic state; the structure is undamaged and its functionality remains in the pre-

earthquake state.

2 Performance Requirements 2: After an earthquake, the lining structure stress exceeds the elastic limit but remains within the yield strength, and the structure is in a transition from elastic to elastic-plastic regime; the structure is slightly damaged locally and can be back in service without repair or with simple reinforcement.

3 Performance Requirements 3: After an earthquake, the lining structure stress exceeds the yield strength but does not reach the maximum bearing capacity of the structure. The structure is in an elastic-plastic state but remains stable. The structure sustains damage, but there shall be no local or overall collapse. The functionality of the structure can be restored through repair and reinforcement.

3.1.3 A two-level seismic fortification should be adopted for seismic design of Classes A, B and C tunnels while a one-level seismic fortification should be adopted for seismic design of Class D tunnels. The seismic design targets for various classes of tunnels shall satisfy the specifications in Table 3.1.3.

Table 3.1.3 Seismic design targets for various classes of tunnels

Seismic fortification category	Seismic design targets	
	E1 earthquake effect	E2 earthquake effect
A and B	Performance Requirements 1	Performance Requirements 2
C	Performance Requirements 1	Performance Requirements 3
D	Performance Requirements 1	—

3.1.4 Seismic measures for each class of tunnels shall be determined according to Table 3.1.4.

Table 3.1.4 Levels of seismic measures for each class of tunnels

<table>
<tr><td rowspan="3">Seismic fortification category</td><td colspan="6">Basic earthquake intensity</td></tr>
<tr><td>Ⅵ</td><td colspan="2">Ⅶ</td><td colspan="2">Ⅷ</td><td>Ⅸ</td></tr>
<tr><td>0.05g</td><td>0.10g</td><td>0.15g</td><td>0.20g</td><td>0.30g</td><td>0.40g</td></tr>
<tr><td>A</td><td rowspan="2">Level 2</td><td rowspan="2">Level 3</td><td colspan="2">Level 4</td><td colspan="2">Higher level, special study</td></tr>
<tr><td>B</td><td>Level 3</td><td colspan="3">Level 4</td></tr>
<tr><td>C and D</td><td>Level 1</td><td colspan="2">Level 2</td><td colspan="2">Level 3</td><td>Level 4</td></tr>
</table>

3.1.5 Seismic importance coefficient, C_i, of each class of tunnels shall be determined according to Table 3.1.5.

Table 3.1.5 Seismic importance coefficient, C_i, of each class of tunnels

Seismic fortification category	E1 earthquake effect	E2 earthquake effect
A	1.0	1.7(1.3[a])
B	0.43	1.3
C	0.34	1.0
D	0.26	—

Note: [a]1.3 for immersed tunnels only.

3.2 Earthquake effect

3.2.1 The seismic effects to be considered in seismic design of tunnels shall be characterized by the basic ground motion parameters and seismic importance coefficient C_i of the region where tunnels are located. For tunnels that have a site-specific seismic safety evaluation, the peak ground acceleration (PGA) values under earthquake effects of any level shall not be lower than those established using the seismic importance coefficient specified in Article 3.1.5 within these Specifications.

3.2.2 When carrying out site-specific seismic safety evaluation at the tunnel site, in addition to meeting the work content and depth requirements specified in the current Evaluation of Seismic Safety for Engineering Sites (GB17741), the established earthquake effects shall also meet the relevant provisions of these Specifications.

3.2.3 In cases where basic peak ground acceleration is required to determine the corresponding seismic fortification intensity, it shall be converted to basic peak ground acceleration for Class II site, according to the corresponding relationship in Table 3.2.3.

Table 3.2.3 Correspondence between peak ground acceleration and seismic fortification intensity

Seismic peak ground acceleration value (g)	0.05	0.10	0.15	0.20	0.30	0.40
Basic peak ground acceleration for Class II site	[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 fortification intensity	Ⅵ	Ⅶ		Ⅷ		Ⅸ

3.3 Seismic design process

3.3.1 Seismic design of tunnels shall be carried out using the following three categories of methods:

1 Category 1: seismic analysis and seismic check under E1 and E2 earthquake effects shall be performed, and the requirements of seismic measures shall be met.

2 Category 2: seismic analysis and seismic check under E1 earthquake effect shall be performed, and the requirements of seismic measures shall be met.

3 Category 3: the requirements of seismic measures shall be met; seismic analysis and seismic check are not mandatory.

3.3.2 Depending on its seismic fortification category and level, the seismic design method for a tunnel should be selected according to Table 3.3.2.

Table 3.3.2 Seismic design methods for tunnels

Seismic fortification category	Seismic fortification intensity					
	Ⅵ	Ⅶ		Ⅷ		Ⅸ
	0.05g	0.10g	0.15g	0.20g	0.30g	0.40g
A	Category 2	Category 1	Category 1	Category 1	Category 1	Category 1
B	Category 3	Category 3	Category 2	Category 2	Category 1	Category 1
C	Category 3	Category 3	Category 3	Category 2	Category 2	Category 1
D	Category 3	Category 3	Category 3	Category 3	Category 2	Category 2

3.3.3 Seismic design shall follow the following procedures:

1 Determine seismic fortification category of the tunnel in accordance with Article 3.1.1 of these Specifications.

2 Establish seismic design targets and seismic performance requirements for the tunnel in accordance with Article 3.1.3 of these Specifications.

3 Determine seismic design content and method, including seismic analysis, seismic check and seismic measures design, in accordance with the tunnel's seismic fortification category, seismic performance requirements and Article 3.3.2 of these Specifications.

4 Establish earthquake effects E1 and E2 for the tunnel according to the current Seismic Ground Motion Parameters Zonation Map of China (GB 18306) or results from site-specific seismic safety evaluation.

5 The seismic design of the tunnel should be carried out according to the flow chart in Fig. 3.3.3.

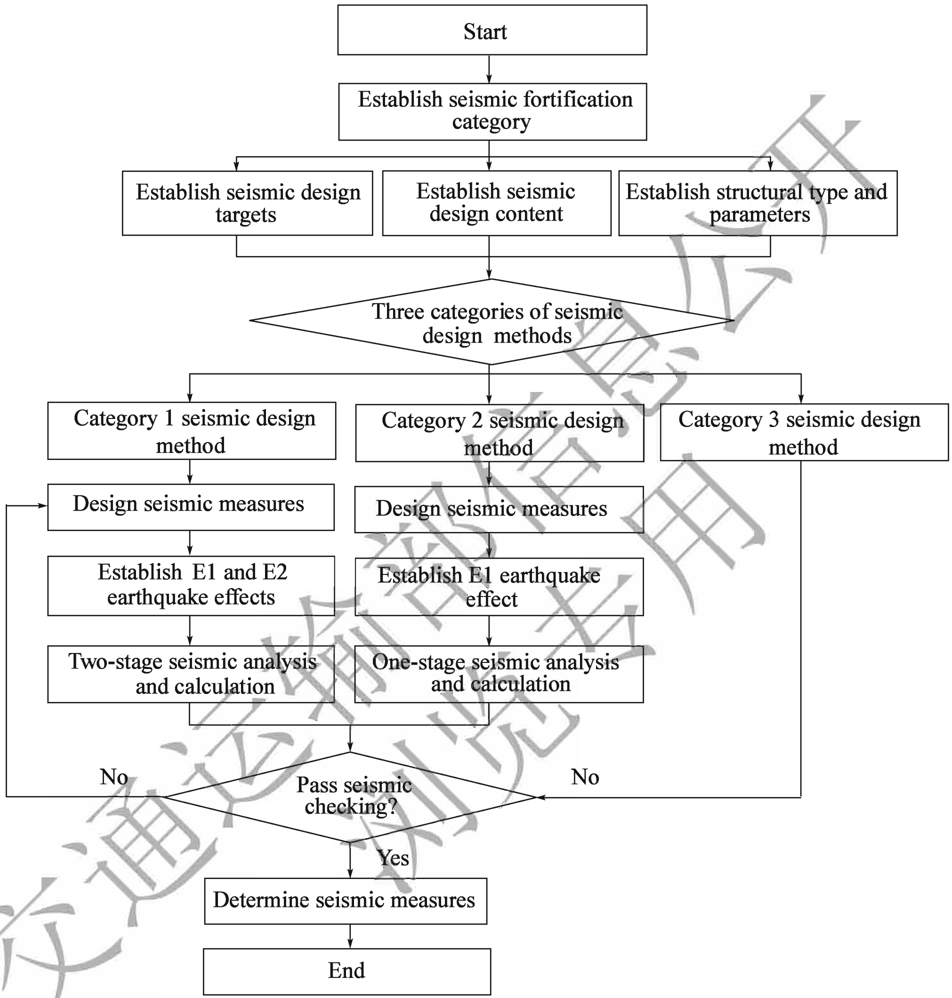


Fig. 3.3.3 Seismic design flowchart for tunnel structures

4 Tunnel Site, Site and Ground

4.1 General requirements

4.1.1 When selecting the site for a tunnel, the following macroseismic damages or earthquake effects shall be considered:

1 Vibratory failure of the tunnel structure caused by severe ground motion;

2 Instability or failure of sites and ground instability caused by severe ground motions, including liquefaction, fissure/cracking, seismic subsidence, landslides and collapses;

3 Fault dislocation, including the failure caused by bedrock fracture and tectonic fissure;

4 Special damages caused by ground motion anomaly due to variation in local topography, landform and stratum structure.

4.1.2 In addition to meeting the requirements of relevant specifications, the tunnel geological investigation shall also include site and ground survey and evaluation on the following contents for seismic design purposes:

1 Site soil type, site category, seismic section category of the site and ground liquefaction evaluation;

2 Location, continuity and activity of active faults and seismogenic faults;

3　Stability of soil/rock concerning potential landslide, cave-in, collapse and goaf in tunnel site area;

4　Stability of tunnels at locations such as fault fracture zones, karst and weak surrounding rocks;

5Soil profile and soil parameters such as dynamic shear modulus and damping ratio, etc.

4.2　Tunnel site and site

4.2.1　The seismic category of tunnel surrounding rock is affected by its depth and surrounding rock conditions, and can be determined according to Table 4.2.1.

Table 4.2.1　Seismic category of rock surrounding the tunnel

Depth	Surrounding rock class					
	Ⅰ	Ⅱ	Ⅲ	Ⅳ	Ⅴ	Ⅵ
Deep embedment	Favorable	Favorable	Favorable	Favorable	General	Unfavorable
Shallow embedment	Favorable	Favorable	Favorable	General	Unfavorable	Hazardous
Portal	Favorable	Favorable	General	Unfavorable	Unfavorable	Hazardous
side and front slope	Favorable	Favorable	General	Unfavorable	Hazardous	Hazardous

4.2.2　During tunnel investigations, the sites at locations of tunnel side and front slope, and tunnel portals shall be classified as favorable, general, unfavorable and hazardous locations for seismic design purposes on the basis of macro-geology and micro-geology conditions. The site locations shall be categorized according to Table 4.2.2.

Table 4.2.2　Categorization of site locations

Location category	Topography, landform and geology
Favorable location	Gentle slope, stable bedrock, hard soil, dense and homogenous medium hard soil, etc.
General location	Other than favorable, unfavorable and hazardous locations
Unfavorable location	Steep slopes, scarps, river bank and slope edges, structural cracks at ground surface, highly weathered rock formation, soft soil, liquefiable soil, soil layers with obvious uneven origin, lithology and state in horizontal distribution (e. g. paleochannels, loose fault fracture zones, underground ponds, streams, ditches, valleys, side cut, hillside section, cut and fill sections), steep strata inclined to outside mountains, plastic loess with high water content, strip-shaped projecting mountain spur and towering isolated hills
Hazardous location	Locations where landslides, collapses, land subsidence, fissure, debris flow, etc. may occur during earthquakes, as well as locations where formation dislocations may occur in seismogenic fault zones

4.2.3 The tunnel site should bypass seismically unfavorable and hazardous locations. If this is difficult to achieve, the tunnel alignment shall pass through seismically unfavorable and hazardous locations by the shortest distance. Specific research shall be conducted if it is necessary to pass a hazardous location with seismic fortification intensity of Ⅷ and above.

4.2.4 The shear wave velocity in rock and soil at the tunnel engineering site shall be determined according to the following provisions:

1 For Class A tunnels, the shear wave velocity in the soil layer at the engineering site shall be determined through field measurement.

2 For Classes B, C and D tunnels, if no measured shear wave velocity is available, the shear wave velocity in each rock and soil layer may be estimated according to Table 4.2.4 on the basis of rock and soil description and characteristics and local experience.

Table 4.2.4 Classification of soil/rock types and shear wave velocity range

Soil/rock type	Soil/rock description and properties	Shear wave velocity range (m/s)
Rock	Hard, relatively hard and intact stable rock	$v_{se}>800$
Hard soil or soft rock	Broken and relatively broken rocks or soft and relatively soft rocks; compact gravelly soil	$800\geqslant v_{se}>500$
Medium hard soil	Moderately compact and slightly compact gravelly soils; compact and moderately compact gravels; coarse (medium) sand; clayey material and silt with $f_{ak}>200$kPa; hard loess	$500\geqslant v_{se}>250$
Medium soft soil	Slightly compact gravel and coarse (medium) sand; fine and silty sand except loose sand; clayey material and silt with $f_{ak}\leqslant200$kPa; fill with $f_{ak}>140$kPa; plastic loess	$250\geqslant v_{se}>150$
Soft soil	Hydric soil and mucky soil; loose sand; recently deposited clayey material and silt; fill with $f_{ak}\leqslant140$kPa; flow-plastic loess	$v_{se}\leqslant150$

4.2.5 Site overburden depth at tunnel portals, shallow tunnels, cut-and-cover tunnels and immersed tunnels shall be determined according to the following provisions:

1 In general, it shall be determined as the distance from the ground surface to the top surface of soil layer where the shear wave velocity is greater than 500m/s and the shear wave velocity of each underlying layers of rock and soil is not less than 500 m/s.

2 If there is a soil layer at greater depths than 5m below the ground surface with a shear wave velocity that is 2.5 times higher than each of its overlying soil layers, and the shear wave velocity

in this layer and any of its underlying layers is not less than 400m/s, it may be determined as the distance from the ground surface to the top surface of this soil layer.

3 Boulder and lenses with shear wave velocity greater than 500m/s shall be regarded as surrounding soil layers.

4 The hard volcanic interlayer in the soil layer shall be regarded as a rigid body, and its thickness shall be deducted from the soil cover.

4.2.6 The average shear wave velocity in the soil layer shall be calculated as follows:

$$v_{se} = \frac{d_0}{t} \tag{4.2.6-1}$$

$$t = \sum_{i=1}^{n} \left(\frac{d_i}{v_{si}} \right) \tag{4.2.6-2}$$

where v_{se}——average shear wave velocity of soil layer (m/s);

d_0——calculated depth (m), taken as the thickness of the overburden or 20m, whichever is less;

t——the propagation time (s) of shear wave from ground surface to the calculated depth;

d_i——the thickness of the i^{th} soil layer within calculated depth (m);

v_{si}——shear wave velocity (m/s) of the i^{th} soil layer within the calculated depth;

n——the number of soil layers within the calculated depth.

4.2.7 Tunnel portals, shallow tunnels, cut-and-cover tunnels, shield tunnels and immersed tunnels sites shall be grouped into four classes according to the shear wave velocity in rocks or the average shear wave velocity in soil layer and the thickness of site overburden, and shall conform to the provisions in Table 4.2.7. If reliable shear wave velocity and overburden thickness are available and their values are near the boundary of the site categories listed in Table 4.2.7, the characteristic period for seismic effect calculation shall be determined by the interpolation method.

Table 4.2.7 Classification of tunnel sites

Shear wave velocity in rock or average shear wave velocity in soil (m/s)	Site category				
	Ⅰ		Ⅱ	Ⅲ	Ⅳ
	I_0	I_1			
$v_{se} > 800$	0	—	—	—	—
$800 \geqslant v_{se} > 500$	—	0	—	—	—
$500 \geqslant v_{se} > 250$	—	<5	≥5	—	—
$250 \geqslant v_{se} > 150$	—	<3	3~50	>50	—
$v_{se} \leqslant 50$	—	<3	3~15	15-80	>80

Note: Data in the table are the thickness (m) of the siteoverburden.

4.2.8 If there is a seismogenic fault within the tunnel engineering site, the impact of fault dislocation shall be evaluated and the following requirements shall be met:

1 When one of the following conditions is satisfied, the influence of fault dislocation on the tunnel may be neglected:

1) Areas where the seismic fortification ground motion classification is less than 0.20g;

2) Non-Holocene faults;

3) For shield, immersed and cut-and-cover tunnels, the soil overburden thickness above Holocene bedrock buried fault is greater than 60m and 90m respectively in areas with seismic fortification ground motion classification of 0.20g(0.30g) and 0.40g.

2 If the above conditions cannot be satisfied, the following measures should be incorporated:

1) The axial direction of the tunnel should not be parallel to the main fracture at close distance;

2) In cases where the tunnel has to pass through an active fault zone, it may be located at the narrow part of the fault zone, and the dislocation velocity and dislocation of the fault zone shall be subjected to special evaluation, and corresponding measures shall be incorporated into the tunnel design;

3) Where the tunnel layout is parallel to an active fracture, it should be arranged on the heading side of the fault zone.

4.3 Ground

4.3.1 When checking the seismic design of natural foundation, its seismic bearing capacity shall be calculated using Equation (4.3.1):

$$f_{aE} = \zeta_a f_a \tag{4.3.1}$$

where f_{aE}——adjusted ground bearing capacity;

ζ_a——adjustment coefficient of seismic bearing capacity of the ground, which shall be obtained from Table 4.3.1;

f_a——corrected characteristic value of ground bearing capacity, which shall be obtained from the current Specifications for Design of Foundation of Highway Bridges and Culverts (JTGD63).

Table 4.3.1 Adjustment coefficient of seismic bearing capacity of the ground

Soil/rock description and properties	ζ_a
Rock, compact gravelly soil, compact gravel, coarse and medium sand; clayey material and silt with $f_{ak} \geqslant$ 300kPa	1.5
Moderately compact and slightly compact gravelly soil; moderately compact gravel, coarse and medium sand; compact and moderately compact fine and silty sand; clayey material and silt with 300kPa $> f_{ak} \geqslant$ 150kPa; hard loess	1.3
Slightly compact fine and silty sand; clayey material and silt with 150kPa $> f_{ak} \geqslant$ 100kPa; plastic loess	1.1
Hydric soil, mucky soil, loose sand, miscellaneous fill, newly accumulated loess and flow-plastic loess	1.0

4.3.2 In checking the vertical bearing capacity of natural ground under earthquake effects, the compressive stress at the bottom of the foundation and maximum stress at the foundation edge calculated according to earthquake effects combination shall meet the following requirements:

$$p \leqslant f_{aE} \tag{4.3.2-1}$$

$$p_{max} \leqslant 1.2 f_{aE} \tag{4.3.2-2}$$

where p——the average compressive stress at the bottom of the foundation under seismic effect combination;

p_{max}——maximum stress at the foundation edge under seismic effect combination.

4.4 Soil liquefaction and soft ground

4.4.1 Tunnel structures sensitive to liquefaction-induced ground subsidence in areas with seismic fortification ground motion classification of 0.05g should be subject to site liquefaction evaluation and treatment according to the requirements for seismic fortification ground motion classification of 0.10g. For areas with seismic fortification ground motion classification of 0.10g or higher, Class A structures shall subjected to site-specific liquefaction investigation and treatment, and Class B and C tunnel structures may be subject to site liquefaction evaluation according to the required seismic fortification intensity in the areas.

4.4.2 Ground with saturated loose sand and saturated silt shall be subjected to liquefaction, except for areas where seismic fortification ground motion classification is 0.05g. The liquefaction potential of saturated sandy soil and silt with high gravel content, saturated silty sand and silty interbedded soil and sandy soil should be subjected to special study.

4.4.3 For pile foundations deeper than 15m that is below the ground surface and from the natural ground, or at more than 5m depths where saturated sandy soil or saturated silt (excluding loess) exists within 20m below the ground surface, the liquefaction potential may be ruled out or the impact of liquefaction may be neglected if one of the following conditions fulfils:

1 In areas where geologic age is Late Pleistocene of Quaternary (Q_3) and earlier, and peak ground acceleration is less than 0.40g, liquefaction potential may be ruled out.

2 In areas where peak ground acceleration is 0.10g (0.15g), 0.20g (0.30g) and 0.40g and the fines content of silt (particles less than 0.005mm in size) is not less than 10%, 13% and 16% respectively, the liquefaction potential may be precluded.

Note: The fines content used for liquefaction evaluation is determined using sodium hexametaphosphate as dispersant. Conversion shall be made according to relevant specifications if other methods are used.

3 For structures embedded in shallow natural ground, if the thickness of the overlying non-liquefiable soil layer and the depth to groundwater table meet one of the following conditions, the impact of liquefaction may be neglected:

$$d_u > d_0 + d_b - 2 \quad (4.4.3\text{-}1)$$

$$d_w > d_0 + d_b - 3 \quad (4.4.3\text{-}2)$$

$$d_u + d_w > 1.5d_0 + 2d_b - 4.5 \quad (4.4.3\text{-}3)$$

where d_u——thickness (m) of overlying non-liquefiable soil layer, from which the thickness of hydric soil and mucky soil layers should be deducted in calculation;

d_0——characteristic depth (m) of liquefiable soil, which may be taken according to Table 4.4.3;

d_w——depth to groundwater table (m), which should be taken as the annual average highest water level in the basic design life or the recent annual highest water level;

d_b——foundation depth below ground surface (m), which shall be taken as 2m if it is less than 2m.

Table 4.4.3 Characteristic depth d_0 (m) of liquefiable soil

Type of saturated soil	Seismic peak ground acceleration value		
	0.10g(0.15g)	0.20g(0.30g)	0.40g
Silt	6	7	8
Sandy soil	7	8	9

4.4.4 When further liquefaction evaluation is required, the standard penetration test (SPT) shall be used for liquefaction evaluation of soil within 15m depth below the ground surface. If pile foundation or foundation buried at depths greater than 5m is adopted, liquefaction evaluation of soil within 15 ~ 20m depth below the ground surface shall also be carried out. If the number of blows in SPT of saturated soil (not corrected for rod length) is less than the critical value of the number of blows in SPT for liquefaction evaluation, N_{cr}, it shall be judged as liquefiable soil. Other evaluation methods, if well established and proven, may also be used. The critical value of number of blows in SPT for liquefaction evaluation shall be calculated as follows:

1 Within 15m depth below ground surface, the critical value of the number of blows in SPT for liquefaction evaluation may be calculated using the following equation:

$$N_{cr} = N_0[0.9 + 0.1(d_s d_w)]\sqrt{3/\rho_c} \quad (4.4.4\text{-}1)$$

2 At 15 ~ 20m depths below ground surface, the critical value of the number of blows in SPT for liquefaction evaluation may be calculated using the following equation:

$$N_{cr} = N_0(2.4\ 0.1d_w)\sqrt{3/\rho_c} \quad (4.4.4\text{-}2)$$

where N_{cr}——adjusted critical value of the number of blows in SPT for liquefaction evaluation;

N_0——the reference value of the number of blows in SPT for liquefaction evaluation, which shall be taken from Table 4.4.4;

d_s——SPT depth in saturated soil (m);

ρ_c——percent of fines content, which shall be taken as 3% when it is less than 3% or in the case of sandy soil.

Table 4.4.4 Reference value N_0 of SPT number of blows for liquefaction evaluation

Characteristic period (S) on Zonation Map	Seismic peak ground acceleration on Zonation Map		
	0.10g(0.15g)	0.20g(0.30g)	0.40g
0.35	6(8)	10(13)	16
0.40 and 0.45	8(10)	12(15)	18

4.4.5 For ground with liquefiable soil layers, the depth and thickness of each liquefiable soil layer shall be verified, the liquefaction index of each borehole shall be calculated according to Equation (4.4.5), and the liquefaction grade of the ground shall be comprehensively determined according to Table 4.4.5.

$$I_{lE} = \sum_{i=1}^{n}\left(1\ \frac{N_i}{N_{cri}}\right)d_i W_i \quad (4.4.5)$$

where I_{lE}——liquefaction index;

n——the total number of standard penetration test points for each borehole within the evaluated depth range;

N_i——measured value of the number of blows in SPT at point i;

N_{cri}——critical value of the number of blows in SPT for liquefaction evaluation at point i; when the measured value is greater than the critical value, the critical value shall be taken;

d_i——soil layer thickness (m) represented by point i, which may be taken as half of the depth difference between the upper and lower standard penetration test points adjacent to the standard penetration test point, provided that the upper bound is not above groundwater table and the lower bound is not below the liquefaction depth;

W_i——the horizon influence weight function value (m^{-1}) of unit thickness of soil layer i. If the evaluated depth is 15m, it shall be taken as 10 when the middle point of this layer is not deeper than 5m, as 0 when the middle point is at 15m depth, and obtained by linear interpolation method when it is at 5 ~ 15m depths. If the evaluated depth is 20m, it shall be taken as 10 when the middle point of this layer is not deeper than 5m, as 0 when the middle point is at 20m depth, and obtained by linear interpolation when it is at 5 ~ 20m depths.

Table 4.4.5 Correspondence between ground liquefaction grade and liquefaction index I_{IE}

Liquefaction grade	Slight	Moderate	Serious
Liquefaction index for 15m evaluated depth	(0,5]	(5,15]	>15
Liquefaction index for 20m evaluated depth	(0,6]	(6,18]	>18

4.4.6 In evaluating the potential for earthquake-induced liquefaction, consideration shall be given to the decrease in effective stress of stratum caused by the presence of tunnel.

4.4.7 If the liquefiable soil layer is relatively flat and uniform, appropriate measures against liquefaction of ground should be selected according to the requirements in Table 4.4.7. Any untreated liquefiable soil layer should not be used as the bearing stratum of natural ground.

Table 4.4.7 Required measures against liquefaction of ground

Seismic fortification category	Soil liquefaction grade		
	Slight	Moderate	Serious
A and B	Partial elimination of liquefaction-induced settlement or treatment of ground and structure	Completely eliminate liquefaction-induced settlement, or partially eliminate liquefaction-induced settlement and treat the ground and structure.	Completely eliminate liquefaction-induced settlement.
C	No measures are required.	Treat the structure and ground, or implement more stringent measures.	Completely eliminate liquefaction-induced settlement, or partially eliminate liquefaction-induced settlement and treat the ground and structure.
D	No measures are required.	No measures are required.	Treat the ground and structure, or take other economic measures.

4.4.8 Measures to eliminate liquefaction-induced ground settlement shall meet the following requirements:

1 The bottom of the foundation for tunnel structures shall be embedded in a stable soil layer below the liquefaction depth, at depths not less than 0.5m. Attention shall be paid to the protection

against buoyancy during earthquake.

2 Where the ground is reinforced by vibro-flotation, vibro-compaction, gravel compaction piles, dynamic compaction and other densification methods, it shall be treated up to the lower bound of liquefaction depth.

3 Replace all liquefiable soil layers with non-liquefiable soil.

4 Where the tunnel structure is in a liquefiable soil layer and treated by densification method or soil replacement method, its treatment width should not be less than the thickness of liquefiable soil layer. If the thickness of the liquefiable soil layer is less than the width of the tunnel bottom surface, all liquefiable soil layers within the width of the tunnel bottom surface shall be treated. If there are soldier piles, continuous walls and other support structures on both sides of the foundation pit of the cut-and-cover tunnel, and the width of the structure is less than the thickness of the liquefiable soil layer, all liquefiable soil layers within the row piles need to be treated only, but the soldier piles shall pass through the liquefiable soil layers.

5 Where the tunnel structure is in a liquefiable soil layer and reinforced by grouting, grouting thickness should not be less than the thickness of the liquefiable soil layer.

6 Embed permanent enclosure into the non-liquefiable soil layer.

4.4.9 To mitigate the impact of soil liquefaction on the foundation and structure, the following measures may be incorporated.

1 Select an appropriate depth of foundation.

2 Adjust the area of foundation bottom to reduce foundation eccentricity.

3 Strengthen the integrity and stiffness of the foundation;

4 Reduce the load, enhance the overall stiffness and uniform symmetry of the structure, and avoid adopting structural types sensitive to differential settlement.

4.4.10 Tunnels should not be located in areas with liquefaction-induced lateral spreading or flow slide potential, e.g. paleochannels and areas adjacent to contemporary riverside, coasts and side slopes. Otherwise, skid resistance checking should be carried out, and measures to resist soil sliding or structural crack resistance measures shall be incorporated.

4.4.11 For the pile foundation of a non-liquefied foundation, the adjustment coefficient of the permissible bearing capacity of the pile foundation can be taken as 1.5 when the seismic calculation is carried out. The adjustment coefficient of the permissible bearing capacity of friction pile can be taken according to the type of foundation soil. When the load test is used to determine the vertical bearing capacity of a single pile, the vertical bearing capacity of a single pile can be increased by 50%, and the horizontal bearing capacity of a single pile can be increased by 25%.

4.4.12 If liquefied soil layers are identified, the following soil parameters shall be revised according to the degree of liquefaction: ground deformation modulus, ground bed coefficient, ground bearing capacity and parameters of bearing capacity of the soil around piles.

4.4.13 The soil parameters of the soil layer judged as liquefied should be corrected by multiplying the reduction coefficient of liquefaction effect C_e of this layer when liquefaction does not occur. The reduction coefficient of liquefaction effect may be obtained from Table 4.4.13. For the soil layer with a reduction coefficient of 0, the resistance effect of the soil layer shall not be included.

Table 4.4.13 Reduction coefficient for soil liquefaction effect, C_e

Liquefaction resistance rate	Soil layer depth (m)	Reduction coefficient C_e
$0.6 \geqslant F_l$	$d_s \leqslant 10$	0
	$10 < d_s \leqslant 20$	1/3
$0.8 \geqslant F_L > 0.6$	$d_s \leqslant 10$	1/3
	$10 < d_s \leqslant 20$	2/3
$1.0 \geqslant F_L > 0.8$	$d_s \leqslant 10$	2/3
	$10 < d_s \leqslant 20$	1

4.4.14 If the number of blows in SPT is used to characterize the liquefaction resistance of soil, the liquefaction resistance rate in Table 4.4.13 may be calculated using Equation (4.4.14):

$$F_L = \frac{N_1}{N_{cr}} \tag{4.4.14}$$

where N_1——measured value of number of blows in SPT for site soil;

N_{cr}——the critical value of number of blows in SPT for liquefaction evaluation;

5 Earthquake Effect

5.1 General requirements

5.1.1 Earthquake effects on various tunnel structures shall be determined according to the following requirements:

1 Horizontal earthquake effect in the direction of cross section of the structure shall be considered.

2 For tunnels in areas with seismic fortification ground motion classification of 0.20g or more and large-span tunnels with four lanes or more in a single tube, and when the earthquake effect caused by vertical motion is significant, both horizontal and vertical earthquake effects along the cross-section plane should be considered.

3 For shield, immersed and cut-and-cover tunnels, both horizontal earthquake effects along the cross-section transverse direction and those in the longitudinal direction along the tunnel axis shall be considered.

5.1.2 Determination of ground motion parameters for seismic design of tunnel structures shall meet the following requirements:

1 For Class A tunnels and extra-long tunnels located in areas with seismic fortification ground motion classification of 0.40g, the parameters shall be determined according to site-specific seismic safety evaluation results.

2 For tunnels that have performed site-specific seismic safety evaluation, the parameters shall be determined according to the evaluation results.

3 For other tunnels, the ground motion parameters may be determined based on the current Seismic Ground Motion Parameters Zonation Map of China (GB 18306) using relevant provisions of these Specifications.

5.1.3 If there are seismogenic faults within the tunnel site area and 10km beyond its periphery, seismic safety evaluation is recommended to be performed for the engineering site. Seismic safety evaluation for the engineering site should be carried out if there are seismogenic faults that might generate an earthquake of magnitude of not less than 6.5 on the Richter scale within the tunnel site area and 5km beyond its periphery.

5.2 Horizontal earthquake effect

5.2.1 When the static method is used for seismic calculation, the horizontal design peak ground acceleration, A_h, at ground surface shall be determined using the equations below:

$$A_h = C_s A_{h\text{II}} \tag{5.2.1-1}$$

$$A_{h\text{II}} = C_i A \tag{5.2.1-2}$$

where C_s——site adjustment coefficient of seismic peak ground acceleration, to be determined by piecewise linear interpolation based on values given in Table 5.2.1; when the ground motion parameters adopted apply to the tunnel site, $C_s = 1.0$;

$A_{h\text{II}}$——horizontal design seismic peak ground acceleration at the surface of Class II site (g);

C_i——seismic importance coefficient, obtained from Table 3.1.5 of these Specifications;

A——horizontal basic seismic peak ground acceleration at the surface of Class II site, obtained from the current Seismic Ground Motion Parameters Zonation Map of China (GB 18306);

Table 5.2.1 Site adjustment coefficient of seismic peak ground acceleration, C_s

Horizontal design seismic peak ground acceleration at the surface of Class Ⅱ site, $A_{h\text{II}}$ (g)	Site category				
	I_0	I_1	Ⅱ	Ⅲ	Ⅳ
≤0.05	0.72	0.80	1.00	1.30	1.25
0.10	0.74	0.82	1.00	1.25	1.20
0.15	0.75	0.83	1.00	1.15	1.10
0.20	0.76	0.85	1.00	1.00	1.00
0.30	0.85	0.95	1.00	1.00	0.95
≥0.40	0.90	1.00	1.00	1.00	0.90

5.2.2 When the response displacement method is used, the horizontal design seismic peak displacement u_{max} at ground surface shall be determined using the equations below:

$$u_{max} = F_u u_{max\,\mathrm{II}} \tag{5.2.2-1}$$

$$u_{max\,\mathrm{II}} = F_{uh\,\mathrm{II}} A_{h\,\mathrm{II}} \tag{5.2.2-2}$$

where F_u——site adjustment coefficient of seismic peak displacement, to be determined by piecewise linear interpolation based on values given in Table 5.2.2;

$u_{max\,\mathrm{II}}$——horizontal design seismic peak displacement at the surface of Class Ⅱ site (m);

$F_{uh\,\mathrm{II}}$——empirical coefficient with unit of s^2; $F_{uh\,\mathrm{II}}$ is taken as 1/15 when peak displacement is in m and peak acceleration is in m/s^2.

Table 5.2.2 Site adjustment coefficient of seismic peak displacement, F_u

Horizontal design seismic peak displacement at the surface of Class Ⅱ site	Site category				
$u_{max\,\mathrm{II}}$ (m)	Ⅰ$_0$	Ⅰ$_1$	Ⅱ	Ⅲ	Ⅳ
≤0.03	0.75	0.75	1.00	1.20	1.45
0.07	0.75	0.75	1.00	1.20	1.50
0.10	0.80	0.80	1.00	1.25	1.55
0.13	0.85	0.85	1.00	1.40	1.70
0.20	0.90	0.90	1.00	1.40	1.70
≥0.27	1.00	1.00	1.00	1.40	1.70

5.2.3 When the generalized response displacement method or time history analysis method is used for seismic calculation, the horizontal acceleration time history used as input shall be the horizontal acceleration time history at the bottom boundary of the calculation model.

5.3 Vertical earthquake effect

5.3.1 The vertical design peak ground acceleration at the site surface, A_v, shall be determined based on the horizontal design peak ground acceleration, A_h, using Equation (5.3.1). In the vicinity of an active fracture, the vertical peak ground acceleration should be taken as horizontal peak ground acceleration.

$$A_V = K_v A_v \tag{5.3.1}$$

where K_v——ratio of vertical peak ground acceleration to horizontal peak ground acceleration, obtained by piecewise linear interpolation from Table 5.3.1.

Table 5.3.1 Ratio of vertical peak ground acceleration to horizontal peak ground acceleration, K_v

Horizontal design peak ground acceleration, A_h (g)	≤0.05	0.10	0.15	0.20	0.30	≥0.40
K_v	0.65	0.70	0.70	0.75	0.85	1.00

5.4 Design ground motion time histories

5.4.1 For tunnel sites that seismic safety evaluation have been conducted, when dynamic structural analysis is carried out by the time history analysis method, the design ground motion time histories shall be established according to the results of site-specific seismic safety evaluation.

5.4.2 For tunnel sites that have not undergone seismic safety evaluation, when the generalized response displacement method and time history analysis method are used for dynamic structural analysis, the design ground motion acceleration time history used as input may be synthesized based on the ground motion acceleration response spectrum, or generated from actual ground motion acceleration records from similar seismic and site environments after appropriate adjustment. The peak acceleration, peak displacement and acceleration response spectrum curve of the ground motion acceleration time history used as input shall deviate less than 5% from the design ground motion peak acceleration, peak displacement specified in these Specifications and the ground motion acceleration response spectrum curve established in this section.

1 The response spectrum (damping ratio 0.05) S (see Fig. 5.4.2) for synthesizing the design acceleration time history is determined by the following equation:

$$S = \begin{cases} S_{max}(5.5T + 0.45) & T < 0.1s \\ S_{max} & 0.1s \leqslant T \leqslant T_g \\ S_{max}(T_g/T)\gamma & T > T_g \end{cases} \tag{5.4.2-1}$$

where T_g——characteristic period of the site (s).

T——natural vibration period of structures (s).

S_{max}——maximum spectrum horizontal acceleration response .

γ——attenuation index of descending section of the curve, typically taken as 1.0 if damping ratio of the structure ξ is 0.05. If damping ratio of the structure ξ is not 0.05, it shall be determined using the equation below: $\gamma = 1.0 + \frac{0.05 - \xi}{0.3 + 6\xi}$。

2 The maximum spectrum horizontal acceleration response, S_{max}, is determined by the following equation:

$$S_{max} = 2.5 C_d A_h \tag{5.4.2-2}$$

where C_d——damping adjustment coefficient, determined from Eq. (5.4.2-3);

A_h——horizontal design seismic peak ground acceleration at ground surface, calculated from Eq. (5.2.1-1) and (5.2.1-2).

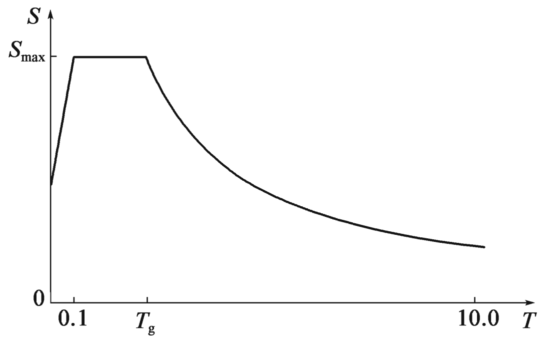


Fig. 5.4.2 Horizontal acceleration response spectrum

3 Unless otherwise specified, the damping ratio ξ of the structure shall be taken as 0.05, and the damping adjustment coefficient C_d in Eq. (5.4.2-3) shall be taken as 1.0. If the damping ratio of the structure is not equal to 0.05 according to relevant provisions, the damping adjustment coefficient C_d shall be determined according to the following equation:

$$C_d = 1 + \frac{0.05\xi}{0.08 + 1.6\xi} \geqslant 0.55 \tag{5.4.2-3}$$

4 The characteristic period T_g shall be determined according to the class of the tunnel site area in Table 5.4.2.

Table 5.4.2 Adjustment for characteristic period of horizontal acceleration response spectrum

Value of characteristic period of the basic peak ground acceleration response spectrum at Class II site (s)	Site category				
	I_0	II_1	II	III	IV
0.35	0.20	0.25	0.35	0.45	0.65
0.40	0.25	0.30	0.40	0.55	0.75
0.45	0.30	0.35	0.45	0.65	0.90

5.4.3 In order to account for the randomness of ground motion, the design acceleration time history shall not be less than 3 sets, and the absolute value of correlation coefficient ρ defined by Equation (5.4.3) between any two sets in the same direction shall be less than 0.1.

$$|\rho| = \left| \frac{\sum_j a_{1j} a_{2j}}{\sqrt{\sum_j a_{1j}^2}\sqrt{\sum_j a_{2j}^2}} \right| \tag{5.4.3}$$

where a_{1y}, a_{2j}——values of time histories a_1 and a_2 at point j.

6 Calculation Method

6.1 General requirements

6.1.1 The seismic calculation method for a tunnel shall be determined taking into consideration factors such as the tunnel construction scale, importance, surrounding environment, topographical and geological conditions, structural type and the type of input ground motion parameters.

6.1.2 Seismic calculation methods for tunnels include static method, response displacement method and time history analysis method. The selection of calculation methods should comply with the provisions in Table 6.1.2.

Table 6.1.2 Seismic calculation methods for tunnels

Method	Structure and ground conditions	Calculation model	Input ground motion
Static method	Cut-and-cover tunnels, shed tunnels and end-walltunnel portals	Transverse	Seismic peak ground acceleration
Modified static method	Simple structural type and homogeneous rock strata	Transverse	Seismic peak ground acceleration
Response displacement method	Simple structural type and homogeneous soil strata	Transverse and longitudinal	Seismic peak displacement
Generalized response displacement method	Simple structural type and complex ground conditions	Transverse and longitudinal	Acceleration time history
Time history analysis	Complex structural type and complex ground conditions	Two-dimensional and 3D	Acceleration time history

Note: For circular shield tunnels in homogeneous soil stratum, the simplified response displacement method given in Appendix B.3 to these Specifications may be used.

6.2 Calculation requirements

6.2.1 The following principles shall be obeyed in the seismic calculation of transverse, longitudinal and three-dimensional (3D) model for a tunnel:

1 For the transverse seismic calculation of a tunnel, a representative tunnel cross section shall be selected according to ground conditions and structural characteristics.

2 For tunnels under complex terrain and geological conditions, such as sections with sharp changes in terrain and geological conditions and tunnels crossing fault fracture zones or at interfaces of soft/hard rock formation, longitudinal seismic calculation is recommended to be carried out. For shield and immersed tunnels, longitudinal seismic calculation shall be carried out.

3 For large-span, important and special tunnel structures, or local sections with sharp changes in terrain and geological conditions, as well as tunnel structures with sharp changes in structural types and significant spatial effect, 3D model calculation should be performed.

6.2.2 When carrying out tunnel seismic calculation, the boundary conditions and earthquake effect of the calculation model shall match the selected method, and the calculation model shall meet the following requirements:

1 When static method or modified static method is used for seismic calculation, the corresponding earthquake effects and boundary conditions may be selected according to Appendix A hereto.

2 When seismic calculation is carried out using the response displacement method, the design earthquake effect base level shall be located at the stratum below the tunnel structure where the shear wave velocity is greater than or equal to 500m/s. The distance from the design earthquake effect base level to the structure shall not be less than 2 times the effective height of the structure for a site with overburden thickness less than 70m. For a site with soil cover thickness of more than 70m, the design earthquake effect base level may be located at 70m depth in the overburden at the site. The corresponding earthquake effect may be selected according to Appendix B hereto.

3 When using time history analysis method for calculation, boundary conditions that can reduce the effect of seismic wave boundary reflection, such as viscous artificial boundary or viscoelastic artificial boundary, should be selected. The calculation scope of the model may be set according to Appendix C hereto.

6.2.3 The selection of element type, the determination of constitutive model and material parameters, and grid size in modeling shall meet the following requirements:

1 If the load-structure model is used, the lining should be simulated by beam elements, and the interaction between surrounding rock and the structure may be simulated by spring or bar elements. If the stratum-structure model is used, the lining may be simulated by beam or shell elements, and the ground may be simulated by plane strain elements or solid elements.

2 The tunnel structure shall be represented by an appropriate constitutive model according to the seismic performance requirements: the elastic constitutive model should be adopted for Performance Requirements 1; elastic-plastic constitutive model should be adopted for Performance Requirements 2 and 3.

3 Static parameters of materials should be used when static method is used for calculation. If time history analysis is used, dynamic parameters of materials should be adopted. The dynamic parameters of materials shall be selected according to the provisions of Section 7.4 herein.

4 The grid size of the calculation model shall be determined considering calculation accuracy and calculation time. The edge length of the cell shall not be greater than 1/10 of the seismic wavelength.

7 Materials and Parameters

7.1 General requirements

7.1.1 The type, specification and performance of tunnel materials shall conform to the provisions of relevant national and industrial standards and shall meet the requirements of seismic design and durability. Meanwhile, their technical economy shall also be considered.

7.1.2 The main tunnel structure in unfavorable geologic zones should be made of reinforced concrete materials, and auxiliary facilities such as tunnel decorations and suspended ceilings should be made of light materials.

7.1.3 The materials for connection between components shall have good integrity, continuity and toughness.

7.2 Material selection

7.2.1 The type and strength grade of mountain tunnel materials shall not be lower than those specified in Table 7.2.1.

Table 7.2.1 Type and strength grade of portal end wall materials for mountain tunnels

Section	Seismic fortification category	Levels of seismic measures		
		Level 2	Level 3	Level 4
Portal end wall	A	Concrete C20	Reinforced concrete C20	Reinforced concrete C20
	B		Concrete C20	
	C			Concrete C20
	D	Rubble concrete C15	Concrete C15	
Retaining wall or wing wall at the portal	$H \leqslant 10m$	Concrete C15	Concrete C20	Concrete C20
	$H > 10m$	Concrete C20		Reinforced concrete C25

Note: H in the table refers to the height of retaining wall or wing wall.

7.2.2 The material and strength grade of linings for seismic design section and for cut-and-cover section of mountain tunnels shall not be lower than those specified in Tables 7.2.2-1 to 7.2.2-3.

Table 7.2.2-1 Type and strength grade of materials for two-lane mountain tunnel linings

Seismic fortification category	Surrounding rock class	Levels of seismic measures		
		Level 2	Level 3	Level 4
A	Ⅲ	Reinforced concrete C30	Reinforced concrete C30	Reinforced concrete C35
	Ⅳ			
	Ⅴ, Ⅵ			
B	Ⅲ	Concrete C25	Concrete C30	Reinforced concrete C30
	Ⅳ		Reinforced concrete C30	
	Ⅴ, Ⅵ	Reinforced concrete C25		
C	Ⅲ	Concrete C25	Concrete C30	Concrete C30
	Ⅳ			Reinforced concrete C30
	Ⅴ, Ⅴ	Reinforced concrete C25	Reinforced concrete C30	
D	Ⅲ	Concrete C25		Concrete C25
				Reinforced concrete C25
	Ⅳ			
	Ⅴ, Ⅵ	Reinforced concrete C25		

Table 7.2.2-2 Type and strength grade of lining materials for mountain tunnels with 3 or more lanes

Seismic fortification category	Surrounding rock class	Levels of seismic measures		
		Level 2	Level 3	Level 4
A	Ⅲ	Reinforced concrete C30	Reinforced concrete C35	Reinforced concrete C40
	Ⅳ			
	Ⅴ, Ⅵ			

continued

Seismic fortification category	Surrounding rock class	Levels of seismic measures		
		Level 2	Level 3	Level 4
B	Ⅲ	Concrete C25	Concrete C30	Reinforced concrete C35
	Ⅳ	Reinforced concrete C25	Reinforced concrete C30	
	Ⅴ, Ⅵ			

Note: 1. Reinforced concrete shall be used for all shallow tunnels.

2. Fiber materials should be added to lining materials for three-and over-three-lane tunnels of seismic measures level 4.

Table 7.2.2-3 Type and strength grade of lining materials for cut-and-cover sections of mountain tunnels

Item		Levels of seismic measures		
		Level 2	Level 3	Level 4
Arched cut-and-cover tunnel	Arch ring	Reinforced concrete C25	Reinforced concrete C30	Reinforced concrete C35
	Exterior wall of single-side pressure cut-and-cover tunnel	Concrete C20	Concrete C30	
Shed tunnel	Top beam	Reinforced concrete C25		
	Outside support structure	Concrete C25	Reinforced concrete C25	Reinforced concrete C30
	Interiorrockbolt type side wall			
	Load retaining side wall	Concrete C20		Reinforced concrete C25

7.2.3 The strength grade of shield tunnel segment concrete shall not be lower than C50, the strength grade of immersed tunnel element concrete shall not be lower than C35, and the strength grade of drill-and-blast tunnel lining concrete should not exceed C60.

7.2.4 The materials for in-tunnel auxiliary facilities e.g. ventilation, civil works, lighting, power supply and distribution and traffic works shall meet the following requirements:

1 The strength grade of small hollow concrete blocks shall not be lower than MU10, and the strength grade of masonry mortar shall not be lower than Mb7.5.

2 The strength grade of concrete shall be not lower than C30 for frame beam, column and node core area of frame-support beam and frame-supported column, and not lower than C25 for constructional column, core column and other various components.

7.3 Material properties

7.3.1 The properties of steel bar and steel material shall meet the following requirements:

1 Steel bars with good ductility, toughness and weldability should be preferred for use in reinforced concrete. Load-bearing steel bars selected should be deformed bars not less than HRB400 and conforming to seismic performance criteria. Stirrups selected should be hot-rolled bars not less than HPB300 and conforming to seismic performance criteria.

2 The steel used in steel structure should be Q235 B, C and D carbon steel structure or Q345 B, C, D and E low-alloy high-strength structural steel. If justified, other steel types and grades may also be used.

3 The ratio of the measured tensile strength to the measured yield strength of load-bearing bars in reinforced concrete structures shall not be less than 1.25. The ratio of measured yield strength to the standard strength shall not be greater than 1.3, and the measured total elongation under maximum tensile force shall not be less than 9%.

4 The steel in steel structures shall have a ratio of measured yield strength to measured tensile strength not exceeding 0.85; the steel shall have clear yield steps and an elongation of not less than 20%, with good weldability and qualified impact toughness.

7.3.2 The properties of fiber reinforced concrete (FRC) shall meet the following requirements:

1 The steel fiber volume ratio for steel fiber reinforced concrete shall be determined according to the design requirements and shall not be less than 0.35%. The deformed steel fiber volume ratio for high strength (tensile strength not less than 1000N/mm 8 1) shall not be less than 0.25%.

2 The size of coarse aggregate used for steel fiber reinforced concrete shall not be more than 20mm nor greater than 2/3 of the length of steel fiber. Coarse aggregates greater than 20mm in particle size may be used only after appropriate fibers are selected for use and subjected to special tests and inspections that demonstrate compliance with reinforcement and toughening criteria required by design.

3 Stainless steel fiber or synthetic fiber shall be selected and used for structures with requirements for resistance to corrosion.

4 The design value of tensile strength of ordinary carbon steel fiber materials shall not be less than 380MPa.

7.4 Physical and mechanical parameters of materials

7.4.1 When the static method is used for seismic calculation of the structure, the corresponding static indexes are selected for the mechanical properties of the rock, soil and structural materials.

7.4.2 When the generalized response displacement method and time history analysis method are used for seismic check of the structure, dynamic mechanical indices should be used as the mechanical property criteria for rock and soil mass and structural materials.

7.4.3 Static mechanical indices may be used as dynamic mechanical indices for steel. In cases where no special tests are performed to determine the dynamic properties of concrete materials for tunnel structure, the dynamic modulus of elasticity of concrete may be increased by 30% from its static value, and the dynamic strength value of concrete may be increased by 20% from its static value.

7.4.4 The dynamic mechanical parameters of the rock and soil mass at the engineering site shall be determined through special tests for areas with seismic fortification ground motion classification of 0.40g or above.

7.4.5 For areas with seismic fortification ground motion classification less than 0.40g, the dynamic mechanical parameters of rock and soil mass should be determined through tests or selected according to the literature under similar conditions. Alternatively, static mechanical indices may be used.

8 Seismic Check

8.1 General requirements

8.1.1 Appropriate criteria and method for seismic performance checking shall be adopted according to the structural characteristics of drill-and-blast tunnels, shield tunnels, immersed tunnels and cut-and-cover tunnels.

8.1.2 Strength checking, deformation checking or stability checking shall be carried out respectively according to the seismic performance requirements, and shall meet the following requirements:

1 For Performance Requirements 1, strength under E1 earthquake effect shall be checked.

2 For Performance Requirements 2 and 3, strength and deformation under E2 earthquake effect shall be checked.

3 Strength checking may be performed using comprehensive safety factor method or partial safety factor method. Strength checking for Performance Requirements 3 should be performed using partial safety factor method.

4 Base stress, ground bearing capacity, and stability against sliding and overturning shall also be checked for portal and cut-and-cover structures. E2 earthquake effect shall be used in checking stability against sliding and overturning for Class A, Class B and Class C tunnels, whereas E1 earthquake effect shall be used in checking stability against sliding and overturning for Class D tunnels.

8.1.3 The following effects shall be considered in seismic check for highway tunnels:

1 Permanent action, including structure and component gravity (dead load), earth pressure and water pressure;

2 Variable action, including the effects of vehicles and people passing through the tunnel and dynamic effects caused by fans and other equipment;

3 Earthquake effect, including earthquake inertia, earthquake-induced earth pressure and ground liquefaction effect imposed on the structure.

8.1.4 Earthquake effect combinations shall include the most unfavorable combination of various effect standard values, and the combination coefficient shall be taken as 1.0.

8.1.5 When considering both horizontal and vertical earthquake effects, the horizontal earthquake effect S_H and vertical earthquake effect S_V may be calculated separately, and the total earthquake effect S shall be calculated using the following equation.

$$S = \sqrt{S_H^2 + S_V^2} \tag{8.1.5}$$

8.2 Strength checking

8.2.1 When the comprehensive safety factor method is used for strength checking, the structural strength shall conform to the provisions of Eq. 8.2.1, and the comprehensive safety factors for Performance Requirements 1, 2 and 3 shall conform to the provisions of Tables 8.2.1-1, 8.2.1-2 and 8.2.1-3 respectively.

$$KS(F_y, \alpha_d) \leq R(f, \alpha_d, C) \tag{8.2.1}$$

where $S(\,)$——the action effect function related to the loads acting on the structure;

$R(\,)$——structure resistance effect function related to structural strength of material and geometric dimensions of components;

F_y——value of load combinations acting on the structure;

f_k——strength of material;

α_d——the geometric parameter value of the structure;

C——constraint limit value of the structure;

K——comprehensive safety factor.

Table 8.2.1-1 Comprehensive safety factor of the structure for Performance Requirements 1

Stress characteristics	Material type	
	Reinforced concrete	Concrete
Concrete reaches compressive strength limit value	—	1.8
Concrete reaches shear or tensile strength limit value	—	2.5
Steel bar reaches design strength or the concrete reaches the compressive strength limitvalue	1.5	—
Concrete reaches shear or tensile strength (major tensile stress) limit value	1.8	—

Table 8.2.1-2 Comprehensive safety factor of the structure for Performance Requirements 2

Stress characteristics	Material type	
	Reinforced concrete	Concrete
Concrete reaches compressive strength limit value	—	1.5
Concrete reaches shear or tensile strength limit value	—	2.0
Steel bar reaches design strength or the concrete reaches the compressive strength limit value	1.3	—
Concrete reaches shear or tensile strength (major tensile stress) limit value	1.5	—

Table 8.2.1-3 Comprehensive safety factor of the structure for Performance Requirements 3

Stress characteristics	Material type	
	Reinforced concrete	Concrete
Concrete reaches compressive strength limit	—	1.2
Concrete reaches shear or tensile strength limit	—	1.5
Steel bar reaches design strength or the concrete reaches the compressive strength limit	1.0	—
Concrete reaches shear or tensile strength (major tensile stress) limit	1.2	—

8.2.2 When checking the strength by partial safety factor method, the structural strength shall conform to Eq. (8.2.2).

$$\gamma_0 \gamma_1 S(\gamma_m F_{r,} \alpha_k) \leqslant R \frac{f}{\gamma_f}, \alpha_k, C \tag{8.2.2}$$

where $S(\ )$——the action effect function related to the effects acting on the structure;

$R(\ )$——structure resistance effect function related to structural strength of material and geometric dimensions;

F_r——standard value of effect combinations acting on the structure;

f_k——strength of material;

α_k——the geometric parameter standard value of the structure;

C——constraint limit value of the structure;

γ_0——coefficient of component operating conditions, taken as 1.0;

γ_1——additional safety factor of the structure, taken as 1.0;

γ_m——the partial coefficient for effect on the structure, which shall be determined in accordance with Articles 8.1.3 and 8.1.4 herein;

γ_f——the partial coefficient for material properties. The partial coefficients of materials in Performance Requirements 1, 2 and 3 shall be as specified in Table 8.2.2.

Table 8.2.2 Material partial coefficient γ_f

Material type	Strength type	Symbol	Partial coefficient of standard strength value for Performance Requirements 1	Partial coefficient of ultimate strength for Performance Requirements 2	Partial coefficient of ultimate strength for Performance Requirements 3
Concrete	Compressive strength	γ_{hy}	1.40[a]	1.20	1.0
	Shear strength	γ_{hj}	1.40[a]	1.25	1.1
	Tensile strength	γ_{hl}	2.15[a]	1.75	1.3
Reinforced concrete	Compressive strength of concrete	γ_{hy}	1.35	1.20	1.0
	Shear strength of concrete	γ_{hj}	1.35	1.20	1.1
	Tensile strength of concrete	γ_{hl}	1.50	1.35	1.2
	Compressive strength of steel bar	γ_{gv}	1.25	1.15	1.0
	Tensile strength of steel bar	γ_{gl}	1.25	1.15	1.0
Steel structure	Compressive strength	γ_{gv}	1.25	1.15	1.0
	Tensile strength	γ_{gl}	1.25	1.15	1.0

Note: [a] Under Performance Requirements 1, the partial coefficient of concrete strength is the partial coefficient of its strength limit value.

8.3 Deformation checking

8.3.1 Deformation checking under earthquake effect combinations shall conform to Eq. (8.3.1).

$$S_q \leqslant C \tag{8.3.1}$$

where S_q——design effect (e.g. deformation and displacement) value of earthquake effect combinations;

C——design limits for deformation, displacement, etc.

8.3.2 For seismic Performance Requirements 2, the overall deformation performance of the structure should be checked and meet the following requirements:

1 For rectangular cross-section structures, the interlayer displacement angle shall be used as the criterion. The limit value of interlayer displacement angle for reinforced concrete structures should be 1/250.

2 The maximum convergence value should be used as criterion for the secondary lining structure of the drill-and-blast tunnel (or quasi-circular tunnel), and the limit value should be 5.0‰ of the tunnel span.

3 The diameter deformation ratio should be used as the criterion for circular structures, and the limit value of the diameter deformation ratio of shield tunnels should be 6.0‰.

8.3.3 For seismic Performance Requirements 3, the overall deformation performance of the structure shall be checked. Its corresponding parameters and calculation model shall be compatible with the calculation requirements in the elastic-plastic stage and meet the following requirements:

1 For rectangular cross-section structures, the interlayer displacement angle shall be used as the criterion. The limit value of interlayer displacement angle for reinforced concrete structures should be 1/80.

2 The maximum convergence value should be used as criterion for the lining structure of the drill-and-blast tunnel (or quasi-circular tunnel), and the limit value should be 15‰ of the tunnel span.

3 The diameter deformation ratio should be used as the criterion for circular section structures, and the limit value of the diameter deformation ratio for shield tunnels should be 18.0‰.

8.3.4 Immersed and shield tunnels shall be checked for longitudinal deformation or displacement and shall meet the following requirements:

1 The amount of opening or displacement at the joint shall not exceed the design permissible value.

2 The displacement of axial steel bar (bolt) at expansion joints shall be less than the yield displacement, and the deflection angle at expansion joints shall be less than the yield deflection angle.

8.4 Seismic check for portal wall and retaining wall

8.4.1 In seismic design of tunnel portals, the wall sectional strength, eccentricity, base stress and stability against sliding and overturning shall be checked respectively, and shall meet the following requirements:

1 Earthquake effect is only combined with wall gravity and water and earth pressure.

2 Wall sectional strength and base stress shall be checked.

3 Coefficient of stability against sliding for portal walls is $K_c \geqslant 1.1$ and the coefficient of stability against overturning is $K_o \geqslant 1.2$.

4 Wall masonry eccentricity is $e \leqslant 0.4h$ (h is wall thickness).

5 The resultant eccentricity of the base shall meet the requirements of Table 8.4.1.

Table 8.4.1 Resultant eccentricity of portal wall base, e

Subsoil	e
Rock, dense gravelly soil, dense gravel, coarse and medium sand; old clayey material; general clayey material with $[\sigma] \geqslant 300$kPa	$\leqslant 2.0\rho$
Moderately dense gravelly soil; dense gravel, coarse and medium sand; old clayey material; general clayey material with 200kPa $\leqslant [\sigma] < 300$kPa	$\leqslant 1.5\rho$
Dense and moderately dense fine sand and silt; general clayey material with 100kPa $\leqslant [\sigma] < 200$kPa	$\leqslant 1.2\rho$
Recently deposited clayey material; soft soil; loose sand and fill; general clayey material with $[\sigma] < 100$kPa	$\leqslant 1.0\rho$

Note: ρ is the core radius of the base section, $\rho = W/A$, W is the resistance moment of section of the base edge, and A is the base area.

8.4.2 Horizontal and vertical seismic loads caused by self-weight of portal wall and the approach retaining wall shall be calculated according to Appendix A.3.2 herein.

8.4.3 Seismically induced active and passive earth pressures shall be calculated according to Appendices A.3.3 and A.3.4 herein.

8.4.4 Seismic bearing capacity of the portal wall foundation shall be checked according to Article 4.3.2 herein.

8.5 Checking of stability against buoyancy

8.5.1 Portions of the tunnel passing through liquefiable ground shall be checked for the tunnel structure's stability against buoyancy using Eq. (8.5.1-1) and (8.5.1-2).

$$\frac{W_s + W_a + F_z}{F_f} \geqslant K_f \tag{8.5.1-1}$$

$$F_f = \gamma_b \gamma_w V \tag{8.5.1-2}$$

where W_s——standard value of self-weight of tunnel structure (kN);

W_a——standard value of effective weight of soil cover above the tunnel (kN);

F_z——standard value of lateral frictional resistance against the ground (kN);

K_f——anti-floating safety factor;

F_f——buoyancy standard value (kN);

γ_b——partial coefficient of buoyancy effect, taken as 1.0;

γ_w——standard unit weight of liquefiable soil mass (kN/m^3);

V——volume of tunnel structure in liquefiable soil layer (m^3).

8.5.2 The frictional resistance of the liquefiable soil layer against the lining structure should be calculated using the reduction coefficient of liquefaction effect determined from the measured liquefaction strength ratio, and the buoyancy shall be calculated based on the unit weight of the liquefiable soil.

8.5.3 The anti-floating safety factor should not be less than 1.15 for Performance Requirements 1, 1.10 for Performance Requirements 2, and 1.05 for Performance Requirements 3.

9 Drill-and-Blast Tunnels

9.1 General requirements

9.1.1 Tunnels shall be designed with minimal length of portions buried at shallow depths or under eccentric load. Portal structures should be designed to avoid a direct connection between a tunnel and a bridge.

9.1.2 Design of twin-arch tunnels should be avoided in areas with seismic fortification ground motion classification of 0.2g or above.

9.1.3 For tunnels located in areas with seismic fortification ground motion classification of 0.4g or above or tunnels passing through an active fault, the intrados dimensions should be increased appropriately.

9.1.4 Where an auxiliary channel is provided in the tunnel, necessary structural measures shall be incorporated to enhance the seismic performance of the structure at the junction of the main tunnel and the auxiliary passage.

9.2 Seismic response calculation

9.2.1 Modified static method, response displacement method and time history analysis method should be used for tunnel seismic response calculation.

9.2.2 If the following conditions are met, the modified static method may be selected:

1 Areas where the seismic fortification ground motion classification is 0.30g or below;

2 Only lateral earthquake effect is considered;

3 The tunnel is located in homogeneous strata and has a simple structural configuration.

9.2.3 If the following conditions are met, the response displacement method may be selected:

1 The seismic response of the tunnel is mainly controlled by the relative displacement of strata.

2 When longitudinal seismic calculation is required, longitudinal response displacement method may be adopted;

3 The tunnel is located in homogeneous strata and has a simple structural configuration.

9.2.4 Time history analysis method should be adopted if one of the following conditions is met:

1 Areas with seismic fortification category A and seismic fortification ground motion classification of 0.2g or above;

2 The stratum-structure interaction effect or its non-linear dynamic characteristics need to be considered;

3 Tunnel geological conditions or structural types are complex.

9.2.5 When time history analysis is used, the seismic response analysis of the tunnel may be considered as a plane strain problem. The seismic response should be analyzed as a spatial problem for locations with marked changes in structural types, at interfaces of the main tunnel and auxiliary channels (such as vertical shafts, inclined shafts, TVF rooms and cross passages) or when the tunnel crosses two distinct geological media.

9.3 Seismic check

9.3.1 Seismic check shall focus on tunnel portals, cut-and-cover and shallow portions at the portal, and tunnel portions in fault fracture zone sections, in soft/hard stratum transition sections and structural type transition section.

Table 9.3.2 Scope of the tunnel to be checked for seismic strength and stability

Items to be checked		Seismic intensity		
		Ⅷ	Ⅷ	Ⅸ
Portal wall and approach retaining wall		No check	Check	Check
Linings in fault fracture zone sections, in soft/hard stratum changing sections and in weak surrounding rock	Class A	Check	Check	Check
	Class B	Check	Check	Check
	Class C	No check	Check	Check
	Class D	No check	No check	Check
Linings of cut-and-cover tunnels and shallow tunnels at the portal	Classes A, B and C	Check	Check	Check
	Class D	No check	Check	Check

9.3.2 The scope of the tunnel to be checked for seismic strength and stability shall conform to Table 9.3.2, as well as the provisions of Articles 3.3.1 and 3.3.2 herein.

9.3.3 For tunnels designed with emergency stop zones, vehicle cross passages and pedestrian cross passages, factors such as embedment depth, surrounding rock and spacing shall be considered to select the most unfavorable portion of the structure for seismic check.

9.4 Lining seismic measures

9.4.1 The seismic design scope for shallow portions of the tunnel at portals shall be determined according to the depth, and the length of the lining structure at depths less than 50m shall be selected.

9.4.2 The seismic design scope for tunnel portions in fault fracture zone sections, soft/hard stratum transition sections and structural type transition sections shall extend towards the better surrounding rock or normal structural section. The seismic design length shall include the lengths extended towards both ends. The length of extension may be determined based on seismic response calculation, but the minimum value shall not be less than that specified in Table 9.4.2.

Table 9.4.2 Minimum length of extension for tunnel seismic design scope (m)

Number of lanes	Surrounding rock class	Seismic peak ground acceleration classification		
		0.10g, 0.15g	0.20g, 0.30g	0.40g
Two lanes	Ⅲ ~ Ⅳ	—	—	10
	Ⅴ ~ Ⅵ	—	5	15

continued

Number of lanes	Surrounding rock class	Seismic peak ground acceleration classification		
		0.10g, 0.15g	0.20g, 0.30g	0.40g
Three lanes	Ⅲ ~ Ⅳ	—	5	15
	Ⅴ-Ⅵ	5	10	20
Four lanes	Ⅲ ~ Ⅳ	5	10	20
	Ⅴ-Ⅵ	10	15	25

9.4.3 Composite lining shall be adopted within the seismic design scope of the tunnel. Properties of construction materials used for secondary lining shall not be lower than the requirements in Tables 7.2.2-1 to 7.2.2-3 herein.

9.4.4 For tunnels running parallel to the strike of an active fault, the distance between the tunnel and the edge of the active fault should be greater than 300 m in the case of seismic measures level 3 and should be greater than 500m in the case of seismic measures level 4.

9.4.5 If unfavorable geologic conditions such as talus, landslide, debris flow, collapse and rockfall that jeopardize the safety of the tunnel, engineering measures shall be taken to prevent the tunnel damages due to secondary earthquake disasters.

9.4.6 The minimum spacing between the construction gauge and the intrados of the seismic design section of the tunnel shall meet the following requirements:

1 When the seismic measures for the tunnel is Level 3, the minimum spacing should be greater than 15cm. When it is Level 4, the minimum spacing should be greater than 25cm.

2 When the tunnel passes through a seismogenic fault, the minimum spacing should be greater than 35cm.

9.4.7 Tunnel linings in seismic design section, in soft/hard stratum transition sections and structural type transition sections shall be provided with seismic joints and shall meet the following requirements:

1 Seismic joints should be considered in conjunction with settlement joints and expansion joints.

2 The longitudinal spacing of seismic joints should be 9 ~ 12m.

3 Seismic joints should be 2 ~ 4cm wide.

4 Seismic design of the joint waterproofing may refer to the waterproofing design method for settlement joints.

9.4.8 When the superstructure of a bridge at the bridge-tunnel junction needs to extend into the tunnel due to constraints by terrain or geological conditions, it shall meet the following requirements:

1 The length of the bridge superstructure extending into the tunnel should be less than 10m.

2 The lining section at the end of the bridge superstructure extending into the tunnel shall be provided with seismic joint.

3 Under the condition of providing sufficient space for bridge construction or formwork erection, the transverse reserved spacing between the bridge superstructure and the tunnel structure should be greater than 20cm and should be separated by rubber pads or other elastic pads in between.

9.4.9 For twin tunnels with narrow pillar, the minimum clearance should be greater than 1 width of excavation width in the case of seismic measures level 3 and greater than 1.5 excavation width in the case of seismic measures level 4. The rock pillar, if it is weak and fractured, should be stabilized by grouting to enhance its seismic capacity.

9.4.10 Twin-arch tunnels should be provided with composite middle wall. The section of the middle wall foundation shall be enlarged and its embedded depth should be increased according to seismic intensity and geological conditions to enhance its seismic stability.

9.4.11 When the tunnel is located in liquefiable soil, the following measures shall be incorporated:

1 Grout the liquefiable soil layer for stabilization or implement removal and replacement and other measures to eliminate or mitigate the impact of site liquefaction.

2 If the impact of site liquefaction cannot be eliminated or mitigated, the lining shall be designed to resist floatation, for example, by providing uplift piles and/or weights.

3 If the tunnel foundation is sandwiched with a thin layer of liquefiable soil, treatment against liquefaction may be dispensed with, but the influence of increased soil pressure and decreased frictional resistance caused by soil liquefaction shall be considered in checking its bearing capacity and stability against buoyancy.

9.4.12 The lining of a cut-and-cover tunnel shall be designed as reinforced concrete structure, and its foundation shall be placed on a stable foundation. The space behind the side wall of the cut-and-cover tunnel shall be backfilled with stone pitching or plain concrete, and circumferential seismic joints shall be provided at the boundary between cut-and-cover tunnel and under-cut tunnel. An anti-eccentric load retaining wall shall be provided where such load exists in the ground.

9.4.13 If there is a risk of falling rocks at the portal, a buffer layer shall be laid on the top of the cut-and-cover tunnel to protect against the impact from falling rocks, and the cut-and-cover tunnel shall be extended as appropriate.

9.5 Seismic design of special tunnel structures

9.5.1 The seismic calculation for shed tunnels shall meet the following requirements:

1 The seismic force on the top beam shall consider the horizontal and vertical seismic force on fill soil and beam.

2 The inner side wall (near the mountain side) shall be designed to resist the earth pressure load from the mountain, the seismic load from the soil mass, and all horizontal and vertical loads transmitted from the top beam.

3 Earth pressure calculation shall conform to Appendix A herein.

4 The seismic force on the outer side wall or column (the free side) shall consider all horizontal and vertical loads transmitted from the top beam.

5 For an integral shed tunnel with frame structure, the structure self-weight, self-weight-induced seismic force, earth pressure load and seismic load generated by soil mass shall be considered.

9.5.2 Seismic measures for the shed tunnel shall meet the following requirements:

1 The shed tunnel should preferably be designed as an integral structure. When designed as a prefabricated structure, structural measures to improve the structural integrity, e. g. reinforcing structural connection, shall be taken. Shed type cut-and-cover tunnels shall be provided with protective measures to prevent beams from falling off.

2 When prefabricated T-shaped top beam or H-shaped beam structure is employed for the shed tunnel, the beam shall be embedded in the groove of reinforced concrete top cap on the inner side wall using tenon as wide as the beam wing. The top beam cast in situ shall be flexibly connected with the top cap of the inner side wall by reinforcing steel bars.

3 The reinforced concrete cap of the inner side wall should be anchored in the bedrock of the side slope. When constructing a hollow structure behind the inner side wall of the cutting, the rock bolt should be anchored in the bedrock of the side slope through the hollow structure.

4 For steel-frame shed tunnels, if the column base is more than 3m below the pavement, reinforced concrete longitudinal and transverse struts shall be provided; if it is embedded deeper than 10m, separate seismic check shall be performed.

5 When half-buried shed tunnels are used, the outer elephant foot shall be anchored in the bedrock of the side slope, the eccentric load retaining wall should be separated from the lining of the cut-and-cover tunnel, and the gap between the retaining wall and the tunnel should be backfilled with stone pitching or plain concrete.

6 Cantilever shed tunnels should not be used in areas with seismic fortification ground motion classification of 0.20g and above.

7 For road sections with frequent cave-ins, landslides and fly rocks after an earthquake, steel shed tunnels or steel truss shed tunnels may be employed during the emergency repair phase, using steel pipes as columns, and structural steel as cross beams. The roof shall be able to resist the impact of fly rocks and should have a certain slope in the transverse direction. The shed tunnels may be removed during the restoration phase; they may not be put back in service until after permanent structures are reconstructed or after evaluation and reinforcement.

8 The steel reinforcement construction measures for the columns and joints of shed tunnel structures should follow the current Code for Seismic Design of Buildings (GB50011).

9.5.3 The seismic calculation for half tunnel structures shall meet the following requirements:

1 For half tunnel structures, the self-weight of the structure, the earthquake inertia caused by the self-weight, the upper and inner earth pressures, and the earthquake effect caused by the overlying soil and the soil mass on the inner side.

2 Earth pressure calculation shall conform to Appendix A herein.

3 Load calculation should follow the relevant provisions herein on cut-and-cover tunnels and shed tunnels.

9.5.4 Seismic measures for half tunnel structures shall meet the following requirements:

1 High cut slope half tunnels should be provided with prestressed anchor cables on the slope surface and inclined prestressed anchor cables attached to the arch crown.

2 The reinforced concrete cap of the side wall should be anchored in the bedrock of the side slope, and anchor cables should be provided in the side wall.

3 The steel reinforcement construction measures for the columns on the outer side, top beams and joints should follow the current Code for Seismic Design of Buildings (GB50011).

10 Shield Tunnels

10.1 General requirements

10.1.1 Seismic design of shield tunnels shall be based on the tunnel seismic fortification category, standard, design ground motion parameters and actual engineering geological conditions, and the seismic design method shall be determined according to Article 3.3.2 herein.

10.1.2 For Large-diameter shield tunnels in areas with seismic fortification ground motion classification of 0.2g and above, various methods should be used for comparative calculation. If necessary, verification by shaking table tests and other means can be used.

10.1.3 Shield tunnels should be built on a dense, uniform and stable foundation and shall be located to reasonably bypass unfavorable geological zones and strata. If this is not possible, reliable treatment measures shall be incorporated.

10.1.4 Response displacement method should be selected for seismic calculation of shield tunnels; the time history analysis method may also be used.

10.1.5 Seismic check of shield tunnels shall include strength and deformation checking at special locations (e.g. segment structure, segment joint, junctions with cross passage, shield working shaft or ventilation shaft joints) and ground stability check.

10.2 Seismic response calculation

10.2.1 The seismic calculation content and methods of shield tunnels shall meet the following requirements:

1 For shield tunnels, representative tunnel cross sections shall be selected for transverse seismic calculation according to stratum conditions and structural characteristics. Transverse response displacement method or time history analysis method may be used for calculation.

2 Longitudinal seismic calculation shall be carried out for shield tunnels. Longitudinal response displacement method should be adopted for general sections. For shield tunnels crossing complex terrain and geologic conditions, longitudinal generalized response displacement method or time history analysis method may be used for longitudinal seismic calculation.

3 For sections with major changes in structural types and significant spatial effects, such as at the interfaces of the shield tunnel with cross passage, working shaft or ventilation shaft, 3D model time history analysis method should be adopted for seismic calculation.

10.2.2 Transverse equivalent-stiffness beam model or beam-spring model may be used for transverse seismic calculation of shield tunnels, and longitudinal equivalent-stiffness beam model or beam-spring model may be used for longitudinal seismic calculation. The specific modeling method shall meet the following requirements:

1 In the transverse equivalent-stiffness beam model, the circumferential joint and the segment should be regarded as a whole, using the stiffness reduction coefficient to characterize the stiffness reduction of lining ring due to the circumferential joint, so that the segmental ring is simplified as an equivalent stiffness ring beam; the surrounding soil may be modeled as ground springs. In the cross-section beam-spring model, segments may be modeled using beam elements, circumferential joints may be modeled using elastic hinges, and surrounding soil may be modeled using ground springs. When it is necessary to simulate the staggered joint assembly of segments and consider the influence of longitudinal joints, the longitudinal 2-ring or 3-ring beam-spring model may be selected for internal force calculation. In this case, the longitudinal joints may be modeled using shear springs.

2 In the longitudinal equivalent-stiffness beam model, the longitudinal joint and segmental ring should be regarded as an integrity, and the tunnel structure is assumed as an equivalent-stiffness

beam in the longitudinal direction. The tensile, compressive and bending stiffness of the equivalent-stiffness beam may be obtained through the consistent condition of longitudinal deformation. The surrounding soil may be modeled using ground springs. In the longitudinal beam-spring model, segmental ring may be modeled using beam elements; longitudinal joint may be modeled using rotary, tension/compression and shear springs; and surrounding soil may be modeled using ground springs.

3 For shield tunnels with secondary lining, the effect of secondary lining should be considered in seismic calculation. In the calculation model, the secondary lining may be modeled using beam elements, and the interaction effect between the secondary lining and segmental ring may be modeled using spring elements.

4 For shield tunnels with internal structures such as travelled way slab and flue sheet, the effect of these internal structures shall also be considered in seismic calculation.

10.3 Seismic check

10.3.1 For shield tunnels, strength or deformation checking shall be performed on segment structure, joint structure, and interfaces with cross passage, working shaft or ventilation shaft. For shield tunnels with secondary lining, the strength or deformation of the secondary lining shall be checked. For shield tunnels passing through liquefiable ground, the tunnel stability against buoyancy shall be checked according to the provisions of Section 8.5 herein.

10.3.2 Seismic check of shield tunnels shall be performed using the partial safety factor method as illustrated in Chapter 8 herein.

10.3.3 The total amount of deformations at segment joints of the shield tunnel and at the interfaces between the tunnel and cross passage, working shaft or ventilation shaft shall not be greater than the permissible value for maintaining a watertight seal, and the deformation of steel bars or bolts at the joint shall be less than the yield deformation.

10.4 Seismic measures

10.4.1 Seismic measures for shield tunnels shall be applied to improve the seismic performance of the shield tunnel structure itself, and vibration damping measures to reduce the seismic energy transferred from the ground to the tunnel structure.

10.4.2 When the level of seismic measures for shield tunnels is Level 2, measures such as straight bolts and increasing the length of joint bolts should be implemented in the longitudinal direction to maintain proper rigidity of shield tunnel segments and joints, and waterproofing treatment shall be strengthened.

10.4.3 When the level of seismic measures for shield tunnels is Level 3, in addition to the measures specified in Article 10.4.2, measures such as providing waterstop strip or waterstop gasket that can accommodate large deformation and providing longitudinal joint bolts with elastic gaskets shall be implemented at the joints. For local areas, flexible segmental rings or steel segmental rings may be used.

10.4.4 A seismic isolation layer may be provided between the shield tunnel and ground according to seismic requirements. During construction, grouting materials with low shear stiffness may be used instead of the conventional grouting materials for application behind the wall.

10.4.5 The following anti-liquefaction measures shall be taken according to the range of potential liquefaction at the site:

1 Provide secondary lining along part or full length of the tunnel.

2 Increase the thickness of the soil cover.

3 Increase the overall longitudinal bending stiffness of the segmental lining.

4 Stabilize the ground by compaction or grouting method. The stabilized ground shall comply with the relevant provisions of Article 4.4.7 herein.

5 Treat liquefiable ground using sand drains, pressure relief wells, drainage piles, and other measures to reduce pore water pressure.

6 Special research shall be carried out in particular circumstances.

11 Immersed Tunnels

11.1 General requirements

11.1.1 For the seismic design of immersed tunnels, appropriate seismic calculation methods shall be selected based on the seismic safety evaluation report of the engineering site, and then to carry out transverse, longitudinal and integral and local seismic calculations. Shaking table tests should be carried out for verification if necessary.

11.1.2 Immersed tunnels shall be located in areas where the underwater bed is stable and conducive to earthquake resistance. Immersed tube sections and portals at both ends shall avoid being located in sections vulnerable to potential geological disasters such as earthquake-induced subsidence, liquefaction and landslide.

11.1.3 Seismic design standards for immersed tunnels shall be determined according to Table 11.1.3.

Table 11.1.3 Seismic design standards for immersed tunnels

Seismic fortification category	seismic fortification ground motion level	Component category	Structural performance requirements	Stress state	Foundation and backfill
A	E1 earthquake effect	Tube structure	Performance Requirements 1	Elastic	No liquefaction
		Shear key and shock-absorbing member	Performance Requirements 1	Elastic	
		Auxiliarymembers such as waterstop	Performance Requirements 1	Elastic	
	E2 earthquake effect	Tube structure and shear key	Performance Requirements 1	Elastic	Slight liquefaction
		Shock-absorbing member	Performance Requirements 2	Locally elastic-plastic	
		Auxiliarymembers such as waterstop	Performance Requirements 2	Locally elastic-plastic	

11.1.4 The design of immersed tunnels tunnel shall incorporate operation and maintenance equipment and facilities with good seismic performance. Operation and maintenance facilities such as fans shall also perform seismic check.

11.2 Seismic response calculation

11.2.1 Seismic response calculation of element structure, ground and foundation of immersed tunnels shall meet the following requirements:

1 Two-dimensional model should be used for element cross-section, ground and foundation calculations; two-dimensional or 3D model may be used for longitudinal seismic calculation; and the 3D model should be used for local calculation.

2 In preliminary research and conceptual design phases, a simplified model integrating the ground, backfill material and immersed tube structure may be developed to carry out the seismic response analysis.

3 When moving to the design phase after the scheme becomes stable, refined global and local models shall be established for seismic response analysis. Meanwhile, comparison and verification using various calculation methods or model tests should be conducted.

11.2.2 Selection of seismic response calculation method for immersed tunnels shall meet the following requirements:

1 The static method, response displacement method or dynamic time history analysis method should be used for seismic response calculation of structural safety of immersed tunnels and stability of surrounding ground.

2 The dynamic time history analysis method should be used for longitudinal seismic response calculation of immersed tunnels.

3 The response displacement method should be used to calculate the structural stability of the ventilation stack and the bearing capacity and deformation characteristics of the ground under earthquake conditions.

11.2.3 Seismic response analysis model of immersed tunnels shall meet the following requirements:

1 An overall model incorporating the immersed tube section and onshore cut-and-cover section shall be developed for seismic response calculation.

2 The whole tunnel and the surrounding soil should be simplified as a beam-particle-spring model, to derive the internal force of the structure and the relative displacement of the joints when subjected to longitudinal tension and compression and longitudinal bending under earthquake conditions using the finite element method.

3 The opening or compression and torsion angles of immersion joints in sections with major changes in ground and loading conditions, the joints between immersed tube elements and buried sections, and the joint element exposed from the existing riverbed or seabed surface shall be subjected to special analysis.

11.2.4 When the static method is used for the transverse seismic calculation of the tunnel, the cross-section of the immersed tube should be simplified as a plane frame model on an elastic foundation. The calculation method may follow Appendix A.

11.2.5 When the seismic calculation is carried out by the response displacement method, the earthquake effects on the immersed tube structure shall mainly consider the relative displacement of the ground, structural inertial force and shear force of surrounding soil. The calculation method may follow Appendix B.

11.2.6 Material constitutive models commonly used for seismic calculation of immersed tunnels should be selected from Table 11.2.6.

Table 11.2.6 Material constitutive models commonly used for seismic calculation of immersed tunnels

Item	Material	Constitutive model
Element structure	Reinforced concrete and steel-concrete composite structure	Linear elastic model
Reinforced concrete shear key	Reinforced concrete	Elastic-plastic model
Steel shear key and limit tie bar	Carbon steel structure	Elastic-plastic model
GINA, OMEGA, and groutable waterstop	Rubber	Mooney-Rivlin model
Prestressed anchor bundle	High strength low relaxation steel strand	Spring model
Ground, foundation cushion and backfilled gravel and sand, protective layer, etc.	Rock and soil mass	Mohr-Coulomb, Hardening Soil, Druck-Prager etc.

11.3 Seismic check

11.3.1 Seismic check of immersed tunnels shall meet the following requirements:

1 Longitudinal stress and displacement checking shall be performed to determine the proper length of elements, joint position and configuration and ventilation shaft position.

2 The most unfavorable load combinations, e. g. the combination of short-term and long-term effects of normal service limit state under E1 earthquake effect and accidental combination of bearing capacity limit state under E2 earthquake effect, shall be checked.

11.3.2 The transverse seismic check of the structural strength of elements shall conform to the relevant provisions of Section 8.2 herein, and the skid resistance of immersed pipe structures and stress deformation of elements shall carry out the seismic check for safety.

11.3.3 The bearing capacity of the foundation at the bottom of immersed tunnels, undisturbed soil layer around the foundation trench, bottom bedding course and subsoil layer, backfill cover and protective layer on the top shall perform the seismic check for liquefaction tendency and settlement.

11.3.4 If pile foundation is used for the immersed tunnel and appurtenances e. g. ventilation stack, the interaction between pile foundation and elements under earthquake effect shall be considered in seismic check. Seismic check of pile foundation for horizontal and vertical bearing capacity and horizontal displacement should follow the current Code for Seismic Design of Buildings (GB50011) and Technical Code for Building Pile Foundations (JGJ94).

11.3.5 Longitudinal seismic check of element structure and joints shall meet the following requirements:

1 The seismic dynamic response of immersed tunnels under the earthquake effects E1 and E2 when temperatures decrease in winter and increase in summer shall be checked. The changes of internal force in the joints caused by temperature changes, concrete shrinkage, etc. and the joint internal force induced by the seismic response to temperature changes in tunnel, as well as the differential settlement of foundation shall be considered.

2 For immersed tunnels with a length of more than 1,000 m, multi-point uniform excitation or nonuniform excitation may be used for longitudinal seismic response analysis and shaking table tests

should be conducted for verification.

3 When performing longitudinal seismic analysis of immersed tunnels, the force-deformation characteristics of the joint connection component shall be determined based on the maximum and minimum joint pressures during construction and operation of the joint waterstop, and relevant parameters shall be obtained through special tests if necessary. Under E2 earthquake effect, the watertight safety factor of waterstops shall not be less than 1.25 under accidental combination conditions.

4 The shear key in the joint shall be checked for shear capacity.

11.3.6 Seismic check of immersed tunnels shall ensure compliance with structural strength, displacement, stability and other control criteria under earthquake effects E1 and E2. In the absence of engineering data, the joint displacement control criteria may be selected according to Table 11.3.6.

Table 11.3.6 Displacement control criteria for immersed tunnel joints

S/N	Item	E1 earthquake effect	E2 earthquake effect
1	Immersion joint opening and closing (mm)	[+20, −20]	[+40, −40]
2	Maximum relative displacement of joint (mm)	[+5, −5]	[+10, −10]
3	Maximum relative deflection angle of joint (rad)	[−1, +1]x10^{-3}	[−3, +3]x10^{-3}

Note: 1. This table lists the calculated control values of joint displacement between elements only, including axial opening or compression displacement, vertical relative displacement and transverse relative displacement of the tunnel. The control criteria of specific projects shall be reasonably determined based on the special calculation results.

2. Due to factors such as varied stratigraphic distribution and property in specific projects, the displacement of immersed tunnel joints caused by earthquake is generally smaller than the calculated value. The numerical values in the table have considered the superposition of the dynamic displacement under seismic conditions and the static displacement caused by changes in water pressure, temperature, etc.

3. For joints at the junction between immersed tube section and buried section and at the interface of distinct geological media with major contrast in stiffness, or local high seismicity areas with seismic peak ground acceleration of 0.15g and above, special joint structures capable of withstanding large deformation may be selected based on seismic calculation results. However, due to the lack of cases in China at present, this table does not include displacement of such joint.

4. The relative deflection angle of the immersion joint is mainly controlled by the stiffness of the surrounding soil. Different foundation stiffness combinations are often used for seismic check.

5. Joint opening and closing, relative displacement and deflection angle are related to joint structural size, foundation and water depth conditions, etc. For specific projects, indicator values in the table shall be modified before application according to local site conditions.

11.4 Seismic measures

11.4.1 When the level of seismic measures is Level 1, the structural characteristics of joints and the water-tightness of waterstops shall be increased as typical in the design of immersed tunnels.

11.4.2 When the level of seismic measures is Level 2, the seismic measures for immersed tunnels shall meet the following requirements:

1 Measures to increase the concrete strength grade of immersed tube structure, reduce the thickness of components, increase the sectional reinforcement ratio, increase the diameter of steel bars, reasonably arrange steel bars, adopt steel shear keys, increase the shear surface of shear keys, add shear pins, etc. should be incorporated so as to improve the structure's ability to resist squeezing, movement and torsion. Rigid steel shall not be used for reinforcement, shear pin and shear key of the element structure.

2 Flexible joints shall be selected for immersed tunnels located on a soft foundation, and rigid or semi-rigid joints may be selected for immersed tunnels located on a hard foundation.

3 For immersed tunnels with large longitudinal seismic response, the joints between prefabricated elements should be provided with steel bars and finally cast as rigid joints.

4 Seismic joints and limit cable devices with large longitudinal and transverse limits should be used at the connection between the immersed tube section and the onshore section.

5 The ground in the immersed tube section shall be treated by means of removal and replacement or pile foundation to avoid large-area liquefaction-induced ground subsidence due to earthquakes. Independent foundations should be used for buildings in the buried section and at portals.

6 Silty sand and medium sand with uniform particles shall not be used as backfill materials for the overburden protective layer and anti-collision structure at the top and both sides of elements. The backfilling materials should be well-graded and have good filter properties to avoid earthquake-induced liquefaction or element floating. Light backfilling materials may be used at the top of the tunnel in the buried section.

11.4.3 When the level of seismic measures is Level 3 or 4, the seismic structure and engineering measures of the element structure shall be determined through individual study in immersed tunnel design.

12 Cut-and-cover Tunnels

12.1 General requirements

12.1.1 Cut-and-cover tunnels should be built on dense, uniform and stable foundation, avoiding unfavorable geological conditions wherever possible. When the tunnel is located in unfavorable sections such as soft soil, liquefiable soil or fault fracture zone, their impact on the seismic stability of the tunnel structure shall be analyzed and corresponding measures shall be taken.

12.1.2 The seismic calculation method for cut-and-cover tunnels shall follow relevant provisions in Chapter 6.

12.1.3 Seismic check of cut-and-cover tunnels shall include structural strength, deformation and stability checking.

12.1.4 When the stratum where the cut-and-cover tunnel is located contains a liquefiable soil layer, the influence of the liquefiable soil layer on the stress and stability of the structure shall be analyzed, and treatment measures against liquefaction shall also be proposed.

12.1.5 Backfilling materials and quality of cut-and-cover tunnels shall meet the relevant seismic requirements.

12.2 Seismic response calculation

12.2.1 A reasonable seismic calculation method shall be selected according to the seismic fortification category, criteria and performance requirements and based on factors such as engineering environment and geological conditions, and shall meet the following requirements:

1 Cut-and-cover structures in uniform and isotropic surrounding ground, with regular, standard sectional shape, and without abrupt changes may be analyzed as plane strain problems. Static method or response displacement method should be adopted for seismic calculation of sections in soil stratum. Time history analysis method may be used when geological conditions or structural types are complicated.

2 When ground conditions along the tunnel alignment are highly variable, the longitudinal response displacement method or time history analysis method may be used for longitudinal seismic calculation.

3 When the cross-sectional shape of the tunnel changes significantly or the tunnel forms a whole with adjacent buildings or structures, 3D dynamic time history analysis method should be used for calculation.

4 For the enclosure joined as a whole with the main structure, it may be calculated together with the main structure; while for the enclosure not joined or weakly joined with the main structure, it may be calculated separately.

12.2.2 When calculating the seismic response of cut-and-cover tunnels, the earthquake effect adopted shall meet the following requirements:

1 Only horizontal earthquake effect along the structural section may need to be considered.

2 For complex structural types with irregular structural layout and major changes in shape, and non-homogeneous ground with abrupt changes in the longitudinal direction, the horizontal earthquake effect in the longitudinal direction of the structure should also be considered.

3 The vertical earthquake effect should be considered for cut and cover tunnels in areas where the tunnel depths or the geological conditions of the basement are highly variable, or the seismic fortification ground motion classification is 0.15g or above.

12.3 Seismic check

12.3.1 For cut-and-cover tunnels, the structural effect value under earthquake effect shall be calculated according to the earthquake effect combinations, and the tunnels shall be checked for structural strength, deformation, or stability according to the performance requirements.

12.3.2 For Performance Requirements 1, sectional bearing capacity under E1 earthquake effect shall be subject to seismic check.

12.3.3 For seismic Performance Requirements 2 or 3, the overall deformation performance of the structure shall be checked under E2 earthquake effect. Its corresponding parameters and calculation model shall meet calculation requirements in the elastic-plastic stage.

12.3.4 When the cut-and-cover tunnel structure is located in liquefiable ground, its stability against buoyancy in the event of liquefaction shall be checked. The frictional resistance of the liquefiable soil layer against enclosures (e. g. diaphragm wall) and uplift piles should be calculated using the reduction coefficient of liquefaction effect determined from the measured liquefaction strength ratio.

12.3.5 For cut-and-cover tunnels with enclosures as a part of the main structure, the anti-floating effect of the enclosures shall be considered in checking the tunnel stability against buoyancy.

12.4 Seismic measures

12.4.1 When the level of seismic measures for cut-and-cover tunnels is Level 1 or 2, their structural requirements shall meet the following requirements in addition to the relevant provisions of Code for Design of Concrete Structures (GB 50010):

1 Each component of the cut-and-cover tunnel structure shall have enough deformation performance and toughness, so that it has good seismic response capability under the shear deformation imposed by surrounding ground.

2 Cast-in-situ structure should be adopted for the main structure of the cut-and-cover tunnel. When some prefabricated members are used, the prefabricated members shall be reliably connected to the surrounding members.

3 The minimum size of underground reinforced concrete frame structural members shall not be smaller than the size specified for similar above-ground structural members.

4 The waterproofing and drainage measures shall be determined based on structure shape, construction method, construction environment, etc.

12.4.2 When the level of seismic measures for a cut-and-cover tunnel is Level 3, its center column, roof and floor designs shall meet the following requirements:

1 The minimum total reinforcement ratio of the center column in the longitudinal direction shall be increased by 0.2%. The connections between the center column and the beam or the roof and floor shall meet the structural requirements of the column stirrup densification area, which is the same in scope as the column members of the above ground structure with the same seismic fortification category.

2 The roof and floor should be designed as a beam-slab structure. When slab column-wall structure is adopted, hidden structural beams should be provided in the slab belt on top of the column, and their structural requirements are the same as those for corresponding members of similar aboveground structures.

3 For a composite diaphragm wall, at least 50% of the negative bending moment of rebar of the roof and floor shall be anchored into the diaphragm wall, and the anchored length shall be determined by stress calculation. Positive bending moment rebars shall be anchored into the lining by an anchorage length, not less than the specified value.

4 When cutting a hole in the diaphragm, the width of the hole shall not be greater than 30% of the width of the diaphragm. The portals should be arranged to make the distribution of structural mass and stiffness more uniform and symmetrical to avoid local abrupt changes. Edge beams or hidden beams meeting the structural requirements shall be set around the holes.

12.4.3 When the level of seismic measures for cut-and-cover tunnels is Level 1, the rebar structural requirements may be in accordance with the seismic provisions in current Code for Design of Concrete Structures (GB 50010). When the level of seismic measures for a cut-and-cover tunnel is Level 2 or 3, the following provisions must be followed in addition to the above provisions:

1 In principle, longitudinal bars should be arranged along the entire length. Where a lap joint is required, welded or mechanical connection may be made.

2 Transverse bars should be closed stirrups or spiral stirrups. The spacing of transverse bars shall be less than 12 times the diameter of longitudinal bars and 1/2 of the minimum sectional dimensions.

3 When providing stirrups in a rectangular cross-section and the length of one side is greater than 48 times the diameter of stirrups, stirrups need to be added at the middle position.

4 When the stirrups need to be lapped, the lap joint must be configured to completely transfer the stress without affecting the overall strength of the stirrups. In addition, the joints cannot be concentrated in a specific direction and shall be set in a staggered way.

12.4.4 When a cut-and-cover tunnel structure passes through a paleochannel where the bank slope may slide during an earthquake or through the ground susceptible to significant differential subsidence, measures such as replacing soft soil or providing pile foundation shall be taken.

12.4.5 Cut-and-cover tunnels shall avoid crossing the ground susceptible to potential liquefaction. When this is not possible, the adverse effects of liquefaction on structural safety and stability shall be analyzed, and the following structural measures shall be taken:

1 Grout the liquefiable soil layer for stabilization or implement soil removal and replacement and other measures to eliminate or mitigate the potential for site liquefaction.

2 If no measures are taken to eliminate or reduce the potential for liquefaction of the liquefiable layer in the surrounding soil and ground, the potential for uplift shall be considered, and anti-floating measures such as adding uplift piles and/or providing weights shall be adopted when necessary.

3 When the cut-and-cover tunnel structure intersects a thin interlayer of liquefiable soil, or when the cut-and-cover tunnel structure, for which a diaphragm wall deeper than 20m is used as enclosure during construction, encounters a liquefiable soil layer, only its underlying layer may need to be treated to prevent liquefaction, while the soil outside the enclosure may not need to be treated. However, the influence of factors such as increased soil pressure and decreased frictional resistance caused by soil liquefaction should be considered in the checking of bearing capacity and stability against buoyancy.

13 Tunnel Portals

13.1 General requirements

13.1.1 Cut-and-cover or reinforced concrete tunnel portals should be selected and arranged perpendicular to the slope surface wherever possible.

13.1.2 The design of the tunnel portal shall minimize disturbance to the mountain as much as possible, and measures shall be taken to control the height of the side and front slope.

13.1.3 When the terrain at the portal is steep, or the side and front slope lacks in stability, measures such as lengthening the cut-and-cover tunnel, appropriately increasing the thickness of backfill on the cut-and-cover tunnel, and providing active or passive protection nets should be taken to prevent falling rock from hitting the cut-and-cover tunnel.

13.2 Seismic response calculation and check

13.2.1 Static method should be adopted for seismic response calculation of tunnel portals.

13.2.2 The strength checking of the cut-and-cover portal is the same as that of the cut-and-cover structure and shall be carried out in accordance with the relevant provisions in Section 9.3 herein. Seismic response calculation of a wall-type portal shall be carried out according to Appendix A.3 herein, and its seismic check shall be carried out according to Section 8.4 herein.

13.2.3 Strength checking of reinforced concrete portals shall comply with relevant provisions in Section 8.2 herein.

13.3 Seismic measures

13.3.1 For tunnel portals with seismic design requirements, the construction materials shall, as a minimum, meet the requirements in Table 7.2.1.

13.3.2 Seismic measures for cut-and-cover portals shall meet the following requirements:

1 The backfill side and front slope ratio should be controlled. When the level of seismic measures for the tunnel is Level 2, the side and front slope ratio should not be greater than 1:1.25. When the level of seismic measures for the tunnel is above Level 2, the side and front slope ratio should not be greater than 1:1.5.

2 The gaps behind the side walls of the cut-and-cover tunnel shall be backfilled with stone pitching, rubble concrete or plain concrete. When the level of seismic measures for the tunnel is Level 3 or 4, the backfill height on both sides should not be less than 7m.

3 Make reasonable use of backfilling materials above the top of the cut-and-cover tunnel.

13.3.3 Seismic measures for wall-type portals shall meet the following requirements:

1 The minimum thickness of the portal wall shall not be less than 0.8m. The wall top shall be at least 1.0m higher than the backfill surface on the wall back.

2 The portal wall and lining are connected by steel bars whose diameter should be the same as that of main bars in the lining. When the level of seismic measures for the tunnel is Level 2, the circumferential spacing of connecting steel bars shall not be more than 25cm. When the level of seismic measures for the tunnel is Level 3, the circumferential spacing of connecting steel bars shall not be more than 20cm. When it is Level 4, the circumferential spacing of the connecting steel bars shall not be more than 15cm.

3 The foundation of the portal wall shall be placed on a stable ground with a bearing capacity of not less than 300kPa. The portal wall foundation shall be embedded not less than 0.5m in rock or

1.2m in soil.

4 When the portal wall is wide, or the ground conditions are highly variable, seismic joints shall be provided. The seismic joints should be considered in conjunction with settlement joints and arranged at a spacing of not more than 10m.

Appendix A Static Method

A.1 Modified static method

A.1.1 When the static method is used for calculation, the earthquake effect shall include three components: earthquake inertia force from lining self-weight, earthquake inertia force from overlying soil column and increment in earthquake-induced lateral earth pressure (Fig. A.1.1).

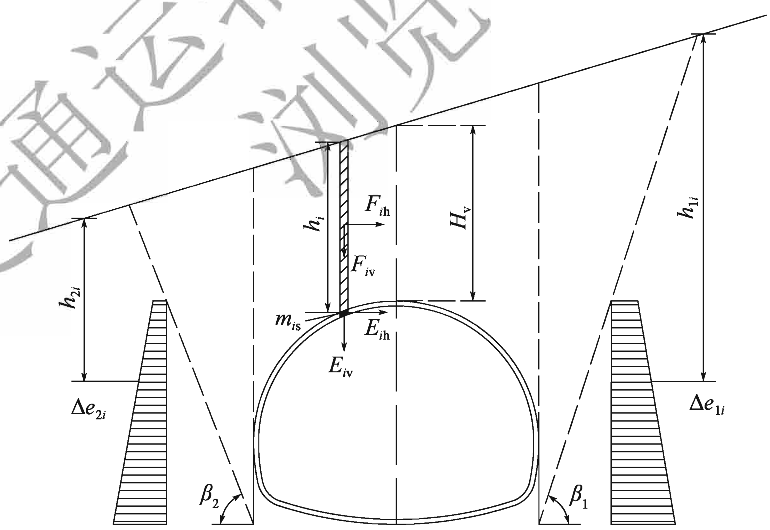


Fig. A.1.1 Diagram of modified static method for calculation

A.1.2 Horizontal and vertical earthquake effects from lining self-weight shall be calculated using Eqs. (A.1.2-1) and (A.1.2-2), respectively.

$$E_{ih} = A_h m_{is} = C_i C_s A m_{is} \tag{A.1.2-1}$$

$$E_{iv} = K_v A_h m_{is} = K_v C_i C_s A m_{is} \tag{A.1.2-2}$$

Where C_i ——seismic importance coefficient, obtained from Table 3.1.5 herein;

C_s ——site adjustment coefficient of seismic peak ground acceleration, obtained from Table 5.2.1 herein;

A—— peak acceleration of the horizontal basic ground motion;

m_{is}——weight at calculation point of tunnel lining (kg);

K_v——ratio of vertical peak ground acceleration to horizontal peak ground acceleration, obtained from Table 5.3.1 herein.

A.1.3 In calculating the earthquake inertia force of overlying soil column, it is assumed to act on the soil unit centroid (Fig. A.1.1), and its value shall be calculated from Eqs. (A.1.3-1) ~ (A.1.3-3). In calculating structural internal force, the shift theorem may be applied to simplify this earthquake inertia into nodal forces and bending moments acting on the upper half of the lining.

Horizontal earthquake effect of overlying soil column:

$$F_{ih} = A_h Q_i / g \tag{A.1.3-1}$$

Vertical earthquake effect of overlying soil column:

$$F_{iv} = K_v A_h Q_i / g \tag{A.1.3-2}$$

Vertical earth pressure of overlying soil column:

$$Q_i = \frac{\gamma}{2}[2h_i B_i(\lambda_1 h_i^2 + \lambda_2 h_i^2)\tan\theta_0] \tag{A.1.3-3}$$

where A_h——peak acceleration of the horizontal design seismic motion, obtained from Article 5.2.1 herein;

g——gravitational acceleration (generally 9.8m/s^2);

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

h_i——height (m) of the overlying soil column, which shall be determined based on the equivalent calculated height, H_v, of the overlying soil column on the top of the tunnel in Table A.1.5;

B_i——width of overlying soil column (m);

θ_0——friction angle on both sides of soil column (°);

λ_1, λ_2——lateral pressure coefficients on inner and outer sides in the event of an earthquake. Here, the inner and outer sides are with respect to eccentric load tunnels; the inner side is the side next to the mountain. When the ground surface is horizontal, λ_1 is taken for each side. Lateral pressure coefficients on inner and outer sides in the event of an earthquake are calculated using Eqs. (A.1.3-4) ~ (A.1.3-11):

$$\lambda_1 = \frac{(\tan\beta_1 - \tan\varphi_1)(1 - \tan\theta_1\tan\theta)}{(\tan\beta_1 - \tan\alpha)[1 + \tan\beta_1(\tan\varphi_1 - \tan\theta_1) + \tan\varphi_1\tan\theta_1]} \tag{A.1.3-4}$$

$$\lambda_2 = \frac{(\tan\beta_2 - \tan\varphi_2)(1 + \tan\theta_2\tan\theta)}{(\tan\beta_2 + \tan\alpha)[1 + \tan\beta_2(\tan\varphi_2 - \tan\theta_2) + \tan\varphi_2\tan\theta_2]} \tag{A.1.3-5}$$

$$\tan\beta_1 = \tan\varphi_1 + \sqrt{\frac{(\tan^2\varphi_1 + 1)(\tan\varphi_1 - \tan\alpha)}{(\tan\varphi_1 - \tan\theta_1)}} \tag{A.1.3-6}$$

$$\tan\beta_2 = \tan\varphi_2 + \sqrt{\frac{(\tan^2\varphi_2 + 1)(\tan\varphi_2 + \tan\alpha)}{(\tan\varphi_2 - \tan\theta_2)}} \tag{A.1.3-7}$$

$$\varphi_1 = \varphi_g - \theta \tag{A.1.3-8}$$

$$\varphi_2 = \varphi_g + \theta \tag{A.1.3-9}$$

$$\theta_1 = \theta_0 - \theta \tag{A.1.3-10}$$

$$\theta_2 = \theta_0 + \theta \tag{A.1.3-11}$$

where φ_g——calculated friction angle of surrounding rock (°);

θ——earthquake angle (°), selected from Table A.1.3;

α——ground slope angle (°); $\alpha = 0$ if the ground surface is horizontal;

β_1, β_2——inner and outer fracture angles at a maximum thrust (°).

Meanings of other symbols are the same as above.

Table A.1.3 Correspondence between horizontal basic peak ground acceleration and earthquake angle

Seismic fortification intensity		Ⅶ	Ⅷ		Ⅸ
Seismic design peak ground acceleration classification A (g)		0.10,0.15	0.20	0.30	0.40
Earthquake angle θ(°)	Above water	1.5	3.0	4.5	6.0
	Underwater	2.5	5.0	7.5	10.0

A.1.4 Lateral earth pressure increment during an earthquake shall be calculated using Eqs. (A.1.4-1) and (A.1.4-2) and applied asymmetrically.

Inner side earth pressure increment:

$$\Delta e_{1i} = C_i C_s \gamma h_{1i}(\lambda_1 \lambda) \tag{A.1.4-1}$$

Outer side earth pressure increment:

$$\Delta e_{2i} = C_i C_s \gamma h_{2i}(\lambda_2 \lambda') \tag{A.1.4-2}$$

where λ, λ'——normal lateral pressure factor on inner and outer sides;

h_{1i}, h_{2i}——the distance from any point i on the inner and outer sides of the lining to ground surface (m);

Meanings of other symbols are the same as above.

A.1.5 The value of equivalent calculated height (H_v) of the overlying soil column over the arch crown shall be determined by Table A.1.5.

Table A. 1. 5 Value of equivalent calculated height (H_v) of the overlying soil column over the arch crown

Surrounding rock mass	Two-lane tunnel	Three-lane tunnel
Ⅰ ~ Ⅱ	0. 5B	0. 5B
Ⅲ	1. 3B	0. 8B
Ⅳ	2. 0B	1. 8B
Ⅴ	2. 5B	2. 0B

Note: In the table, B is the tunnel span (m).

A. 2 Seismic calculation of cut-and-cover tunnels and shed tunnels

A. 2. 1 Static method may be used for seismic calculation of cut-and-cover tunnels and shed tunnels. The earthquake effect includes three components: earthquake inertia force from lining self-weight, earthquake inertia force from overlying backfill and increment in earthquake-induced lateral backfill pressure (Fig. A. 2. 1).

A. 2. 2 Inertia forces from self-weight of the structure, E_{ih} and E_{iv}, may be obtained from Eqs. (A. 1. 2-1) and (A. 1. 2-2).

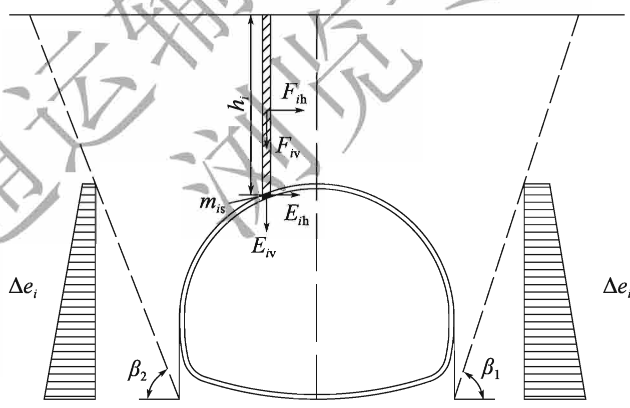


Fig. A. 2. 1 Sketch for seismic calculation of cut-and-cover tunnels and shed tunnels

A. 2. 3 The earthquake effect of backfill at the top of the tunnel is calculated using Eqs. (A. 2. 3-1) and (A. 2. 3-2). When calculating the internal force in the structure, the shift theorem of force may be adopted to simplify the horizontal earthquake effect of backfill into node forces and bending moments applied on the upper half of the structure.

The horizontal earthquake effect of backfill on the roof of tunnel:

$$F_{ih} = C_i C_s A h_i \gamma / g \qquad \text{(A. 2. 3-1)}$$

The vertical earthquake effect of backfill on the roof of tunnel:

$$F_{iw} = K_v C_i C_s A h_i \gamma / g \qquad (A.2.3\text{-}2)$$

where h_i——backfill thickness at calculation point (m);

γ——unit weight of backfill (kN/m^3);

g——gravitational acceleration (m/s^2);

Meanings of other symbols are the same as above.

A.2.4 The lateral pressure increment generated by the side backfill may be calculated using Eqs. (A.1.4-1) and (A.1.4-2).

A.3 Seismic calculation of end-wall tunnel portals

A.3.1 The structural internal force in tunnel portals under earthquake effect may be calculated by the static method. For the earthquake effect, the inertia caused by the self-weight of the portal wall and the retaining wall and the seismic active (passive) earth pressure shall be considered.

A.3.2 The earthquake inertia caused by self-weight of portal wall and retaining wall may be calculated using Eqs. (A.3.2-1) and (A.3.2-2).

The horizontal earthquake inertia caused by self-weight of portal wall and retaining wall:

$$E_{ihw} = C_i C_s A \psi_{iw} m_{iw} \qquad (A.3.2\text{-}1)$$

The vertical earthquake inertia caused by self-weight of portal wall and retaining wall:

$$E_{ivw} = K_v C_i C_s A \psi_{iw} m_{iw} \qquad (A.3.2\text{-}2)$$

where E_{ihw}——horizontal seismic load at wall gravity center above the ^{ith} section (kN);

E_{ivw}——vertical seismic load at wall gravity center above the i^{th} section (kN);

m_{iw}——wall weight above the i^{th} section (kg);

ψ_{iw}——distribution factor of horizontal seismic load along wall height, which may be taken as specified in Table A.3.2. Meanings of other symbols are the same as above.

Table A.3.2 Distribution factor of horizontal seismic load along wall height, ψ_{iw}

Wall height	Class of highway	
	Expressway; Class-1 and Class-2 highways	Class-3 and Class-4 highways
$H \leq 2m$	1	1
$H > 12m$	$1+\frac{H_{iw}}{H}$	1

Note: H in the table is the height from the toe of the wall to the top of the wall; H_{iw} is the height from the center of gravity of the wall above section i to the bottom of the wall, as shown in Fig. A.3.2:

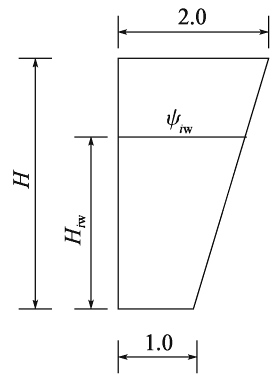


Fig. A.3.2 Schematic diagram of H and H_{iw} (unit: m)

A.3.3 The seismic active earth pressure acting on the back of the portal wall and retaining wall may be calculated using Eqs. (A.3.3-1) ~ (A.3.3-3) (refer to Fig. A.3.3).

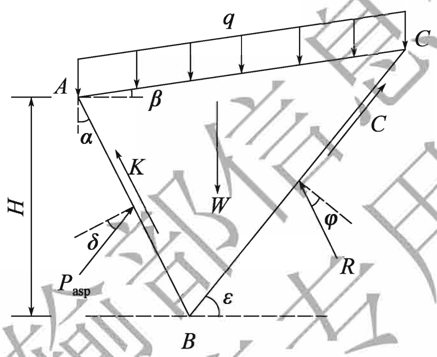


Fig. A.3.3 Schematic diagram for calculation of seismic earth pressure

$$E_{ea} = \left(\frac{1}{2}\gamma H^2 + qH\frac{cos\alpha}{cos(\alpha-\beta)}K_a - 2cHK_{ca}\right) \tag{A.3.3-1}$$

$$K_a = \frac{\cos^2(\varphi - \alpha - \theta)}{\cos\theta\cos^2\alpha\cos(\alpha + \delta + \theta)\left[1 + \sqrt{\dfrac{\sin(\varphi + \delta)\sin(\varphi - \beta - \theta)}{\cos(\alpha - \beta)\cos(\alpha + \delta + \theta)}}\right]^2} \tag{A.3.3-2}$$

$$K_{ca} = \frac{1 - \sin\varphi}{\cos\varphi} \tag{A.3.3-3}$$

Where, r——unit weight of backfill (kN/m^3);

H——portal wall or retaining wall height (m);

q——uniformly distributed load on sliding wedge (kPa);

α——angle between portal wall or retaining wall back and vertical line (°);

β——angle between fill surface and horizontal plane (°);

c——cohesion factor of fill;

K_a——coefficient of seismic active earth pressure, calculated by Eq. (A.3.3-2);

K_{Ca}——coefficient of active earth pressure caused by soil cohesion, calculated by Eq. (A.3.3-3);

φ——internal friction angle of fill (°);

δ——friction angle between fill and portal wall or retaining wall back (°);

θ——earthquake angle (°), selected from Table A.1.3.

A.3.4 The seismic passive earth pressure acting on the back of the portal wall and retaining wall may be calculated using Eqs. (A.3.4-1) ~ (A.3.4-3).

$$E_{ep} = \left(\frac{1}{2}\gamma H^2 + qH\frac{\cos\alpha}{\cos(\alpha-\beta)}\right) \tag{A.3.4-1}$$

$$K_{psp} = \frac{\cos^2(\varphi+\alpha-\theta)}{\cos\theta\cos^2\alpha\cos(\alpha-\delta+\theta)\left[1+\sqrt{\frac{\sin(\varphi+\delta)\sin(\varphi+\beta-\theta)}{\cos(\delta+\theta-\alpha)\cos(\alpha-\theta)}}\right]^2} \tag{A.3.4-2}$$

$$K_{cp} = \frac{\sin(\varphi-\theta)+\cos\theta}{\cos\theta+\cos\varphi} \tag{A.3.4-3}$$

where K_{psp}——coefficient of seismic passive earth pressure;

K_{cp}——coefficient of passive earth pressure caused by soil cohesion, calculated using Eq. (A.3.4-3);

Meanings of other symbols are the same as above.

Appendix B
Response Displacement Method

B.1 Transverse response displacement method

B.1.1 In the transverse response displacement method, the lining structure should be modeled using beam elements, and the interaction effect between lining and ground should be modeled using compression and shear springs (Fig. B.1.1).

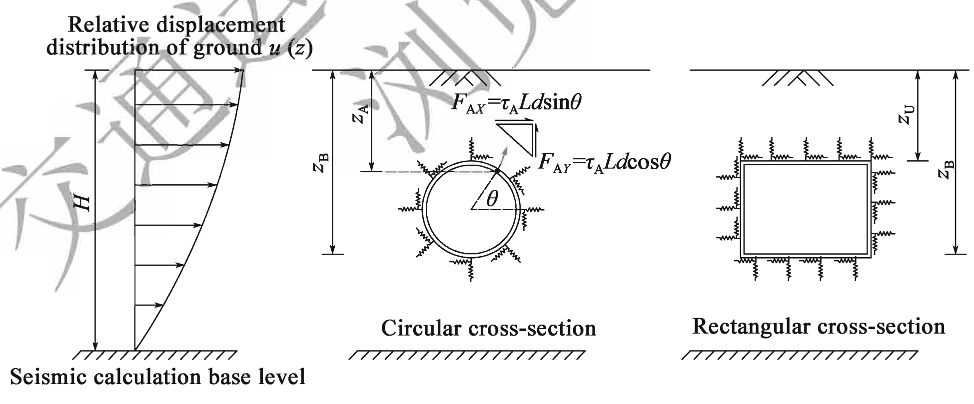


Fig. B.1.1 Schematic diagram of calculation based on transverse response displacement method

B.1.2 Earthquake effects on structures in the transverse response displacement method shall include relative displacement of the ground, structural inertial force and shear force of surrounding ground.

1 Ground relative displacement may be calculated using Eqs. (B.1.2-1) and (B.1.2-2):

$$u'(z) = u(z) - u(z_B) \qquad \text{(B.1.2-1)}$$

$$u(z) = \frac{1}{2}u_{\max}\cos\left(\frac{\pi z}{2H}\right) \tag{B.1.2-2}$$

where $u'(z)$——the difference in free-field ground displacements (m) at depth z and at the bottom of the structure;

$u(z)$——the relative displacement (m) between the free-field ground at depth z and the seismic base level; the actual displacement response of the free-field ground may be calculated based on the acceleration time history at the engineering site; when the ground is homogeneous, it may also be calculated using the simplified Eq. (B.1.2-2), with the peak displacement of the ground surface, $u_{\max}$, selected from Article 5.2.2 herein;

$u(_{Zb})$——the relative displacement (m) between the free-field ground at the elevation of structure bottom and the seismic base level, using the same calculation method as u (z);

H——the thickness of the soil layer from the ground surface to the seismic base level (m).

2 The nodal force at the element A caused by ground shear stress may be calculated using Eqs. (B.1.2-3) to (B.1.2-6).

Shear force at the perimeter of the circular cross section:

$$F_{AX} = \tau_{A}\mathrm{Ld}\sin\theta \tag{B.1.2-3}$$

$$F_{AY} = \tau_{A}\mathrm{Ld}\cos\theta \tag{B.1.2-4}$$

$$\tau_A = \frac{\pi G_D}{4H}u_{\max}\sin\left(\frac{\pi z_A}{2H}\right) \tag{B.1.2-5}$$

Shear stress in the side wall of rectangular cross section:

$$\tau_s = (\tau_u + \tau_B)/2 \tag{B.1.2-6}$$

where F_{AX}, F_{AY}——horizontal and vertical nodal forces (kN) acting on point A;

τ_A——shear stress (kPa) at point A, which should be calculated based on the acceleration time history at the engineering site; when the strata are approximately horizontally layered, it may also be calculated using Eq. (B.1.2-5);

L——influence length (m) of the ground spring, taken as the sum of one-half lengths of two adjacent elements;

d——calculated width of the structure in the longitudinal direction, generally taken as unit length (m);

θ——cangle between normal direction and horizontal direction at point A (Fig. B.1.1) (°);

G_D——dynamic shear modulus of the ground (kPa);

Z_A——depth of action point A (m);

τ_u ——shear stress in the roof of a rectangular structure, using the same calculation method as τ_A (kPa);

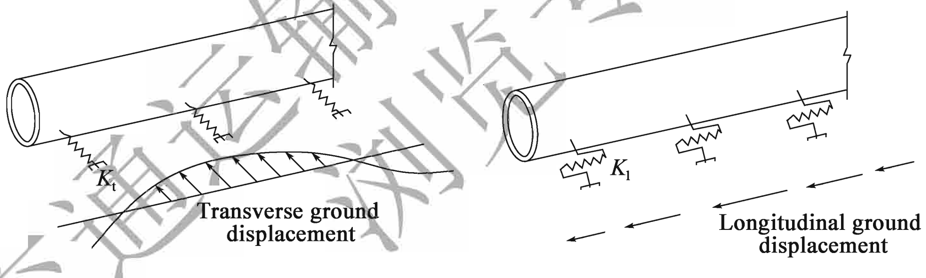
τ_B ——shear stress in the floor of a rectangular structure, using the same calculation method as τ_A (kPa);

Meanings of other symbols are the same as above.

B.1.3 The stiffness of ground springs may be calculated according to Article 6.6.2 of the Code for Seismic Design of Urban Rail Transit Structures (GB50909—2014).

B.2 Longitudinal response displacement method

B.2.1 When the longitudinal response displacement method is used for calculation, the lining structure should be modeled using beam elements and the interaction effect between the lining and ground may be modeled using transverse and axial ground springs. The total length of the model should not be less than the wavelength of a seismic wave or should be the full length of the tunnel (Fig. B.2.1).



a)Transverse   b)Longitudinal

B.2.1 Schematic diagram of tunnel longitudinal response displacement method

B.2.2 When the ground is approximately homogeneous, it may be assumed that both the longitudinal displacement of the ground along the tunnel axis direction, u_A, and the transverse displacement in the direction perpendicular to the tunnel axis are distributed according to sinusoidal law (Fig. B.2.2). Longitudinal and transverse displacements of the ground may be calculated by Eqs.

(B.2.2-1) to (B.2.2-6).

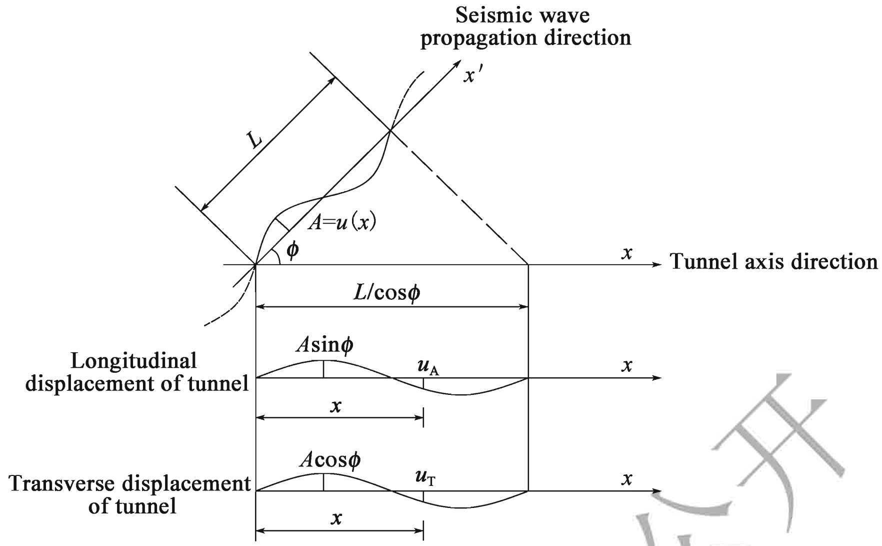


Fig. B.2.2 Ground displacement decomposition model based on longitudinal response displacement method

Longitudinal displacement of the ground:

$$u_A(x,z) = u(z)\sin\varphi\sin\left(\frac{2\pi\cos\varphi}{L}\cdot x\right) \tag{B.2.2-1}$$

Transverse displacement of the ground:

$$u_T(x,z) = u(z)\cos\varphi\sin\left(\frac{2\pi\cos\varphi}{L}\cdot x\right) \tag{B.2.2-2}$$

$$L = \frac{2L_1L_2}{L_1 + L_2} \tag{B.2.2-3}$$

$$L_1 = V_sT_S \tag{B.2.2-4}$$

$$L_2 = V_0T_S \tag{B.2.2-5}$$

$$T_G = \frac{4H}{V_s} \tag{B.2.2-6}$$

where $u_A(x,z)$——component (m) of free-field ground displacement in the longitudinal direction with respect to tunnel axis at depth z;

$u_T(x,z)$——component (m) of free-field ground displacement in the transverse direction with respect to tunnel axis at depth z;

L——apparent wavelength of the ground (m);

L_1——shear wavelength (m) at ground surface;

L_2——shear wavelength (m) at calculation base level;

V_s——shear wave velocity (m/s) at ground surface;

V_0——calculated shear wave velocity (m/s) of ground at base level;

T_s——natural period of the ground, taken as 1.25 T_G(s) for calculation:

ϕ——included angle between the propagation direction of seismic wave and the axis of the shield tunnel (Fig. B.2.2) (°).

B.2.3 In the longitudinal response displacement method, the stiffness of ground springs may be calculated according to Article 6.8.3 of the Code for Seismic Design of Urban Rail Transit Structures (GB50909—2014).

B.3 Simplified calculation formula in response displacement method for circular shield tunnels in homogeneous ground

B.3.1 Seismic internal force in circular shield tunnel cross section in homogeneous ground may be calculated using Eqs. (B.3.1-1) to (B.3.1-4). The sign of the seismic internal force in the transverse direction shall be determined according to Fig. B.3.1.

$$M(\theta) = \frac{1.3 \times 3\pi E_s I_s}{2RH} U \sin\left(\frac{\pi H_c}{2H}\right) C \sin(2\theta) \qquad (B.3.1\text{-}1)$$

$$N(\theta) = -\frac{1.3 \times 3\pi E_s I_s}{R^2 H} U \sin\left(\frac{\pi H_c}{2H}\right)\left(1 + \frac{G_D R^3}{6E_s I_s}\right) C \sin(2\theta) \qquad (B.3.1\text{-}2)$$

$$Q(\theta) = -\frac{1.3 \times 3\pi E_s I_s}{R^2 H} U \sin\left(\frac{\pi H_c}{2H}\right) C \cos(2\theta) \qquad (B.3.1\text{-}3)$$

$$C = \frac{4(1 - \nu_D) G_D R^3}{(3 - 2\nu_D) G_D R^3 + 6(3 - 4\nu_D) E_s I_s} \qquad (B.3.1\text{-}4)$$

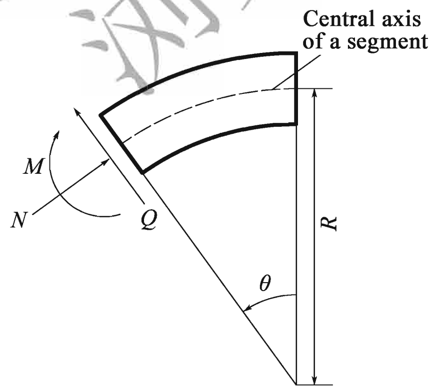


Fig. B.3.1 Sign notation of seismic internal forces in the transverse direction (+shown in the diagram)

where U——the maximum relative displacement (m) of the ground surface, which may be determined with z = 0 according to Eq. (B.1.2-2);

H_c——distance from ground surface to tunnel center (m);

R——radius of central axis of tunnel cross-section lining (m);

E_s——modulus of elasticity of the lining (kPa);

I_s——moment of inertia of lining section (m^4), $I_s = \dfrac{bt^3}{12}$, where b represents the width of the segment and t represents the thickness of the segment;

v_D——Poisson ratio of the ground;

G_d——dynamic shear modulus of the ground (kPa).

B.3.2 Longitudinal seismic internal force for a circular shield tunnel in homogeneous ground may be calculated using the following equations:

1 When the angle between the propagation direction of seismic wave and the tunnel axis is 45°, and the tunnel is subjected to the maximum axial force, the maximum axial tension, axial pressure and correlation coefficient may be calculated using Eqs. (B.3.2-1) ~ (B.3.2-13).

$$N_{T\max} = \beta_T \alpha_C \frac{2\pi U_{hc}{}'}{}(EA)_{eq}^{C} \quad \text{(B.3.2-1)}$$

$$N_{C\max} = \beta_C \alpha_c \frac{2\pi U_{hc}{}'}{}(EA)_{eq}^{C} \quad \text{(B.3.2-2)}$$

$$\beta_T = \frac{(EA)_{eq}^{T}}{(EA)_{eq}^{C}} \frac{\alpha_T}{\alpha_C}\left\{1 - \frac{\cos(2\pi\eta/L')}{\cosh(\lambda_T \eta)}\right\} \quad \text{(B.3.2-3)}$$

$$\beta_C = 1 + \frac{\cos(2\pi\eta/L')}{cosh\{\lambda_c(L'/2 - \eta)\}} \quad \text{(B.3.2-4)}$$

$$\lambda_T = \sqrt{\frac{K_l}{(EA)_{eq}^{T}}} \quad \text{(B.3.2-5)}$$

$$\lambda_C = \sqrt{\frac{K_l}{(EA)_{eq}^{C}}} \quad \text{(B.3.2-6)}$$

$$\alpha_T = \frac{1}{1 + (\frac{2\pi}{\lambda_T L'})^2} \quad \text{(B.3.2-7)}$$

$$\alpha_C = \frac{1}{1 + \left(\frac{2\pi}{\lambda_C L'}\right)^2} \quad \text{(B.3.2-8)}$$

$$\frac{2\pi}{\lambda_T L'}\alpha_T \tanh(\lambda_T \eta) + \frac{2\pi}{\lambda_C L'}\alpha_C \tanh\left\{\lambda_C\left(\frac{L'}{2} - \eta\right)\right\} = (\alpha_T - \alpha_C)\tan\left(2\pi\frac{\eta}{L'}\right) \quad \text{(B.3.2-9)}$$

$$(EA)_{eq}^{C} = E_s A_s \quad \text{(B.3.2-10)}$$

$$(EA)_{eq}^{T} = \frac{E_s A_s}{1 + \frac{E_s A_s}{l_s K_J}} \quad \text{(B.3.2-11)}$$

$$L' = \sqrt{2}L \quad \text{(B.3.2-12)}$$

$$K_J = n \times k_j \tag{B.3.2-13}$$

where β_T ——coefficient of tensile axial force;

β_c ——coefficient of compressive axial force;

K_1—— stiffness of longitudinal ground spring per unit length of the structure in the longitudinal direction (kN/m);

η ——axial tension or compression range (m);

L′—— Seismic wave wavelength (m) along the tunnel axis at an angle of incidence of 45° with respect to the longitudinal axis of the tunnel, $L' = \sqrt{2}L$, where L is calculated by Eq. (B.2.2-3).

U'_{hc} ——component of maximum horizontal relative displacement (U_{hc}) of the ground at tunnel center in the axial direction of the tunnel at an angle of incidence of 45° with respect to the longitudinal axis of the tunnel, i. e. $U'_{hc} = U_{hc}/\sqrt{2}$, where U_{hc} is the value of Eq. (B.1.2-2) when z is taken as the depth of tunnel center, H_c;

E_s——Modulus of elasticity of the lining (kPa);

A_s——tunnel cross-sectional area (m^2);

$(EA)_{eq}^{C}$ ——equivalent compression stiffness of shield tunnels;

$(EA)_{eq}^{T}$ ——equivalent tensile stiffness of shield tunnels.

2 When the propagation direction of the seismic wave is consistent with the axial direction of the tunnel, i. e. $\Phi = 0$, the lining structure will generate the maximum bending moment. The maximum bending moment and correlation coefficient may be calculated using Eqs. (B.3.2-14) to (B.3.2-19).

$$M_{\max} = \alpha_M \frac{4\pi^2 U_{hc}}{L^2} (EI)_{eq} \tag{B.3.2-14}$$

$$\alpha_M = \frac{1}{1 + \left(\dfrac{2\pi}{\lambda_M L}\right)^4} \tag{B.3.2-15}$$

$$\lambda_M = \sqrt[4]{\frac{K_t}{(EI)_{eq}}} \tag{B.3.2-16}$$

$$(EI)_{eq} = \frac{\cos^3\varphi}{\cos\varphi + (\pi/2 + \varphi)\sin\varphi} E_s I_s \tag{B.3.2-17}$$

$$\varphi + \cot\varphi = \pi\left(0.5 + \frac{K_J}{E_s A_S / l_s}\right) \tag{B.3.2-18}$$

where α_M ——bending moment coefficient;

K_t—— stiffness of transverse ground spring per unit length of the structure in the longitudinal direction (kN/m);

I_s——moment of inertia of lining ring section (m^4), $I_s = \frac{\pi(D^4 - d^4)}{64}$, where D represents the outer diameter of the tunnel and d represents the inner diameter of the tunnel;

$(EI)_{eq}$——equivalent bending stiffness of shield tunnels;

K_J——tensile stiffness of bolts in tunnel cross section (kN/m);

K_j——tensile stiffness of single bolt (kN/m);

n——number of cross-sectional bolts;

h——width of lining ring (m);

Meanings of other symbols are the same as above.

B.4 Generalized response displacement method

B.4.1 When the tunnel structure is in non-homogeneous ground or acceleration time history at the engineering site is available and the calculation accuracy is required to be high, the generalized response displacement method shall be adopted.

B.4.2 When the generalized response displacement method is used in the cross-sectional direction of the tunnel, the actual displacement and shear stress response at the tunnel location in the free-field ground shall be first obtained based on the acceleration time history at the engineering site, and then the ground displacement and shear stress at the location of the structure at each instant shall be applied to the tunnel to calculate the transverse seismic internal force in the structure (Fig. B.4.2).

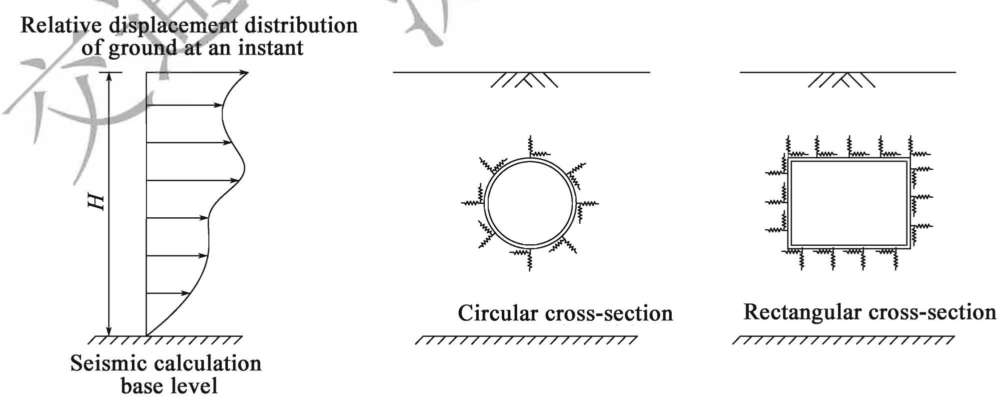


Fig. B.4.2 Calculation model based on generalized response displacement method (transverse)

B.4.3 When the generalized response displacement method is used in the longitudinal direction of the tunnel, the 3D (the location of the tunnel axis, time and ground displacement) time history response of free-field ground shall be first obtained according to the acceleration time history at the engineering site (Fig. B.4.3-1), and then the ground displacement time history at the location of the tunnel axis shall be applied to the end of the beam-spring model in the longitudinal direction to calculate the longitudinal seismic internal force in the structure (Fig. B.4.3-2).

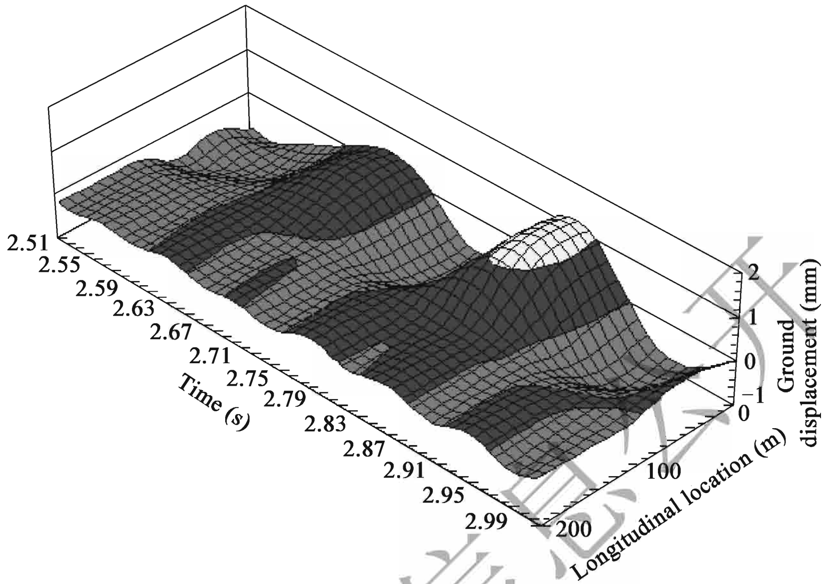


Fig. B.4.3-1 3D (the location of the tunnel axis, time and ground displacement) **time history response**

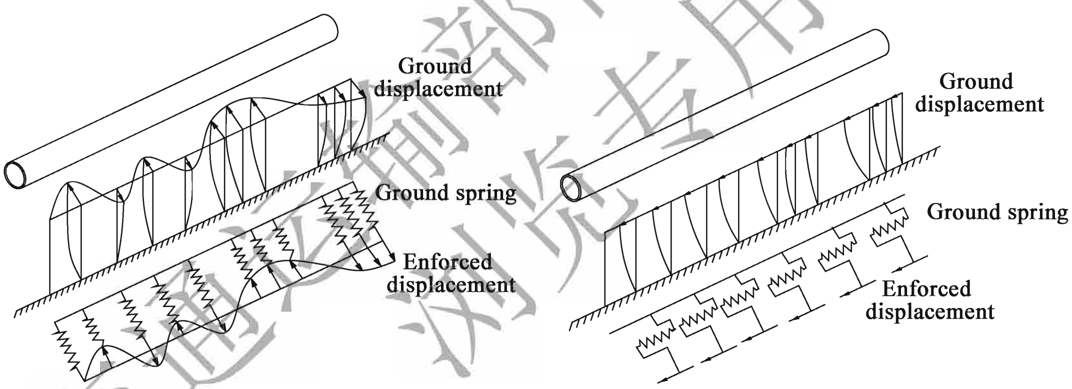


Fig. B.4.3-2 Calculation model based on generalized response displacement method (longitudinal)

Appendix C
Time History Analysis Method

C.0.1 The time history analysis method may be applied to the seismic calculation of tunnels under various topographical and geological conditions, with different structural types and constructed using different construction methods. When using the time history analysis method for calculation, the acceleration time history should be taken as the input ground motion, and the selection of acceleration time history may be made in accordance with the provisions in Section 5.4 herein.

C.0.2 When the structural type of the tunnel is continuous and regular in the longitudinal direction with the cross-sectional configuration unchanged, and the surrounding rock or soil is uniformly distributed along the longitudinal direction of the tunnel, only the seismic calculation in the cross-sectional direction may need to be carried out, and the calculation may be approximately treated as a plane strain problem.

C.0.3 In cases where the structural types change greatly, e.g. at the junction of the cross passage and the main tunnel and the interfaces of a shield tunnel with shaft or ventilation shaft, surrounding rock or soil conditions are non-homogeneous and topographical and geological conditions are complex, it shall be treated as a spatial problem solved by 3D modeling.

C.0.4 Viscous artificial boundary or viscoelastic artificial boundary should be used for model boundary.

C.0.5 Selection of the scope of the ground model shall follow the principles as below:

1 The distance from the structural side wall to the boundary in the horizontal direction is at least 3 times the structural width (Fig. C.0.5-1 to Fig. C.0.5-3).

2 The top surface boundary in the vertical direction should extend to the ground surface. When the tunnel is embedded at a great depth, the distance from the top of the structure to the ground surface should be 3-5 times the vertical effective height of the structure, and the influence of initial in-situ stress field should be considered (Fig. C.0.5-1 to Fig. C.0.5-3).

3 When the underground structure is relatively deep and the distance between the structure and the bedrock is less than 3 times the vertical effective height of the underground structure, the bottom boundary of the calculation model should extend to the bedrock surface (Fig. C.0.5-2).

4 When the underground structure is embedded into the bedrock, the bottom boundary of the calculation model shall extend below the bedrock surface (Fig. C.0.5-3).

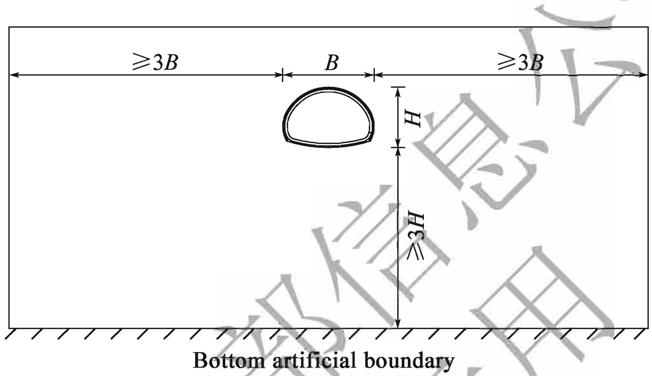


Fig. C.0.5-1 General artificial boundary conditions

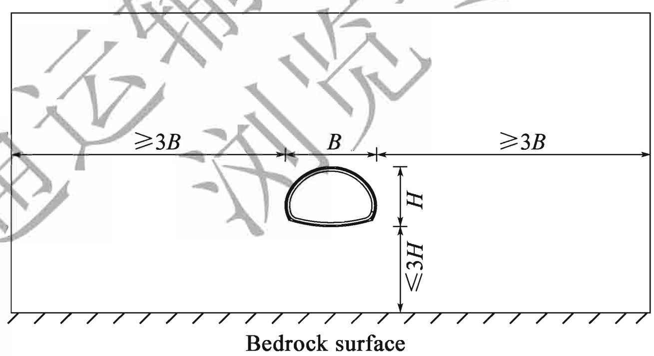


Fig. C.0.5-2 Calculation model for a relatively great depth

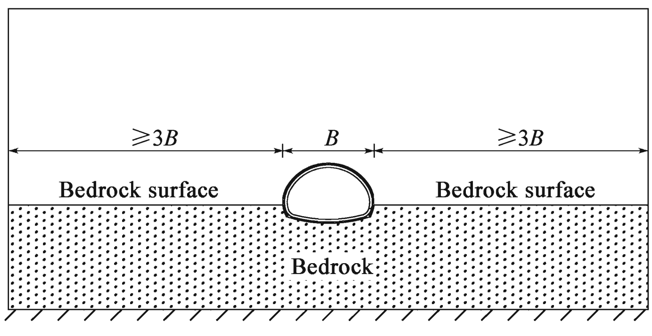


Fig. C.0.5-3 Calculation model of underground structure embedded in bedrock

C.0.6 Seismic wave selection shall meet the following requirements:

1 The number of selected seismic waves is generally not less than 3, and at least two sets of actual strong earthquake records and one set of artificial acceleration time history curves obtained from seismic safety evaluation may be selected according to the engineering site category and design earthquake.

2 When the number of seismic wave samples is less than 3, the envelope value of calculation results shall be taken for seismic design.

3 When the number of seismic wave samples is more than 7, the average value may be taken for seismic design.

4 The duration of ground motion input should be reasonably determined. The duration of ground motion acceleration may be taken as 5 ~ 10 times the basic natural vibration period of the structure, regardless of whether actual strong earthquake records or synthetic ground motion time histories are used.

Wording Explanation for the *Specifications*

1　The strictness in execution of theSpecifications 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 versions to be used. Otherwise they shall be the latest available versions.

Background to Provisions

1 General Provisions

1.0.1 In order to detail the provisions of the *Specification of Seismic Design for Highway Engineering* (JTG B02—2013) (hereinafter referred to as the "Seismic Specification") on the seismic design of highway tunnels and meet the needs for the seismic design of highway tunnels, the requirements and provisions on the seismic design of highway tunnels are compiled into a separate book and developed as these Specifications on the basis of drawing on seismic design methods of tunnels at home and abroad.

1.0.2 Various classes of highway tunnel include: expressway, Class-1, Class-2, Class-3 and Class-4 highway tunnels. Various types of highway tunnels include: drill-and-blast tunnels, shield tunnels, immersed tunnels and cut-and-cover tunnels.

1.0.3 For "the importance of highway tunnels and the difficulty of repair (emergency repair)", the tunnel design life, structural type, road network function, tunnel environment and other factors are considered. Meanwhile, it is necessary to meet the current *Standard for Classification of Seismic Protection of Building Constructions* (GB 50223) and be consistent with the seismic fortification category in the Specification of Seismic Design for Highway Engineering.

3 Basic Requirements

3.1 Seismic fortification category and seismic fortification standard

3.1.1 In order to meet the seismic fortification needs of highway tunnels in China and improve the operability of seismic design, more detailed provisions are made on the classification of seismic fortification importance of highway tunnels. In addition, from the experience and lessons of the Wenchuan earthquake, some low-class highway tunnels play an essential role in earthquake relief as lifeline projects in disaster areas, and their seismic fortification categories need to be increased.

3.1.3 earthquake, and no collapse due to large earthquake", the continuity and consistency with the seismic performance targets in the Seismic Specification and the requirements for tunnel seismic design targets at home and abroad, Class A tunnels shall not be damaged under E1 earthquake effect (return period of 475 years), and is allowed to experience limited damage under E2 earthquake effect (return period of about 2000 years), but shall maintain normal traffic after the earthquake. Class B tunnels shall not be damaged under E1 earthquake effect (return period of 75 years) and is allowed to experience limited damage under E2 earthquake effect (return period of about 1000 years), but shall maintain normal traffic after the earthquake. Class C tunnels shall not be damaged under E1 earthquake effect (return period of 50 years) and shall not collapse locally or completely under E2 earthquake effect (return period of 475 years). Class D tunnels shall not be damaged under E1 earthquake effect (return period of 30 years).

3.1.5 In order to further consider the plastic deformation performance requirements of the structure based on the elastic seismic design in one stage adopted in the Specification of Seismic Design for Highway Engineering, these Specifications include the second-stage elastic-plastic seismic design under the E2 earthquake effect, which is very important for immersed tunnels, shield tunnels and cut-and-cover tunnels. The fortification level under the E1 earthquake effect in the first stage is consistent with the Specification of Seismic Design for Highway Engineering, and this level is judged as reasonable by investigation and evaluation after the Wenchuan earthquake.

The design ground motion parameters specified in Specification of Seismic Design for Highway Engineering (JTJ 004-89) are adjusted by the "seismic importance coefficient" and "comprehensive influence coefficient". These Specifications integrate the above two coefficients into one coefficient, namely "seismic importance coefficient" herein. Only the "seismic importance coefficient" is used to adjust the design ground motion parameters, and the "comprehensive influence coefficient" is abolished.

For Classes B, C and D tunnels, the seismic importance coefficients corresponding to E1 earthquake effect are taken as 0.43, 0.34 and 0.26 respectively, and the corresponding design ground motion return periods are around 75, 50 and 30 years respectively. For Class A tunnels, in order to ensure a high seismic fortification level and referring to the seismic design of shield tunnels at home and abroad, cut-and-cover tunnels and the immersed tunnel of Hong Kong-Zhuhai-Macao Bridge, the seismic importance coefficient is taken as 1.0; the design ground motion is 10% probability of exceedance in 50 years, and the return period is about 475 years.

For Class B, C and D tunnels, the importance coefficients under E2 earthquake effect are taken as 1.7, 1.3 and 1.0 respectively, and the corresponding design ground motion return periods are roughly 2000, 1000 and 475 respectively. In recent years, the seismic fortification standards for large underwater tunnels at home and abroad typically adopt a 2% ~3% probability of exceedance in 50 years for rare earthquakes with a return period of about 1600-2400 years. However, the 2%-3% probability of exceedance in 100 years (corresponding to a return period of 1,000 years) is generally adopted for immersed tunnels. Therefore, these Specifications specify that for Class A tunnels, the importance coefficient of immersed tunnels is taken as 1.3 and that of other tunnels 1.7.

3.2 Earthquake effect

3.2.1 For tunnels which have performed a site-specific seismic safety evaluation, the "earthquake effects of any level" refers to earthquake effects with the same return periods as those corresponding to the seismic importance coefficient in Article 3.1.5 herein.

3.2.2 In general, the seismic design of highway tunnels does not require site-specific seismic safety evaluation, and the earthquake effect is determined according to the relevant provisions in Chapter 5 herein. For tunnels requiring seismic safety evaluation at engineering sites according to these Specifications and the current Evaluation of Seismic Safety for Engineering Sites (GB 17741), the probability of exceedance under the earthquake effect shall be determined based on the seismic fortification objectives of tunnel and the return period corresponding to the seismic importance coefficient in Table 3.1.5.

The seismic zonation map specifies the minimum requirements for seismic fortification. When the result of seismic safety evaluation is lower than the seismic fortification parameters specified in these requirements, the seismic fortification parameters specified in the zonation map shall be adopted to ensure safety.

3.2.3 The "design basic ground acceleration" is proposed according to the Notice on Design Value of Ground Motion Acceleration in Seismic Design Specifications (JB [1992] No. 419) from the former Ministry of Construction and called "basic seismic peak ground acceleration for Class II site" or A value for short. Table 3.2.3 herein directly refers to Tables G.1 and F.1 of the GB standard Seismic Ground Motion Parameters Zonation Map of China (GB18306-2015). The conversion between seismic peak ground acceleration for non-Class II sites and that for Class II sites shall be in accordance with Table E.1 of Seismic Ground Motion Parameters Zonation Map of China (GB 18306-2015).

3.3 Seismic design process

3.3.2 In order to ensure the seismic safety of the tunnel structure and reduce the calculation workload as much as possible, referring to the current relevant specifications at home and abroad, these Specifications stipulate: only seismic measures design may need to be performed for Class B, C and D tunnels in areas with a basic seismic peak ground acceleration classification of 0.05g. For Class D tunnels in areas where the basic seismic peak ground acceleration classification is above 0.30g (including 0.30g), only the seismic analysis and checking under E1 earthquake effect may need to be carried out, and the requirements for seismic measures shall be met. Class A tunnels shall be subject to seismic analysis and checking under E1 and E2 earthquake effects, and the requirements for seismic measures shall be met. In general, for Classes B and C tunnels, the requirement for seismic analysis and checking under E1 and E2 earthquake effects may be established on a project-specific basis, as specified in Chapters 9, 10, 11 and 12 herein.

4 Tunnel Site, Site and Ground

4.1 General requirements

4.1.2 According to the characteristics of the tunnel structure, this article gives the contents of investigation and evaluation of the site and ground for the seismic design of the tunnel project. Although these contents are different, they overlap each other. One or more of them shall be carried out according to the site conditions and project-specific conditions and requirements. When the time history analysis method is used for seismic calculation, soil profile and such parameters as dynamic shear modulus and damping ratio of soil shall be provided according to design requirements.

4.2 Tunnel location and site

4.2.1 This article specifies the relationship between the tunnel depth, the surrounding rock class, and the category of seismic sections. In general, tunnels in hard and intact rock mass are favorable for earthquake resistance, while tunnels in unfavorable geological zones are unfavorable for earthquake resistance. Deep tunnels are favorable for earthquake resistance, while shallow tunnels are unfavorable for earthquake resistance. Compared with the tunnel structure, tunnel portals and side and front slope are unfavorable to earthquake resistance. Therefore, considering the depth of the tunnel and the geological conditions of the tunnel, the seismic sections of the tunnel under different conditions are classified into 4 types: favorable, general, unfavorable and hazardous.

4.2.2 The classification into favorable, general, unfavorable and hazardous sections for earthquake resistance of tunnel structures is based on the relevant provisions of the Code for Seismic Design of Buildings (GB 50011—2010). According to the characteristics of tunnel portals, side and front slopes such as terrain, sloping ground and ground heterogeneity, tunnel sites are divided into favorable, general, unfavorable and hazardous sections.

4.2.4 The classification of soil is based on the provisions of Code for Seismic Design of Buildings (GB 50011—2010). The former hard soil or rock area is divided into two categories: rock site and hard site or soft rock, further refining the representation of soil type.

4.2.7 The engineering site classification method in the Code for Seismic Design of Buildings (GB 50011—2010) is adopted. The Code for Seismic Design of Buildings uses two parameters, namely the average shear wave velocity and the thickness of the overburden, as the evaluation criteria. This classification method has been generally recognized by the engineering community in China, and some issues and comments have arisen during its application. The main issue is that this classification method involves stepped variation and the variation in site category due to minor changes in the thickness of the overburden or the average shear wave velocity near the boundary line may lead to a large difference in the value of earthquake effect. In order to overcome the step-change in the site category, these Specifications permit determining the characteristic period value by interpolation method according to Table 5.4.2 provided that the shear wave velocity and overburden thickness data are reliable and near the boundary line of site category (within 15%).

4.2.8 The Code for Seismic Design of Buildings (GB 50011—2010) specifies the scope in which the impact of seismogenic fault dislocation on above-ground buildings may be neglected, and stipulates that when this impact needs to be considered, measures to avoid the main fault zone shall be taken in principle and the distance from the main fault zone is clearly specified. Considering the differences between tunnel structures and above-ground buildings, the distance from the main fault zone to immersed tunnels, cut-and-cover tunnels, tunnel portals and side and front slope structures may follow the provisions in the Code for Seismic Design of Buildings (GB 50011—2010). For deep and shallow tunnels, since concealed faults are related to the tunnel location, it is impossible to determine from the avoidance distance whether the influence of fault dislocation on the tunnel structure may be ignored.

In view of the particularity of tunnel structures, in many cases the tunnel site is determined according to the requirements of traffic planning. It is difficult to avoid main fault zones and special research and treatment are required. These Specifications do not give strict provisions on the influence of fault dislocation.

4.3 Ground

4.3.1 For the seismic bearing capacity of ground, reference is made to the relevant provisions of the Seismic Specification. In seismic check of natural ground, the provisions on adjustment coefficient of characteristic value of bearing capacity of subsoil mainly take into account two factors: the strength of subsoil under finite cyclic dynamic effects is generally higher than static strength, and the structural reliability under earthquake effect is allowed to decrease to a certain extent.

4.3.2 The so-called "static method" is generally used for seismic check of the ground. This method assumes that earthquake effect acts like static force, and then checks the bearing capacity and stability of the ground under this condition. The listed equations are mainly put forward by reference to the provisions of relevant specifications.

4.4 Soil liquefaction and soft ground

4.4.1 The provisions of this article are mainly based on the results from earthquake damage investigation at liquefied sites. Most data show that the earthquake damage caused by liquefaction to general tunnel structures is relatively minor in the area with seismic fortification ground motion classification of 0.05g, and liquefaction evaluation and treatment is not required. However, tunnel structures sensitive to liquefaction-induced ground subsidence (such as immersed tunnels, cut-and-cover tunnels, mountain tunnels and shallow sections at the portal of shield tunnels) may be evaluated and treated according to the requirements for the seismic fortification ground motion classification of 0.10g. Since the earthquake effect for Class A tunnel structures (including other especially important projects equivalent to Class A tunnel structures) shall be increased by one degree from the seismic fortification intensity in the region, and special research shall be conducted when the seismic fortification intensity is VIII or IX degrees, this article stipulates that Class A engineering structures shall be subject to special liquefaction investigation and treatment in areas where the seismic fortification ground motion classification is greater than or equal to 0.10g.

4.4.2 Saturated loose sand and saturated silt are liquefiable, which has been confirmed by the results of investigation into previous earthquake damages. Therefore, ground with saturated loose sand and saturated silt shall be subject to liquefaction evaluation except for the area where the seismic fortification ground motion classification is 0.05g.

Gravel-bearing sandy soil, silty clay and silty sand interbed and sandy soil have the potential for liquefaction, but available research on its liquefaction property is not sufficient. It is inappropriate to treat their liquefaction problem as sandy soil or silt, and special research should be conducted.

4.4.3 The preliminary liquefaction assessment method is referenced from the Code for Seismic Design of Buildings (GB 50011—2010). This method is based on the statistical analysis of the liquefaction of soil layers with liquefaction potential in the earthquake zones of major earthquakes since the founding of the People's Republic of China. At the same time, it also draws on foreign research results and experiences.

The macroscopic investigation of soil liquefaction in Tangshan earthquake area shows that the epicenter area was Luanhe River second terrace, the stratigraphic chronology was Late Pleistocene (Q_3), the groundwater table was 3-4mand the surface layer was about 3.0m clayey material underlaid by saturated sand layer, which experienced no liquefaction under the earthquake intensity X. However, the strata with relatively new geological age distributed in the first terrace and high floodplain experienced large-area liquefaction, although the earthquake intensity was only VII and VIII.

No liquefaction cases were found in fluvial alluvial formation of older geological age in other earthquake regions. The research by foreign scholars Youd and Perkins shows that saturated and loose hydraulic fill is basically liquefiable; Holocene cohesionless soil is also vulnerable to liquefaction; the occurrence of liquefaction in Pleistocene is very rare; and the occurrence of liquefaction in Pre-Pleistocene is even rarer. These conclusions were given based on the worldwide earthquake-induced liquefaction data before 1975 and later confirmed by the two earthquakes in Japan in 1978 and those in Romania in 1977.

Laboratory tests show that the liquefaction strength of soil increases with fines content. The on-site survey data on Haicheng and Tangshan earthquakes also show that when the fines content reaches a certain value, liquefaction rarely occurs. Therefore, it is stipulated that in areas where peak ground acceleration is 0.10g (0.15g), 0.20g (0.30g) and 0.40g and the fines content of silt (particles less than 0.005mm in size) is not less than 10%, 13% and 16% respectively, the liquefaction potential may be ruled out. The fines content is determined using sodium hexametaphosphate as dispersant. If other dispersants or other particle analysis methods are used, conversion shall be made according to relevant specifications.

The use of the thickness of the overlying non-liquefiable soil layer and the depth to groundwater table as the limit value for preliminary liquefaction assessment is determined according to the survey results from Tangshan, Haicheng and Niigata (Japan) earthquake areas and considering safety factor. Previous earthquake damage investigations show that there are many examples of liquefaction when the groundwater table is high. When the groundwater table is low or there is a thick non-liquefiable overburden at ground surface, even if the underlying liquefiable soil layer is liquefied, the ground will not suffer a large amount of subsidence or differential settlement because the highly effective pressure from the overburden may inhibit the liquefiable soil from gushing out of the ground.

4.4.4 This article mainly gives a method to further evaluate site potential for earthquake-induced liquefaction.

Investigations of soil liquefaction at the site of the Wenchuan earthquake found that liquefaction-induced water gush reached more than 10m high in 4 villages in different regions, and confirmed the authenticity of soil liquefaction within 20m depth. In the past major earthquakes, it was also found that the silty sand layer 15-20 m below the ground surface might experience liquefaction. In addition, considering the particularity of the tunnel structure, the overburden above the tunnel portal and shallow tunnel is thin. Therefore, liquefaction of the soil layer within 15-20m below the ground surface may cause serious damage or float to the underground tunnel portal and tunnel structure. It is very necessary to assess the potential for liquefaction of the soil layer 20 m below the ground surface. Therefore, the depth of the liquefaction discrimination in this article refers to the provisions of the Seismic Specification, that is, saturated sand and silt with a depth of 20m below the ground should be judged by the standard penetration test method.

Current research on liquefaction of deep soil at more than 20m depths is not adequate. When the underside of the tunnel structure is deeper than 20m, it is necessary to carry out special research on the liquefaction of the deep soil layer.

For further assessment, various methods should be adopted for analysis, comparison and judgment. Other liquefaction evaluation methods, if well established and proven, may also be used. Representative methods include:

(1) NCEER method: Seed simplified method modified by Youd, et al. which is currently widely accepted in other countries.

(2) Probabilistic liquefaction evaluation method: a probabilistic liquefaction evaluation method with peak ground acceleration as criterion developed by Chen Guoxing et al. (2005) based on the measured data on 344 sites in 25 major earthquakes at home and abroad.

(3) Static penetration test identification method: this method has been incorporated into the Code for Seismic Design of Railway Engineering (GB 50111—2006).

(4) Shear wave velocity identification method

(5) Dynamic triaxial test identification method.

4.4.5 The liquefaction classification method is referenced from the Seismic Specification. The liquefaction classification provides a simple method for estimation of liquefaction hazard by roughly assessing the degree of sand and water gushing at the site and the possible damage to underground structures and the foundation of aboveground structures based on liquefaction grade. Liquefaction grades include slight, moderate and serious. Based on the data on more than 100 seismic liquefaction hazards in China, sand and water gush from ground surface and levels of hazard to aboveground structures under various liquefaction grades (the evaluated depth is 15m) are shown in Table 4-1.

Table 4-1 Liquefaction grade and corresponding level of hazard to the structure

Liquefaction grade	Liquefaction index (15m)	Sand and water gush from ground surface	Level of hazard to the structure
Slight	<5	No sand and water gush from ground surface, or sporadic sand and water gush points in depressions or near the river	Low level of hazard, generally no significant damage
Moderate	5 ~ 15	High possibility of sand and water gush from slight to serious, mostly moderate	Relatively high level of hazard, likely to cause differential settlement up to 200mm and cracking

continued

Liquefaction grade	Liquefaction index (15m)	Sand and water gush from ground surface	Level of hazard to the structure
Serious	>15	Generally serious sand and water gush with obvious ground surface deformation	High level of hazard, differential settlement likely to exceed 200mm, structures with high center of gravity likely to experience unallowable tilt

4.4.7 Measures against liquefaction involve comprehensive treatment of liquefiable ground. Pay attention to the following points:

(1) For a soil layer judged as liquefiable, if measures against liquefaction are taken, it is not necessary to modify the soil parameters according to its degree of liquefaction, because the soil has been treated against liquefaction and would not be liquefied, and its soil property parameters are no longer those of liquefiable soil.

(2) This article specifies that any untreated liquefiable soil layer should not be used as the bearing stratum of natural ground. Both theoretical analysis and shaking table tests have proved that the main hazards of liquefaction come from the outer side of the foundation. The part located directly below the foundation within the liquefiable bearing stratum is the most difficult part to liquefy. The influence of initially liquefied area on the non-liquefied part directly below the foundation makes it lose the support of lateral earth pressure. The slightly liquefiable soil layer may be used as the bearing stratum of the foundation provided that the influence of the liquefiable area on the outer side is under control. In addition, the investigation of earthquake damage and finite element analysis show that when the ratio of foundation width to liquefiable layer thickness is greater than 3, the liquefaction-induced ground subsidence will not exceed 1% of the liquefiable layer thickness to cause serious structural damage. Therefore, it is not absolutely banned to use lightly and moderately liquefiable soil layers as the bearing stratum, subject to rigorous evaluation and validation.

(3) Soil liquefaction on sloping ground often leads to large-area soil mass sliding, resulting in serious consequences. However, soil liquefaction on the level ground generally only results in differential settlement and tilt of structures. This article does not apply to sloping ground with a slope greater than 10° or highly non-homogeneous liquefiable soil layers.

(4) Sites with slight liquefaction grade generally require no special treatment, except for Classes A and B structures which need to ensure safety due to their importance, because sand and water gushes may be absent at such sites, and even if they do occur, they will not cause serious damage to the structures.

(5) For sites with moderate liquefaction grade, structural measures that are easy to implement for foundation and structure treatment shall be considered preferably, and stabilization of the liquefiable soil layer is not always necessary.

(6) In the case of a thick liquefiable layer, measures to partially eliminate liquefaction-induced ground subsidence or floatation can leave part of the liquefiable layer without treating to the lower

boundary of this layer.

4.4.12 Soil parameters measured in geotechnical tests are obtained under certain loading conditions, which may not completely conform to the real loading conditions of structures. For example, ground consisting of saturated loose sand and soil will lose its bearing capacity due to liquefaction during earthquake. Therefore, this article specifies that for a soil layer judged as liquefiable, its soil parameters shall be modified according to its degree of liquefaction.

4.4.13 When determining the reduction coefficient of soil liquefaction impact, if another proven method of liquefaction evaluation is used, R and R_{cr} in the calculation formula of liquefaction resistance rate are respectively the liquefaction resistance strength and seismic effect appropriate to this method. For example, when the Seed simplified method is used for evaluation, R is the anti-liquefaction shear stress ratio (anti-liquefaction shear strain amplitude) of the soil layer, and R_{cr} is the dynamic shear stress ratio (dynamic shear strain amplitude) at the depth where the soil layer is located.

5 Earthquake Effect

5.1 General requirements

5.1.2 For different types of tunnel structures, the consequences of earthquake damage vary, especially the severity of secondary disasters, the degree of influence on lifeline network connectivity and post-earthquake emergency rescue work, and the level of repair difficulty. In order to effectively reduce the earthquake damage to the project and its social impact, different requirements need to be imposed on the determination of seismic design ground motion parameters of different types of engineering structures.

For Class A tunnels and extra-long tunnels located in areas with seismic fortification ground motion classification of 0.40g, ground motion parameters based on site-specific seismic, geological setting and engineering geological conditions need to be used in seismic design.

In general, for Classes B and C structures, the ground motion parameters used in seismic design may be determined according to these Specifications. In cases where the site-specific seismic safety evaluation has been carried out, approved ground motion parameters shall also be adopted for seismic design but shall not be lower than the design ground motion parameters specified in these Specifications.

The results provided by seismic safety evaluation or special research work shall meet the requirements for seismic design, such as horizontal peak ground acceleration and acceleration response spectrum of each major control point along the route at the ground surface, underground design depth, and bedrock surface; vertical peak ground acceleration and acceleration response spectrum; ground surface peak displacement; changes in peak ground acceleration and displacement with depth; as well as the ground motion acceleration time history compatible with the peak ground acceleration, acceleration response spectrum, and peak displacement at the ground surface, underground design depth, and bedrock surface.

5.1.3 If there is an active fault within the engineering site and nearby areas which may generate strong earthquakes, this fault and the strong earthquakes that may occur in the future will form the spectrum features of near-fault ground motions at the engineering site and cause serious seismological disasters, such as fault rupture at or near the ground surface. In order to fully account for the impact of this seismological environment, site-specific seismic safety evaluation is required. Meanwhile, it is necessary to reasonably consider characteristics of the near-fault ground motion caused by strong earthquakes based on the site-specific seismic safety evaluation results, including the vertical ground motion characteristics near the fault.

5.2 Horizontal earthquake effect

5.2.1 Ground motion observation data and relevant research show that there are obvious differences in the amplification of ground motion at different types of sites, including different levels of changes in peak ground acceleration and displacement. Considering the above factors, these Specifications select different adjustment coefficients Cs (Table 5.2.1) according to the specific site category and the peak ground acceleration of Class II site, for determination of the design peak ground acceleration using Eqs. (5.2.1-1) and (5.2.1-2).

5.2.2 This article gives the provisions on the value of horizontal seismic peak displacement at the site, which is obtained by transforming the relationship between seismic peak displacement and peak ground acceleration. Based on the research results at home and abroad, the ratio of horizontal peak displacement (m) to peak ground acceleration (m/s^2) at Class II sites may be taken as 1:15. The seismic peak displacement at other sites needs further adjustment by applying an appropriate adjustment coefficient F_u selected from Table 5.2.2 based on the site category and seismic peak displacement of Class II sites.

5.2.3 Ground motion parameters at the ground surface are generally larger than those below the ground surface, and the underground ground motion input may be taken by correspondingly reducing the earthquake effect at the ground surface according to depth. When the ground surface, soil layer interface and bedrock surface are relatively flat, one-dimensional (1D) shear soil layer model may be adopted for determination. When the soil layer interface, bedrock surface or ground surface fluctuates considerably, two-dimensional (2D) or 3D site model should be used for determination.

According to results from the analysis of a large number of seismic safety assessment data on engineering sites all over China, the seismic peak displacement at the site decreases as soil depth increases, but its variation is characterized by significant discreteness, with the seismic peak displacement at 30m depth in the soil layer mostly deceased by between 1/3 and 1/2. Therefore, some studies have shown that the seismic peak displacement at a depth of 50m at the site is 1/2 of the peak free-field ground displacement at the site, and the seismic peak displacement decreases linearly as soil depth increases. The above research results are all based on the pattern of variations in ground motion with the depth in the soil layer. Most of the highway tunnels are typically located in mountainous areas with topographic relief, and the topography also has a great influence on the ground motion. The Code for Seismic Design of Buildings (GB 50011—2010) addresses the impact of topographical effect of ground motions based on the macro seismic disaster experience and seismic response analysis results by specifying a mandatory provision that the horizontal earthquake effect coefficient shall be multiplied by an enhancement coefficient after comprehensive judgment and consideration. The enhancement coefficient is 1.1-1.6 depending on the specific case. Based on the above considerations, in the absence of recognized quantitative result for the change patterns of ground motion in rock strata and the influence of fluctuating topography on ground motion, the qualitative stipulation is given that when the response displacement method is adopted, the earthquake effect input at the bottom boundary of the calculation model may generally be ground surface earthquake effect.

5.3 Vertical earthquake effect

5.3.1 The ratio of the vertical peak ground acceleration to the horizontal one at the site is related to the seismic environment. The ratio near a fault may approach or reach 1.0, but it will decrease as epicentral distance increases. These Specifications established how to take the value of the ratio of the vertical peak ground acceleration to the horizontal one considering the impact of the seismic environment. For the sake of safety, the value of the vertical peak ground acceleration at the site shall not be less than 0.65 times the horizontal peak ground acceleration.

5.4 Design ground motion time histories

5.4.2 "The time history may be synthesized based on the ground motion acceleration response spectrum" using trigonometric series method. "The time history may be generated from actual ground motion acceleration records from similar seismic and site environments after appropriate adjustment" involves multiplying the time coordinate t and acceleration coordinate α of the appropriate actual ground motion acceleration records by appropriate constants respectively to make the acceleration time-course approach various requirements. This is called the proportional method, and it is usually difficult to meet the error requirements of Article 5.4.2 herein. The trigonometric series method generally assumes a uniformly and randomly distributed phase spectrum of ground motion, without considering the phase distribution of ground motion related to the seismic environment. These Specifications recommend that the dynamic acceleration time history recorded during strong earthquakes be used as the initial time history to synthesize the ground motion time history suitable for the engineering site, thus incorporating information on the real phase of ground motion. "The time history may be generated from actual ground motion acceleration records from similar seismic and site environments after appropriate adjustment" also has this meaning.

6 Calculation Method

6.1 General requirements

6.1.2 Seismic calculation methods commonly used for tunnels at present mainly include static method, response displacement method and time history analysis method. Different calculation methods have their respective characteristics and applicability. The selection of calculation methods shall be based on a combination of such factors as tunnel importance, seismic fortification category, seismic performance requirements, cross-sectional shape, structural characteristics, engineering geological conditions in tunnel site area and input ground motion parameters.

The static method requires parameters that are easy to determine, has been applied in many projects, and is easy to be accepted by design engineers. China's relevant codes including tunnel seismic design content, such as Specification of Seismic Design for Highway Engineering (JTG B02), Code for Seismic Design of Railway Engineering (GB 50111), and Technical Manual for Railway Engineering Design (Tunnel), almost all adopt this method. However, when the depth is very small, the internal lining force calculated by the static method is relatively small, whereas when the depth of the tunnel increases to a certain value, the lining bending moment calculated by the static method under earthquake effect increases sharply, which is inconsistent with the earthquake damage and seismic response mechanism of the underground structure. The modified static method leads to more reasonable calculation results by using the modified calculated height of the overlying soil column. Therefore, the modified static method should be adopted in the seismic calculation of homogeneous rock tunnels except for cut-and-cover tunnels, shed tunnels and end-wall tunnel portals.

The response displacement method is widely used in seismic calculation of underground structures in

Japan. It can reflect the characteristics of tunnel vibration with strata and has a clear concept and requires a small workload of calculation. The time history analysis method can consider the peak value, spectrum feature and duration of ground motion, deal with inhomogeneity, anisotropy, nonlinear and complex geometric boundary conditions in the medium, and fully reveal the response law of tunnels under earthquake effects. However, the dynamic influence of surrounding rock medium on the structure is coupled in both time and space, the dynamic analysis is complicated, and the solution cost is very high, so it is still difficult to be applied by practitioners. Therefore, under general conditions, the response displacement method or modified static method is recommended for seismic calculation. When the tunnel is located at a non-uniform site, the calculation accuracy is required to be high, and the site design ground motion time history is available, the generalized response displacement method is recommended for seismic calculation. For important projects and complex structures, which require full consideration of the peak value, spectral characteristics and duration of ground motions to obtain the internal force and displacement responses of ground and structures throughout the duration of earthquake, the time history analysis method is recommended for seismic calculation.

6.2 Calculation requirements

6.2.1 In general, the tunnel is characterized by large longitudinal length and basically unchanged transverse configuration and structure. According to its structural characteristics and plane strain principle, one or more cross-sections are selected along the longitudinal direction of the tunnel for transverse seismic calculation. Representative cross-sections e. g. those with shallow overburden, eccentric load, major changes in water level or poor geotechnical mechanical properties are generally selected.

When the tunnel longitudinally passes through areas with complex topography and major changes in engineering geological conditions, such as the approach section or large fault fracture zones and the interfaces of soft/hard rock formations, and tunnel structures with joints in the longitudinal direction, such as shield and immersed tunnels, the structure may generate relatively complex internal force responses in the longitudinal direction under the earthquake effect, causing structural damage or affecting its normal use. Particular consideration needs to be given to the longitudinal seismic performance of such tunnels.

Time history analysis method can deal with the inhomogeneity, anisotropy, nonlinear and complex geometric boundary conditions of the medium, take into account the peak value, spectral characteristics and duration of the ground motion, and reveal the dynamic response characteristics of the tunnel structure and surrounding rock and soil mass throughout the earthquake period. Therefore, it is especially suitable for large-span, important, complex and special tunnel structures or local sections with major changes in topographic and geological conditions or tunnel structures with major changes in longitudinal structural types and significant spatial effects.

6.2.2 The boundary conditions of the calculation model and the earthquake effect shall be appropriate to the selected method. The static method adopts the load-structure model without the need to consider the boundary of ground. The earthquake effect is represented by the peak ground acceleration. The calculated depth of the tunnel is the major consideration. The inertia of the overlying soil column within the equivalent calculated depth is applied to the tunnel structure for calculation. The specific value may be determined according to the provisions of Table A.1.5 in Appendix A herein.

In the response displacement method, the earthquake effect is represented by relative displacement and shear stress of ground and is determined by ground surface peak displacement and surface stratum thickness. In the generalized response displacement method, the dynamic response of free-field ground needs to be solved first, with due attention to the selection of calculation boundary, especially the determination of design earthquake effect base level. The specific provisions for the selection of design earthquake effect base level are given in the provisions of these Specifications.

In the calculation by time history analysis method, the model elements suitable for tunnel structure and component characteristics are selected. The calculation range and boundary conditions of the model shall meet the required calculation accuracy. In order to prevent seismic wave reflection at the model boundary, the boundary conditions that reduce seismic wave reflection from the boundary are preferred.

6.2.3 According to experience, when the load-structure model is adopted, the lining is modeled using beam elements, and the ground spring is modeled using spring elements or bar elements. When the stratum-structure model is adopted, the lining is modeled using beam or shell elements, and the stratum is modeled using plane or solid elements. When the slippage between the structure and the ground is considered, contact elements may be arranged between the structural elements and the ground elements. In general, static parameters of materials are used for static calculation. Dynamic parameters of the material are used for dynamic calculation. When the strain rate of the material is small, the static parameters of the material may also be approximately adopted for calculation. The commonly used constitutive models include elastic constitutive model, elastic-plastic constitutive model and viscoelastic (plastic) constitutive model. The selection of the constitutive model shall be based on the working behavior of structures and ground under different levels of earthquake effects.

7 Materials and Parameters

7.1 General requirements

7.1.1 Seismic materials shall meet the durability requirements of the main structure of the tunnel, so as to avoid failure and damage of seismic materials prior to failure of the main structure of the tunnel, loss of their own seismic function and even jeopardizing the safety of the main structure of the tunnel. Their seismic performance as well as economic rationality shall be considered in selection of seismic materials.

7.1.2 Based on the seismic research and design investigation on tunnel projects at home and abroad, as well as the investigation and mechanism analysis of the earthquake damage of the "5.12" Wenchuan earthquake to tunnels, the structure of the approach section of a mountain tunnel is mainly damaged due to the influence of earthquake inertia; the side and front slope cracks and collapses; and the tunnel liner is damaged to varying degrees as surrounding rock deforms due to the constraint force of surrounding rock deformation. Therefore, materials with different properties shall be reasonably adopted depending on specific earthquake damage mechanisms. For the lining material of the tunnel proper, high-grade concrete is adopted. Steel fiber, polyester fiber and the like may be added to the reinforced concrete and lining material to improve the ductility, flexural resistance, tensile resistance and toughness of the concrete and to enhance the seismic performance of the tunnel structure. Reinforced concrete structure is often adopted for tunnel sections in poor geological conditions. Light materials are used for auxiliary members including portal walls, in-tunnel decoration materials and duct partitions, to reduce the mass of the structure itself and thereby reduce the earthquake inertia, preventing damages or material falls caused by earthquakes from affecting traffic safety.

7.2 Material selection

7.2.2 Provisions on mountain tunnel lining and cut-and-cover tunnel construction materials are minimum requirements determined based on the characteristics of the structure itself while ensuring to meet the stress requirements as well as its economy and functionality. These minimum requirements may be raised where appropriate based on the specific conditions of the project such as surrounding rock class. Fiber materials should be added to lining materials for three-lane tunnels of seismic measures level 4 to improve the seismic performance of structures. The survey of tunnel distresses after earthquakes in China found that few tunnel distresses occur for deep sections and those in good geological conditions, so the minimum requirements are specified largely for seismic fortification sections.

7.2.3 At present, there are many kinds of materials and cross-sectional shapes used for shield tunnel segments and immersed tunnel elements, but concrete segments (elements) are the most commonly used. The provisions for concrete strength in this article are mainly based on the construction methods and structural characteristics of these two types of tunnels, and may be appropriately raised in design on a project-specific basis.

7.2.4 This article is referred from the Code for Seismic Design of Buildings (GB 50011—2010), taking into account the current research experiences in seismic design of tunnels.

7.3 Material properties

7.3.1 This article is mainly referenced from the Reinforcement Concrete-Part 2: Hot-rolled Ribbed Steel Bars (GB/T 1499.2—2018) and Code for Seismic Design of Buildings (GB 50011—2010), taking into account the current research experience in seismic design of tunnels.

7.3.2 This article is mainly referenced from Articles 3.3.3 and 3.3.4 of Technical specification for Fiber Reinforced Concrete Structure (CECS 38:2004) and Articles 10.3.2-10.3.5 of Specification for Design of Hydraulic Tunnel (DL/T5195—2004). Under the effect of earthquake force, in order to accommodate repeated vibration and deformation of tunnel structure, it is an effective measure to add 3%-6% steel fiber into concrete. The measured data show that the tensile strength and bending strength of the concrete may be increased by 30% ~60% and 30% ~90%

respectively by adding a proper amount of steel fiber with diameter (0.3-0.5) mm and strength not lower than 380MPa.

7.4 Physical and mechanical parameters of materials

7.4.3 Relevant concrete tests show that the dynamic strength of the concrete increases by about 20% compared with its static strength under the rapid loading conditions corresponding to earthquake effect. Available tests from domestic and abroad show that the dynamic and static elastic moduli of concrete are not much different from each other. Since the static modulus of elasticity consider the creep effect under long-term loads, the dynamic modulus of elasticity may be 30% higher than its static value.

8 Seismic check

8.1 General requirements

8.1.1 Drill-and-blast tunnels, shield tunnels, immersed tunnels and cut-and-cover tunnels are quite different in structural types. It is inappropriate to adopt the same model for seismic calculation and checking for each type of tunnels. Instead, each type requires a different seismic calculation and checking method depending on its structural characteristics.

8.1.2 Determine the content and criteria for strength and deformation checking according to the seismic performance requirements. Seismic check requires the determination of the appropriate target performance after the seismic performance requirements are determined. At present, the main indicators selected include stress level (strength), functionality (amount of deformation, crack width, joint opening, etc.), and tunnel stability. The specific permissible indicators shall be determined considering the importance of the structure, earthquake effect level, structural type and surrounding rock conditions, etc.

8.3 Deformation checking

8.3.3 According to national codes of various countries, earthquake damage experience, experimental research results and analysis of engineering examples and consulting the code for seismic design of aboveground structures, it is reasonable and feasible to use the interlayer displacement angle as the criterion to measure the structural deformation capacity of the rectangular cross-section structure and thus to judge whether it meets the structural functionality requirements.

In accordance with the Code for Seismic Design of Buildings (GB 50111—2010), the Code for Seismic Design of Subway Structure (DG/TJ08—2064—2009) and the latest research results, the limit value of interlayer displacement angle for underground reinforced concrete structures is taken as 1/550 for elasticity, 1/250 for yield point and 1/80 for Performance Requirements 3.

For drill-and-blast tunnel structures, a comprehensive and systematic seismic response analysis of arched (horseshoe) highway tunnels was carried out as part of the effort to develop these Specifications. Based on the elastic-plastic damage constitutive model of concrete, a large number of numerical calculations were carried out for the stiffness, depth and structural stiffness of different rock and soil masses; the seismic capacity curve of tunnel structure based on damage degree was obtained; and its corresponding relationship with earthquake damage was studied. Based on studying the response sensitivity and performance criterion characteristics of the structure, the maximum deformation rate (i. e. the maximum convergence) of the tunnel was selected as the seismic performance criterion. On the typical seismic capacity curve, three performance levels were studied, i. e. intact structure, slight damage and relatively serious damage. Considering the influences of design, construction and maintenance, and through statistical analysis, the final performance criterion values are recommended: for Performance level 2, the maximum convergence is 5 ‰; and for Performance level 3, the maximum convergence is 15‰.

Circular section structures are mostly shield tunnels. Japan uses the ratio of the maximum relative displacement of the ground between the crown and invert of the shield tunnel structure to the outer diameter of the tunnel, i. e. the diameter deformation ratio, as the checking criterion. According to notes to the provisions of the Code for Design of Metro (GB 50157—2013), the diameter deformation ratio of shield tunnels shall be limited to 3 ‰ to 6 ‰ based on experience in the engineering practice. The Code for Seismic Design of Subway Structure (DG/TJ08—2064—2009) stipulates that the maximum diameter deformation ratio under equivalent structural performance 1 shall not exceed the permissible value determined by the safe use of joint waterproofing materials. Then the assumed diameter deformation ratio is the deformation limit of shield tunnel structure at the elastic design stage. Performance Requirements 2 herein only allow slight structural damage and prohibit the failure of joint waterproofing. Therefore, it is appropriate to take the diameter deformation ratio of 6‰ as its overall checking limit.

The Performance Requirements 3 herein allow damages to the structure, but do not allow local block falling or collapse, or waterproofing failure. Literature research found that the maximum diameter deformation ratio of the subway shield tunnel structure at yield is about 10‰, while the diameter deformation ratio is over 25‰ when the structure reaches the maximum bearing capacity. Due to a lack of structural test data on large-diameter two-lane and three-lane shield tunnels, considering the size effect of subway shield tunnels and large-diameter road shield tunnels, the corresponding diameter deformation ratio of road tunnels should be smaller. If joint waterproofing is allowed to fail, the limit value may be taken as 3-4 times the elastic limit value, i.e. 18‰-24‰. If joint waterproofing is not allowed to fail, the limit value is controlled by the diameter deformation ratio when the joint waterproofing of shield tunnels fails. This value is generally greater than 12‰. It should be noted that in the case of Performance Requirements 2 and 3, the material has entered the elastic-plastic stage, and its dynamic mechanical parameters need to be increased in calculation, especially when performing deformation checking, its dynamic modulus of elasticity is often increased to 3-5 times the static modulus of elasticity.

8.3.4 The deformation capacity, ultimate bearing capacity and waterproof capacity of various connections of the tunnel need to be fully considered in longitudinal seismic check. The deformation performance and waterproofing performance of expansion joints and other connections shall be properly controlled based on tests according to materials and construction factors, for reasonable modeling and parameter setting.

8.5 Checking of stability against buoyancy

8.5.2 If liquefaction mitigation measures such as grouting and soil replacement have been taken to treat liquefiable ground, the lateral frictional resistance of the soil layer to the lining structure is calculated according to the reduction coefficient of liquefaction effect, which is determined by the measured liquefaction strength ratio of the stabilized ground.

8.5.3 There is no uniform specification on the anti-floating safety factor of tunnels. This article determines this safety factor in accordance with relevant codes and engineering practice. At present, different codes provide different provisions on checking of stability against buoyancy. The Code for Design of Metro (GB 50157—2013) specifies that the anti-floating safety factor shall not be less than 1.05 when the lateral frictional resistance of ground is excluded, and shall be 1.10 ~ 1.15 when the lateral frictional resistance of ground is included, depending on the geological and hydrogeological conditions in different regions. The Structural Design Code for Pipelines of Water Supply and Waste Water Engineering (GB 50332—2002) stipulates that for pipelines buried below surface water or groundwater level, the stability against buoyancy of pipeline structures shall be calculated according to design conditions. All effects shall be taken as standard values during calculation, and the resistance coefficient of stability against buoyancy shall not be less than 1.10.

As the earthquake effect increases, the safety reserve provided by the side wall friction of a tunnel in the liquefiable soil layer decreases. According to different seismic fortification objectives, as well as relevant codes and engineering examples, these Specifications recommend the use of anti-floating safety factors appropriate to the seismic fortification objectives as the design minimum limit.

9 Drill-and-Blast Tunnels

9.1 General requirements

9.1.1 According to the investigation on a large amount of earthquake damage data, it is found that the depth of the tunnel has a great influence on the level of earthquake damage. When the depth of the tunnel is greater than 50m, the level of damage to the tunnel is considerably lower, and the tunnel deeper than 300m seldom suffers serious damage.

Roads in mountainous and hilly areas often encounter the junctions between bridges and tunnels. Because the dynamic characteristics of the two structures are significantly different, when an earthquake occurs, it may cause serious damage to the bridge or tunnel structure. For example, the entrance of the Shaohuoping Tunnel on the Wolong link to Dujiangyan-Yingxiu Expressway is connected with the Minjiang River Bridge. During the "5.12" Wenchuan earthquake, the tunnel portal was dislocated up to 40cm from the abutment, and both the tunnel and the bridge were seriously damaged, especially the bridge, where the beam was rotated and displaced horizontally. Therefore, for high seismic intensity areas, reasonable route selection, structural optimization and other measures may be taken to avoid connecting tunnel and bridge wherever possible.

9.1.2 Many studies show that twin-arch tunnels are more susceptible to damage than other kind of tunnels in earthquakes due to their special structural types. Under earthquake load effects, the middle wall is repeatedly subjected to tension and compression. Shear failure may occur at the joint between the middle wall and the main structure, and it is difficult to repair after failure. In serious cases, the tunnel may even collapse.

9.1.3 Based on results from the investigation on earthquake damage at home and abroad, the earthquake damage is relatively more serious for tunnels in high seismic intensity areas or tunnels crossing active faults. This is especially true for tunnels crossing active faults, because the relative movement of the hanging and heading sides of faults leads to a large amount of structural dislocations, which may encroach upon the tunnel structure gauge and increase the difficulty of post-earthquake repair. Therefore, this article specifies the need to appropriately increase the size of intrados.

9.3 Seismic check

9.3.1 Some scholars have investigated the damage to underground structures caused by 85 earthquakes worldwide. They found that the degree of damage to the structure decreases with the increase of overburden thickness, and the damage to underground structures in soft rock or poor surrounding rock conditions is more serious than those in hard rock conditions. The investigation on the earthquake damage to 46 mountain tunnels due to the "9.21" Taiwan Chi-Chi earthquake found that the degree of earthquake damage to the tunnels is related to the route location, depth, structural type, and whether the tunnels pass through fault fracture zones and adverse geological sections. In addition, the damage to the portal section is more serious than that to the tunnel body. The investigation on the earthquake damage to mountain tunnels in the "5.12" Wenchuan earthquake found that the factors influencing the degree of tunnel damage mainly include whether the tunnel passes through fault fracture zone, depth, in-situ stress, surrounding rock conditions, and structural type.

For seismic strength and stability checking of tunnel portal wall, portal retaining wall and embankment retaining wall, see Chapter 13 herein.

9.4 Lining seismic measures

9.4.1 Foreign scholars investigated more than 100 cases of earthquake damage to tunnels and underground works, and found that the depth had a great influence on the degree of earthquake damage to tunnels and underground structures: of 49 cases involving slight earthquake damage, 29% had a depth of less than 50m; of 23 cases involving moderate earthquake damage, 39% had a depth of less than 50m; of 22 cases involving serious earthquake damage, 45% had a depth of less than 50m. The investigation on the damage of the "5.12" Wenchuan earthquake to tunnels found that tunnels in hard homogeneous rock mass sustained moderate to slight earthquake damage when the depth was more than 50m and suffered almost no earthquake damage or slight earthquake damage when the depth was more than 100m.

In addition, from the shaking table model test and analysis of a large number of numerical calculation results, it is known that within a depth range of between 50m and 100m, both dynamic response indicators of tunnel structure, i. e. peak ground acceleration and peak displacement, decrease rapidly as depth increases; once the depth exceeds 100 m ~ 200 m, the change is not obvious.

Based on comprehensive analysis, the fortification length of the shallow sections at tunnel portals is determined according to the depth of the tunnel and shall be classified by the length of the lining structure at depths less than 50m.

9.4.4 The minimum distance between the tunnel and an active fault is determined mainly based on the rupture width data from faults during seismic activities at home and abroad. When it is difficult to meet the minimum distance requirement in the tunnel route selection process, it shall be determined through seismic safety evaluation.

9.4.6 A large amount of earthquake damage data show that tunnel lining damage is often caused by the relative deformation of ground. A certain distance is reserved between the tunnel structure gauge and the intrados. This serves two purposes: First, it may prevent the lining from encroaching upon the gauge due to local deformation. Second, some space is reserved for the structural reinforcement of the tunnel after an earthquake to facilitate repair.

9.4.8 Due to significant difference in dynamic characteristics between tunnel structures and bridge structures, when it is impossible to avoid connecting tunnel and bridge because of constraints imposed by terrain or geological conditions, appropriate measures shall be taken to reduce structural damage caused by interaction between the two types of structures during the earthquake.

9.4.12 High modulus of elasticity materials such as stone pitching or plain concrete shall be used for backfilling the side wall back of the cut-and-cover tunnel, to effectively inhibit the "whipping effect" of the tunnel and reduce the displacement and deformation. The provision of seismic joints at the boundary between cut-and-cover tunnel and under-cut tunnel is based on the structural and damping needs.

9.4.13 Earthquake damage shows that the cut-and-cover tunnel is prone to damage by falling rocks from the high side and front slope during an earthquake. Therefore, it is necessary to provide a buffer structure layer for shock resistance and absorption on the roof of the tunnel to reduce the impact of falling rocks on the cut-and-cover tunnel structure during the earthquake. Meanwhile, an extended cut-and-cover tunnel may effectively reduce the threat of falling rocks to passing traffic and improve traffic safety.

9.5 Seismic design of special tunnel structures

9.5.2 1 The integrity of shed-tunnel structures is very important to earthquake resistance. Therefore, integral structure is preferred. In recent years, prefabricated assembly technology has been increasingly used. However, the joint of members is the weakest part of the structure and tends to lead to beam falling under the effect of earthquake force during an earthquake. In order to improve the seismic capacity, it is required to adopt seismic measures such as adding earthquake-resistant steel bars, displacement limiting devices, earthquake-resistant plates or blocking structures.

6 Cantilever shed tunnels have poor seismic performance and are prone to earthquake damage when the earthquake intensity is high. Once damaged, their emergency repair is very difficult, hence this requirement is made.

7 This paragraph provides the technical requirements for emergency repair phase of seismic fortification and gives the proposed engineering measures following the general principle of "rapid traffic maintenance and combination of permanent structures with temporary structures."

8 The provisions of this paragraph are mainly based on the consideration that the shed tunnel structure is more like a building structure.

9.5.3 Half tunnel structures are usually integral structures, and their loading conditions are similar to those of cut-and-cover tunnel and shed tunnel structures. The calculation may be performed according to the similar requirements for cut-and-cover tunnel and shed tunnel structures in these Specifications.

10 Shield Tunnel

10.1 General requirements

10.1.2 The shield tunnel is a tunnel structure formed by connecting prefabricated segments through transverse and longitudinal connecting bolts. It is safe, quick to construct and environmentally friendly. Such tunnel type has been widely used in underground lines of urban rail transit in China and underwater tunnel projects such as Wuhan Yangtze River Tunnel, Nanjing Yangtze River Tunnel, Qianjiang Tunnel, and Shiziyang Tunnel on Guangzhou-Shenzhen-Hong Kong Passenger Line, including many high seismic intensity areas. Under strong earthquake effects, shield tunnels may experience segment cracking, concrete spalling, dislocation, leakage, bolt breakage, joint plate damage and other seismic damages, as observed in Chengdu subway shield tunnel due to the Wenchuan earthquake in 2008 and Kobe subway shield tunnel due to the earthquake in southern Hyogo of Japan in 1995. Therefore, it is stipulated in this article that for large-diameter shield tunnels in high seismic intensity areas, aseismic calculations should be taken using various methods, and if necessary, verified by shaking table tests and other means.

10.1.3 In addition to the effect of seismic waves, the deterioration of site conditions caused by earthquakes, such as topsoil dislocation and cracking, differential settlement of subsoil and ground liquefaction, are also important reasons for the earthquake damage to shield tunnels. Therefore, the shield tunnel should be located in dense, homogeneous and stable ground, avoiding adverse geological zones. If this is not possible, reliable treatment measures shall be taken.

10.1.4 The apparent specific gravity of shield tunnels (the average value including cavity) is smaller than that of surrounding ground. The tunnel undertakes less inertia under earthquake effect, and experience faster vibration attenuation because it is constrained by surrounding soil. Shield tunnel lining is formed by prefabricated segments with bolts connection. There are a number of circumferential and longitudinal joints in the tunnel. The joint stiffness is usually less than the segment stiffness. The deformation capacity of a shield tunnel under earthquake effect is strong. The tunnel structure accommodates the seismic deformation of ground. The additional stress and deformation generated in segments are mainly caused by the relative displacement of ground. The response displacement method is exactly a seismic calculation method proposed according to the above-mentioned vibration characteristics of shield tunnels. For shield tunnels, the response displacement method is recommended for seismic calculation.

10.1.5 Shield tunnel lining is formed by bolting prefabricated segments. There are a number of circumferential and longitudinal joints in the tunnel. In addition to segments, the stress and deformation at the joints also play a controlling role in the safety and normal service of the structure. The interfaces between the tunnel and the cross passage, the junctions between the tunnel and the shield working shaft or ventilation shaft, and other parts with major changes in structural types and significant spatial effects, are prone to stress concentration and excessive deformation. Seismic check of shield tunnels shall include segments, joints, structural connections or intersections.

If the ground surrounding the tunnel s liquefied, the uplift of the tunnel, the dynamic earth pressure and dynamic water pressure, the drainage and settlement of the ground after the earthquake, and the lateral flow of the ground may lead to the instability of the tunnel structure. Relevant ground stability checking shall be carried out according to the actual ground conditions, such as liquefaction evaluation, and the overall stability of the structure shall be checked based on the changes in the ground after liquefaction.

10.2 Seismic response calculation

10.2.1 In principle, the time history analysis method is applicable to the seismic calculation of tunnels in all situations. When the structural type of the tunnel is continuous and regular in the longitudinal direction with the cross-sectional configuration unchanged, and the surrounding ground is uniformly distributed along the longitudinal direction of the tunnel, the seismic calculation in the cross-section can be treated as a plane strain problem. There are a large number of joints between rings in shield tunnels. They have strong deformation capacity and experience large deformation between rings under earthquake, which may lead to earthquake damages such as tunnel water leakage, dislocation between rings, and bolt yielding. Therefore, longitudinal seismic calculation is required for shield tunnels. For tunnel sections in complex topographic and geological conditions and with major changes in structural types and significant spatial effect, e. g. the interface between shield tunnel and cross passage, and the connection between tunnel and shield working shaft or ventilation shaft, 3D spatial model should be used for seismic calculation. In order to prevent the reflection of seismic waves at the model boundary, the boundary conditions that can reduce the reflection of seismic waves, such as viscous or viscoelastic artificial boundary, are preferred for the model boundary. Generally, the acceleration time history of ground motion is chosen as the input ground motion in the time history analysis method.

10.2.2 1 A shield tunnel consists of segments and joints. In order to reasonably reflect the characteristics of the shield tunnel, the influence of joints needs to be represented in the structural model. The transverse equivalent-stiffness beam model and the beam-spring model in the cross-sectional direction of the shield tunnel are shown in Fig. 10-1.

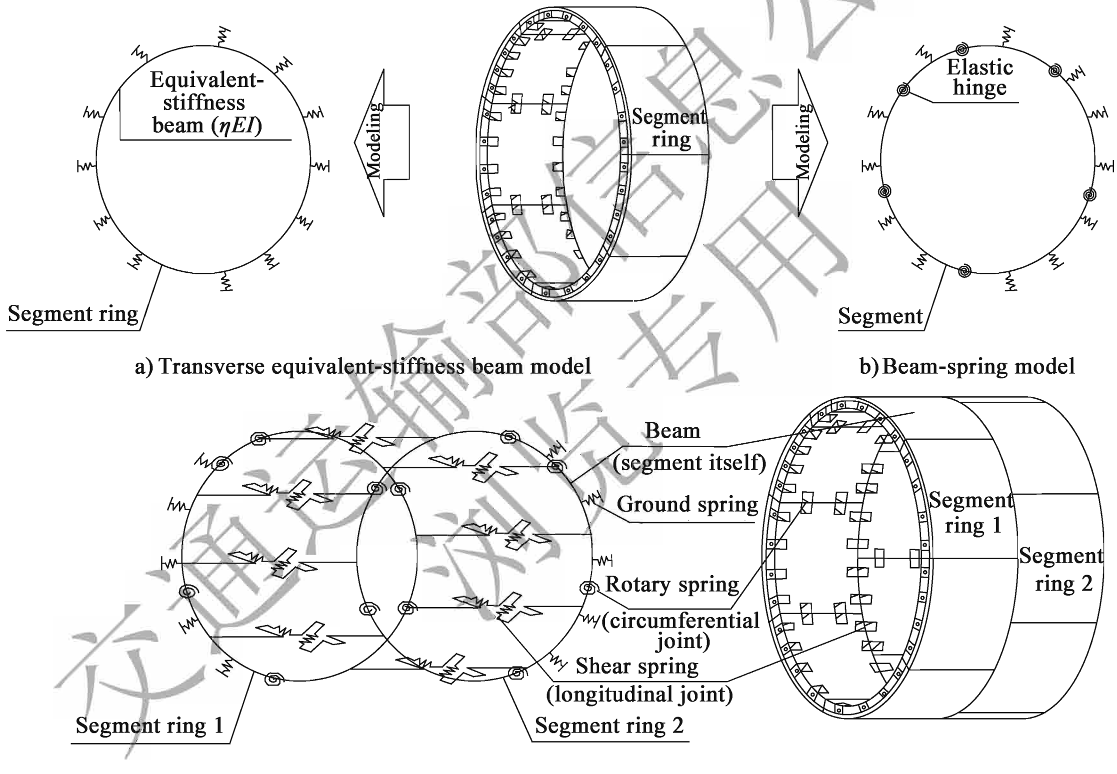


Fig. 10-1 Schematic diagram of equivalent-stiffness beam model and the beam-spring model in the transverse direction of shield tunnel

2 The equivalent-stiffness beam model and beam-spring model in longitudinal direction of shield tunnel are shown in Fig. 10-2.

3 The secondary lining of a shield tunnel is concrete placed on the interior of segment rings to perform design functions when it is difficult to achieve the intended purpose of the tunnel by segment rings alone. Shaking table test proves that the application of secondary lining improves the rigidity of shield tunnels and reduces the seismic induced axial force and strain in the segment ring

and joint. This suggests that the secondary lining can reinforce the structure prior to failure.

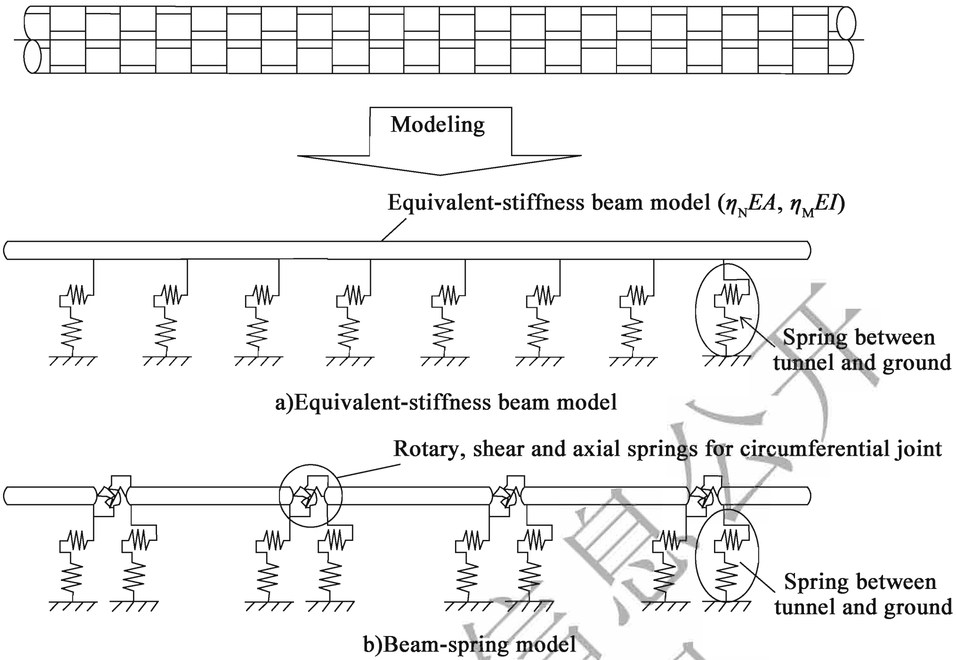


Fig. 10-2 Schematic diagram of equivalent-stiffness beam model and the beam-spring model in the longitudinal direction of shield tunnel

The secondary lining is divided into non-structural secondary lining and structural secondary lining. The purpose of the non-structural secondary lining is to reinforce segment ring, prevent corrosion and reduce vibration, improve lining appearance and correct tunnel route. The purpose of the structural secondary lining is to form structural members of the tunnel together with segment rings. For the non-structural secondary lining, special seismic calculation is not required in seismic design because it is not considered as a structural member. For the structural secondary lining, the International Tunnelling Association (ITA) Guidelines for the Design of Shield Tunnel Lining and China's Code for Design of Metro (GB 50157—2013), divide it into superposed structure (with interlayer shear strength and able to transmit axial force and shear force) and composite structure (with on interlayer shear strength and only able to transmit axial force) according to the smoothness of the interlayer bonding surface of the double-layer lining. When performing seismic calculation of the structural secondary lining, the lining is modeled using beam elements. The interaction effect between secondary lining and segment ring in composite structure and superposed structure is modeled using spring elements. The composite structure and superposed structure are simulated respectively by setting different interlayer spring stiffness. The modeling of segment rings is the same as that for single-layer lining. The double-layer lining model (segment ring modeled using spring elements as an example) is shown in Fig. 10-3.

Elastic hinge 4 In order to meet the normal traffic requirements for highway shield tunnels, travelled way slab, flue sheet, maintenance channel and other structures shall be constructed inside the segmental lining. The working state and mechanical properties of the internal structures under earthquake effect will directly affect the capacity of the shield tunnel to maintain normal or emergency passage after the earthquake, so the internal structure shall also be considered in the seismic calculation. Similar to the secondary lining of shield tunnels, the stiffness of connections between internal structure and segmental lining is also varied. For example, the connection between the travelled way slab, maintenance channel, and segments requires high stiffness and may be rigid. Flue sheets and the like have low stiffness and weak connection with segments, and such connection may be elastic or hinged.

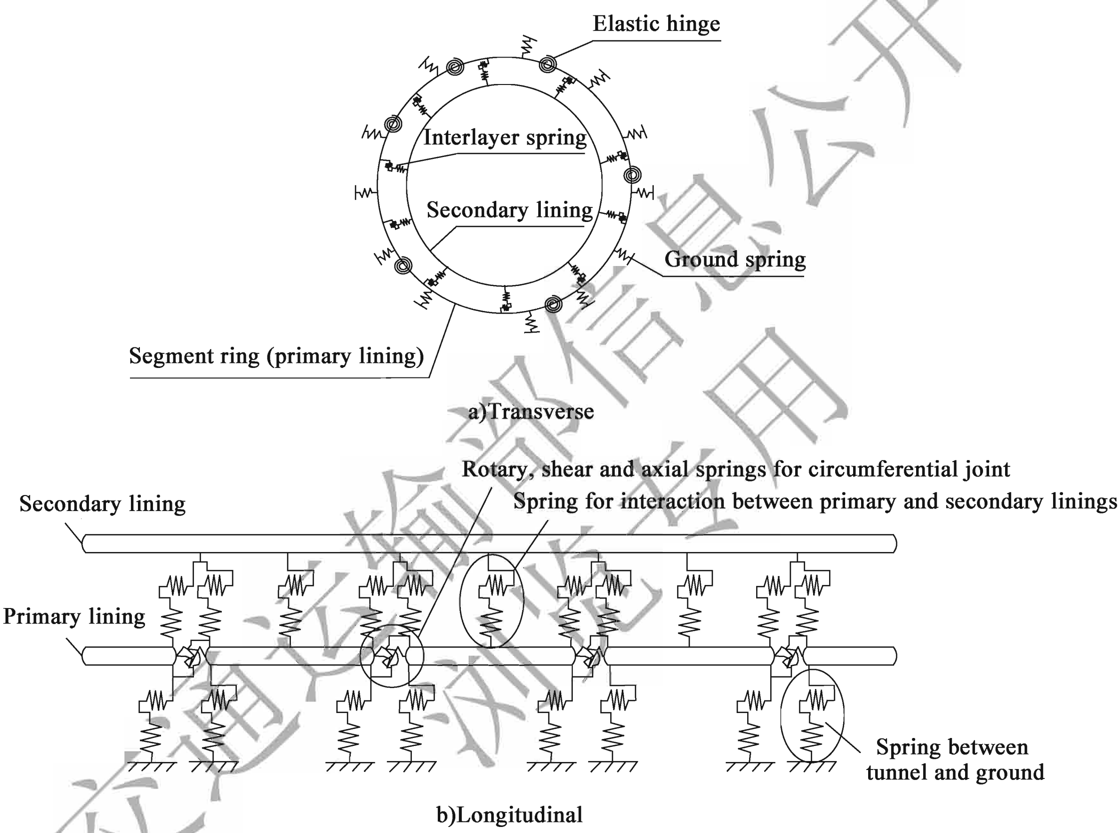


Fig. 10-3 Schematic diagram of double-layer lining model

10.3 Seismic check

10.3.3 The interfaces between shield tunnel and cross passage, working shaft or ventilation shaft, due to complex configuration and significant spatial effect, are susceptible to stress concentration and excessive deformation and tend to experience earthquake damages such as shearing of connecting bolts, water leakage at joints, dislocation of segments and concrete stripping. During the 1985 earthquake in Michoacan, Mexico, circumferential joints at an interface between a sewer shield tunnel in Mexico City and its working shaft sustained damages at 5 locations within a range of 2 ~ 3 rings. In the 1995 earthquake in southern Hyogo, Japan, an interface between a shield tunnel and the shaft suffered earthquake damages such as concrete falling, segment dislocation, and serious water leakage. Therefore, seismic check is required for these structural connections. In seismic check, their deformation capacity, bearing capacity and waterproofing capacity under design earthquake effects shall be fully considered. There is limited research data on the segment ring deformation rate and joint opening amount in shield tunnels under earthquake effect. According to the available research results from Japan, it is generally believed that when the inclination angle (transverse deformation difference corresponding to the top and bottom/tunnel diameter) in the shield tunnel cross section under L2 earthquake effect is 1/150-1/200, the structural material may be considered to be within the elastic limit and essentially safe. If the opening between rings is within 2-3mm, the structural material may be considered to be within the elastic limit and essentially safe. It should be noted that the existing research results are based on a given earthquake effect (L1 or L2 in Japan) and shield tunnels of ordinary size. With an increase in the cross-section size of shield tunnels and the change in joint structure, the indicator for deformation checking is not a fixed value and needs adjustment on a project-specific basis for seismic design.

10.4 Seismic measures

10.4.1 Proper seismic measures are more economical and reasonable than simply relying on raising the seismic fortification standard to increase seismic capacity. Due to the constraints of surrounding media, the seismic response characteristics of tunnels, especially shield tunnels are different from those of aboveground structures. The seismic response of shield tunnels mainly depends on the relative displacement of ground. There are mainly two methods to control the relative displacement of ground: one is to take necessary structural measures to make the tunnels vibrate easily with the vibration of the ground and improve the seismic performance of the tunnels themselves; the other is to reduce the seismic energy transmitted from the ground to the tunnel structure by engineering means, such as bypassing unfavorable geological zones, improving soil mass, and placing a seismic isolation layer between the shield tunnel and the ground.

10.4.3 According to the experience from engineering practice, the deformation of shield tunnels mostly occurs at the tunnel joints. The use of flexible segments or steel pipe sheet ring at special locations, e.g. the interfaces between the tunnel and the cross passage or shaft or at the abrupt changes in topographic and geological conditions enable the tunnel to accommodate the displacement of surrounding ground, and reduce the structural internal force caused by earthquake.

However, flexible segments or steel pipe sheet ring are complicated in structure and expensive, and therefore are generally only used when the seismic fortification requirements are high.

Fig. 10-4 is a schematic diagram of straight bolts and bent bolts. Obviously, straight bolts are easier to accommodate seismic deformation, and cause less damage to the segment structure when they deform, so straight bolts should be used for segment connection from the perspective of earthquake resistance.

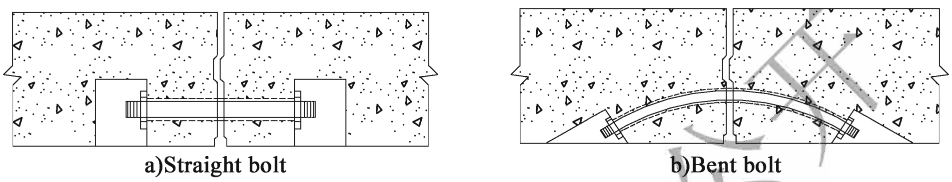


Fig. 10-4 Schematic diagram of connections using different bolts

Using elastic waterstops film with strong resilience (Fig. 10-5) at the joint of a shield tunnel, with appropriately increasing the thickness of the film and applying pre-stress for fastening, can effectively prevent leakage during an earthquake, and thus ensure the normal operation of the tunnel. In addition, installation of rubber waterstops along the joint between shield tunnel and shaft can prevent water leakage due to excessive deformation.

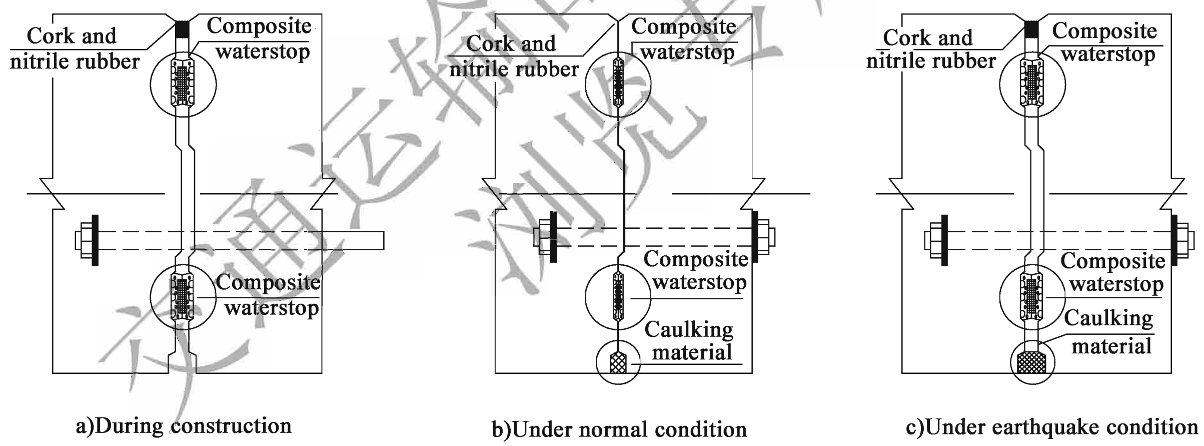


Fig. 10-5 Schematic diagram of elastic waterstop film

Using replaceable water-swellable rubber seal ring (Fig. 10-6) as bolt hole seal washer can not only stop water, but also reduce structural stress concentration caused by earthquake.

10.4.4 Installation of an isolation device between the periphery of the lining and the surrounding rock turns the original lining-rock system into a lining-isolation-rock system. Its purpose is to separate the lining from the surrounding rock medium by the isolation layer and to reduce or change the intensity and mode of earthquake effect on the structure and thus reduce structural vibration. The isolation layer is required not only to absorb the cyclic dynamic strain or relative dynamic displacement between the lining and the ground, but also to have sufficient elasticity to ensure that after undergoing plastic deformation during an earthquake, it can play a role again in the next one. Pressure-injected isolation layer is a newly developed isolation material, including asphalt, urethane, rubber, and silicone series. These materials are usually in liquid form and are injected into the gap between surrounding rock and the lining together with hardening additive. Once hardened, they form an isolation layer. This type of isolation material is highly resistant to shear deformation, durable, workable, and tends not to produce harmful substances. For shield tunnels, silica gel-based seismic isolation materials have been applied during wall back grouting in Japan.

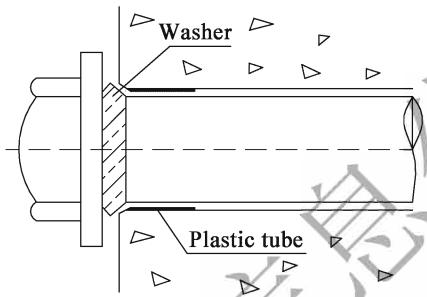


Fig. 10-6 Schematic diagram of bolt hole seal washer

10.4.5 A number of joints present in shield tunnels can accommodate the seismic displacement of ground surrounding the tunnel by their own deformation under the effect of earthquake, to reduce the internal force in segments. Meanwhile, it should be noted that if the structural stiffness of a shield tunnel is too small, excessive deformation of the structure will lead to its failure or affect its normal use. Earthquake damage investigation and shaking table tests show that the provision of secondary lining in the shield tunnel can strengthen the structure and reduce earthquake damage under certain conditions. Therefore, secondary lining may be installed at the local location or along the full length of the tunnel according to the conditions to increase the structural stiffness and integrity for earthquake resistance.

Liquefaction of the ground surrounding a shield tunnel may lead to instability of the tunnel structure. The tunnel route shall be selected to avoid crossing the ground with potential liquefaction. If this is not possible, appropriate measures shall be adopted to improve the ground around the tunnel or strengthen the tunnel structure to ensure the stability and safety of the tunnel. Special research shall be carried out if necessary.

11 Immersed Tunnels

11.1 General requirements

11.1.1 Seismic design of immersed tunnels usually involves three steps. First, investigate and collect data on historical earthquakes within a radius of at least 200km in the proposed project area, especially collection and analysis of the historical records of recent major and strong earthquakes. Use the fixed value method, probabilistic method or combination method for seismic safety evaluation, and establish the fortification criterion and ground motion parameters (including peak ground acceleration and velocity, displacement amount, response spectrum, and vibration duration curve). Second, evaluate the potential for ground failure or deformation in the project area during earthquake, such as earthquake-induced subsidence, liquefaction, fault dislocation, instability of underwater foundation trench slope, longitudinal extension and compression and longitudinal bending and transverse deflection of ground, and propose the foundation treatment measures to be taken and material requirements. Third, evaluate the seismic response of the tunnel structure (free field, considering the soil-structure interaction effect), and perform structural calculation and design such that the tunnel structure always has sufficient ability against floatation, and the joints always remain watertight. It should be noted that the seismic performance and stability calculation of immersed tunnel joints is an important step in the design of immersed tunnels. Various possibilities often need to be considered in seismic calculation, and satisfactory results may be obtained only through repeated iterations.

11.1.2 When it is impossible to completely avoid seismic-induced settlement or liquefaction of soft soil, reliable and effective engineering measures shall be incorporated in design based on the seismic calculation. In addition, verify the immersed tube section always has an adequate anti-floating safety factor, and the joint always remains in a watertight state.

11.1.3 According to the provisions of Section 3.1 herein, the seismic design of immersed tunnels shall be performed per Class A. Reinforced concrete structure or steel shell concrete structure may be used for elements. Integral or segmental elements may be used for longitudinal system. A two-level (E1 and E2) fortification criterion is adopted for seismic design of most immersed tunnel projects. Some immersed tunnels in foreign countries have been in operation for over 100 years and have withstood the test of many earthquakes. It is unnecessary and uneconomical to take excessive seismic measures. Based on the domestic and abroad engineering experience, seismic calculation under E1 fortification level may be performed using the linear elastic model; the immersed tube structure, the shear key, the shock-absorbing member, etc. are required to be in elastic state and maintain normal functionality during earthquake. E2 fortification level allows the element structure or joint to be in an elastic-plastic transition zone, and the use of non-linear elastic-plastic model for seismic calculation. It requires that the element structure and the shear key remain within the yield strength, and the shock-absorbing members and the like may be slightly damaged and remain functional without maintenance or with simple reinforcement.

11.1.4 According to the characteristics of immersed tunnel projects, the seismic performance of ventilator and other equipment shall be checked in coordination with the design of civil structure.

11.2 Seismic response calculation

11.2.1 Generally, an appropriate seismic response calculation model may be selected according to the specific engineering characteristics, progress stage, technical requirements, applicability, etc. As the project progresses, the seismic response calculation model is required to be more precise and more sophisticated verification methods and means are required.

If the immersed tube structure is regular in the longitudinal direction, the cross-sectional configuration is consistent, and the surrounding soil layers are distributed uniformly along the longitudinal direction, a two-dimensional model is preferred to calculate the transverse seismic response as a plane strain problem. In the case of major changes in the immersed structure form, unevenly distributed soil, sharp changes in topography, geology and strata, and structural intersection or overlaps, 3D modeling may be employed to solve it as a spatial problem. Whether the local 3D model of the connection element at the immersion joint is properly selected or not has a great influence on the calculation results.

11.2.2 Under normal circumstances, appropriate calculation methods are selected according to the site conditions and structural conditions of immersed tunnels and the technical requirements of different structures. The static method, response displacement method and dynamic time history analysis method are most commonly used for domestic and abroad calculation, combining global and local models and integrating 2D and 3D structural models. Major projects employ various calculation methods for cross check. No matter which calculation method is adopted, it is very important to select appropriate connection elements and spring coefficient to simulate the connection characteristics between each of tunnel joints, structures and geotechnical media respectively.

Seismic calculation for the structural safety of immersed tunnels and the stability of surrounding ground usually includes the calculation of stress, deformation, skid resistance and stability against buoyancy of the element structure as well as calculation of liquefaction resistance and dynamic settlement of subsoil surrounding the element structure. In general, the response displacement method or generalized response displacement method is selected depending on availability of design ground motion time history curve of the proposed engineering site and the required calculation accuracy. See Appendix B herein for the detailed calculation formulae. In the transverse response displacement method, the displacement of the free-field ground in the cross-sectional direction of the tunnel and the peripheral shear force are applied to the end of the beam and spring models simulating the immersed tube structure to calculate the transverse internal force in the structure during the earthquake. In the longitudinal response displacement method, the immersed tube structure is simplified along the longitudinal direction into beams and springs with certain stiffness to simulate longitudinal tension and compression and longitudinal bending; the displacement of the free-field ground in the longitudinal direction of the tunnel is applied to the end of the ground spring to calculate the longitudinal internal force in the structure during earthquake.

11.2.3 Beam-particle-spring model and immersed tube-soil finite element model are commonly used for overall seismic analysis of immersed tunnels. An efficient, timesaving, simple and feasible method is used to directly calculate the deformation of soil in free field by finite element method, and then apply the results to the immersed tube structure for pseudo-static seismic response calculation. In order to account for the interaction effect between the immersed tube structure and the surrounding soil, it is very effective to build a 3D finite element model of the whole tunnel for seismic response calculation.

11.2.4 Earthquake effect on immersed tube structures generally includes horizontal earthquake inertia caused by self-weight of the structure, horizontal earthquake inertia caused by backfill overburden, and lateral pressure increment caused by earthquake, as shown in Fig. 11-1.

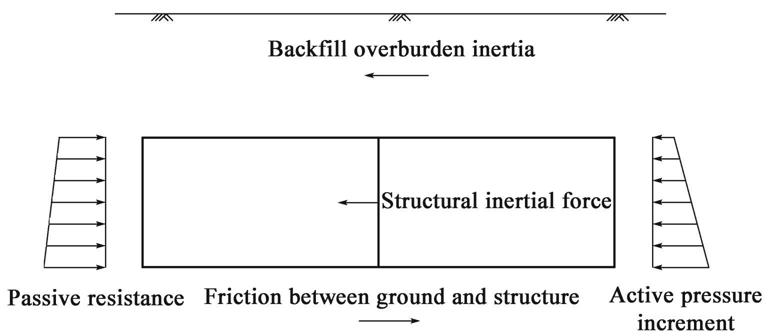


Fig. 11-1 Static method calculation model

11.3 Seismic check

11.3.1 In the design of immersed tunnels, element length, joint location and configuration and the location of the ventilation shaft are first determined through comprehensive comparison and selection, and then verified by longitudinal seismic check. According to engineering experience, the immersed tunnel shall be checked for joint opening and stress and bearing capacity under earthquake conditions, using the combination of short-term and long-term effects at normal service limit state (E1 earthquake effect) and accidental combination at bearing capacity limit state (E2 earthquake effect). When the seismic wave transmitted from the seismic source deep in the ground enters the immersed tunnel at a certain oblique angle, it will lead to instantaneous high-frequency cyclic tension and compression deformation in the longitudinal direction of the immersed tunnel while generating deflection in the cross-section of the immersed tube. If the obtained maximum strain is within the elastic range, no special joint is required. Otherwise, special seismic joints between elements as well as between elements and ventilation stack or buried section shall be considered. If necessary, highly adaptable large deformation joints shall be developed and used to ensure that waterstops are always in reliable watertight state.

It should be noted that if the period of synthetic seismic wave is close to the natural vibration period of the immersed structure system, resonance effect tends to occur, which will lead to distortion of calculated displacement and axial force. In fact, because seismic waves are random, the resonance effect is unlikely to happen, and generally not considered.

The traveling wave effect of seismic wave propagating in the soil layer at the site of immersed tunnels is very complex, and causes vibrations at each point in the longitudinal direction of the tunnel that vary in phase and amplitude, so the seismic response of immersed tunnels can be more truthfully reflected by employing multi-point nonuniform seismic excitation than uniform seismic excitation. When conditions permit, shaking table tests may be carried out for verification. Before performing shaking table tests, 3D models incorporating subsoil, immersed tunnels and flexible joints are usually developed using solid elements for refined finite element analysis, and for dynamic response analysis by inputting seismic waves, thus obtaining deformation, stress and strain in the ground and structures at every moment. On one hand, this provides guidance on the planning of shaking table test schemes, and on the other hand, serves the purpose of mutual verification. For example, a complete 3D model including immersed tunnel was developed for Shanghai Outer Ring Tunnel to perform 3D non-linear numerical analysis of seismic response. This model truly reflected the seismic response of the structure. Moreover, the determination and selection of model boundary conditions, horizontal, longitudinal and vertical spring dynamic stiffness, input position and angle of ground motion, and damping coefficient of ground around the tunnel have a great influence on theseismic check results of immersed tunnels, and often require much calculation experience from designers. 3D shaking table physical model test focuses on whether the boundary constraint conditions are appropriate and may employ sponge boundary.

If the tunnel project requires accurate determination of the soil damping coefficient and dynamic stiffness of the joint structure, the best way to obtain them is through special tests. The dynamic stiffness of the ground spring includes stiffness in three directions: horizontal, vertical and longitudinal. The dynamic stiffness of the spring in the horizontal direction is related to the stratum distribution and backfill conditions, and only consider the compression characteristics. The dynamic stiffness of the spring in the vertical direction is related to stratum distribution and foundation type, and can be obtained by multiplying vertical static stiffness by a coefficient. The dynamic stiffness of the spring in the longitudinal direction is determined based on the vertical load, and friction characteristics between the structure and backfill or foundation bed.

The seismic influence on immersed tunnels is restricted by the seismic peak ground acceleration, ground properties and immersed structure characteristics. Results show that the angle of incidence of seismic wave has a significant influence on the seismic response of immersed tunnels. For example, if the angle of incidence is 0° (along the axial direction of tunnel), the axial relative displacement and axial force of the flexible joint are the largest. When the angle of incidence is 90° (perpendicular to the tunnel axis), the bending moment, transverse shear displacement, relative angular displacement and transverse shear force of the flexible joint are the largest.

11.3.5 The watertight safety factor of waterstop is the ratio of the maximum water head pressure that the waterstop can withstand to the actual water head pressure of the project. It is related to the composition and strength of material of the waterstop, and specified in these Specifications by reference to engineering examples. See Table 11.1.3 and Table 11.3.6 herein for the requirements to element structure and joint performance under E2.

11.3.6 In seismic design of immersed tunnels, their seismic performance is enhanced by increasing the structural strength of elements, while avoiding joint failure or leakage during earthquake by increasing their capacity to absorb deformation. To this end, the design of immersed tunnels shall avoid the structure with complex shape and abrupt changes in stiffness.

In the seismic calculation of immersed tunnels, the natural vibration of the element structure is generally not considered, but the influence of its stiffness shall be considered. Under earthquake conditions, the maximum internal force in the immersed tube structure shall not exceed its own permissible stress. As a weak link in seismic resistance of immersed tunnels, the performance of the joint changes with time and stress conditions. The joint must be able to withstand the tensile and shear deformations caused by static force, dynamic water pressure, temperature, concrete shrinkage and creep, seismic force and so on while always maintaining water tightness. Longitudinal and transverse vibrations may cause immersion joint opening/compression and transverse dislocation and torsion, i. e. relative axial displacement and horizontal or vertical dislocation and torsion. Therefore, regardless of joint stiffness, the joint shall be able to withstand axial internal force (including axial pressure, shear force and bending moment) corresponding to joint stiffness while being able to absorb displacement not less than permissible value, in order to maintain a watertight joint structure during earthquake.

The joint opening is related to the permissible settlement of the base at the joint, the friction force around the tube, the length of the element, and the height of the element at the joint. Generally, the joint opening under earthquake conditions is obtained through longitudinal calculation. It should be noted that the seismic calculation of immersed tunnels shall consider the heating condition in summer, when the expansion of immersion joints and elements increases GINA and joint stiffness, resulting in the closure of most joints, while the cooling in winter does the opposite. Research and investigation found that it is very difficult to accurately give the joint displacement control criteria suitable for different immersed tunnel projects. Based on the engineering examples and calculation results of Guangzhou Pearl River Tunnel and Shanghai Outer Ring Tunnel, the control values for joint displacement calculation under general conditions are presented, but excluding joints at the junction between immersed tube section and buried section, and the interface between soft and hard strata, or areas where the seismic peak ground acceleration is 0. 15g or more. Special joint structures capable of withstanding displacement of at least 5-10cm or more are selected for local sections. Different simplified methods, input seismic waves, calculation models and joint models may differ or even vary widely. For example, when the axial displacement is similar, the axial force, transverse internal force and displacement in the element may differ by several times. For example, an increase in the thickness of back-silting at the top of the tube will increase the internal force in the structure by more than 20%. In order to limit the displacement of joints of immersed tunnels in highly seismicity areas, especially strong joint structural measures are usually required.

The smaller the permissible value of longitudinal or transverse differential settlement of immersed tunnels is, the better it is for the structure and joint performance. If the structure size is basically determined, this permissible value needs to be determined based on comprehensive consideration of the stiffness of the ground and foundation at the bottom of the tube, the weight of backfill (including back-silts) and protective layer on the top of immersed tube section, and the safety reserve of shear key. In any case, the vertical shear of immersion joint with prestressed anchor cable in the longitudinal direction is much smaller than that without such prestressed anchor cable. This is closely related to the value of applied prestress. If this value is large enough, the joint stiffness is considerably higher, and the vertical shear and horizontal displacement are approximately 0 mm.

11.4 Seismic measures

11.4.1 With reference to similar projects abroad, measures to properly strengthen the structural configuration and ensure water tightness with waterstops are usually taken for the immersed tunnel when the basic seismic intensity is VI. For safety factors of reinforced concrete structure, steel structure and rubber waterstops, please refer to the current codes or specification.

11.4.2 The immersed tunnel is generally buried at a shallow depth. Its horizontal and vertical vibrations caused by ground motion tend to result in longitudinal extrusion or expansion and horizontal dislocation of the joint shear key. Therefore, both the element structure and waterstops require sufficient safety reserve. When the integral reinforced concrete element structure is adopted, steel plate encasement or waterproof membrane shall be used to improve the waterproofing performance.

The immersion joint usually consists of horizontal and vertical shear keys, the first-pass waterstops (such as GINA) and the second-pass waterstop (such as OMEGA). Under the earthquake effect, the maximum internal force and deformation response of the joint cannot exceed the permissible range of the joint structure. In order to reduce the relative displacement of joints under earthquake conditions, it is necessary to fully leverage and increase the design stiffness of joint structures. In general, the joint structure can withstand a relative displacement of at least 10mm, and usually a bearing or a compressible cushion may be arranged on the load-bearing contact surface.

When the joint opening or torsion angle obtained from seismic calculation of immersed tunnels exceeds the permissible deformation capacity of the waterstops, in addition to reasonably designing the shear key device, it is also important to provide waterstops capable of withstanding the design water pressure and deformation and a limiting device capable of withstanding the design opening displacement based on results from the longitudinal seismic calculation of immersed tunnels. When the toughness of the element material is not sufficient to absorb the shear deformation of the cross section imposed by the ground, appropriate seismic structural measures shall also be taken in the design to allow the plastic deformation of individual portions of the cross section, andseismic check of the element structure shall be carried out using plastic deformation method. Generally, longitudinal limiting devices such as steel cables, tie rods or unbonded prestressed tendons, Ω-shaped or double corrugated steel plates may be provided, as shown in Figs. 11-2 to 11-4.

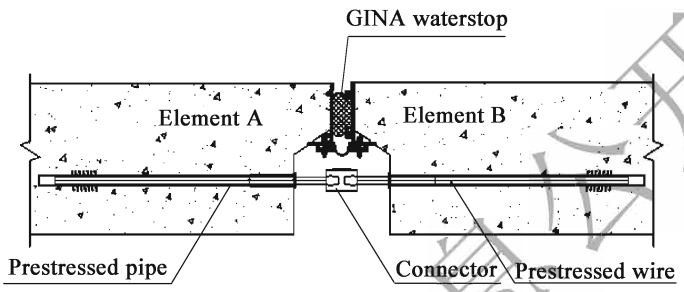


Fig. 11-2 Diagram of limit cable and embedded parts at immersion joint

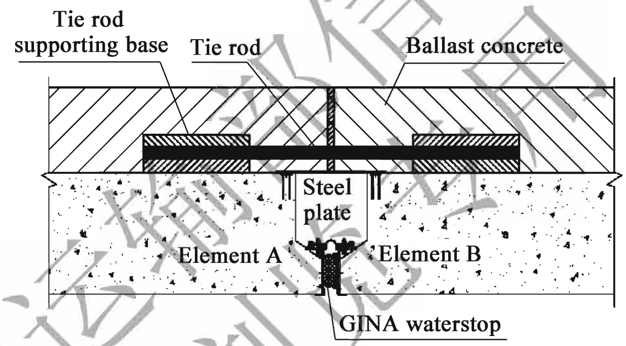


Fig. 11-3 Limit tie bar device in the ballast layer

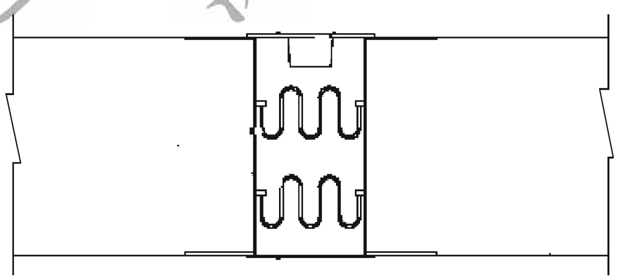


Fig. 11-4 Schematic diagram of double corrugated steel plates capable of withstanding large deformation

When sand is used as filling material in the foundation bed for immersed tunnels, sand samples shall first be tested for physical properties and strength against liquefaction. If the shear strength against liquefaction is low, a certain proportion of cement clinker may be added to ensure sufficient chemical bonding force between sand particles to prevent liquefaction of sand bed. In the design of pile foundation, the range of earthquake-induced subsidence or liquefaction of surface soil shall be analyzed. Negative frictional resistance shall be considered in the calculation of bearing capacity. The length of the bottom end of pile foundation extending into the stable soil layer below liquefaction depth shall be determined through calculation. This length shall not be less than 0.5m for gravelly soil, gravel, coarse and medium sand, hard clayed material and dense silt, and should not be less than 1.5m for other soils other than rock.

Silty sand and medium sand with uniform particles are prone to liquefaction when in a saturated state underwater. Undisturbed sand layer and low plasticity silt under the immersed tunnel shall be subjected to liquefaction analysis along the tunnel axis based on the test results. If the liquefaction analysis indicates a section does not meet the design requirements, then measures such as replacement and ground stabilization shall be taken to eliminate the liquefaction impact.

China's Pearl River Immersed Tunnel and Eastern Cross Harbour Tunnel in Hong Kong, which have been completed, are both located in the VII earthquake zone. Natural weathered bedrock was used as the foundation and sand injection method was used to construct the bed course. The ground for the Western Harbour Tunnel in Hong Kong was treated using replacement sand. The immersed tunnel in Busan, South Korea, which is also located in a VII earthquake zone, employed a treatment solution in which the original soft soil layer was improved using deep cement mixing piles and sand compaction piles before laying a gravel bed course.

In order to reduce the shear force and internal force in the joint under earthquake conditions, independent foundations should be adopted for structures in the buried section and at tunnel portals, and light materials should be used for backfilling the top of the tunnel section on land wherever possible.

11.4.3 More than 10 immersed tunnels built in China are mostly located in the VII earthquake zone, where the seismic peak ground acceleration of the engineering site does not exceed 1.5m/s^2. Due to the lack of engineering examples of immersed tunnels in VIII and above zones at this stage, specialized researches shall be conducted on a project-specific basis.

12 Cut-and-cover Tunnels

12.1 General requirements

12.1.1 The seismic response of cut-and-cover tunnel is greatly affected by the topographic and geological conditions at the construction site, and their deformation follows that of the surrounding ground. Therefore, these tunnels shall be built on a favorable foundation in dense, homogeneous and stable geological conditions, with the surface fluctuation as small as possible, and thus conducive to the structure stability when subjected to earthquake effect.

12.1.5 The backfill soil cover for cut-and-cover tunnels shall meet the physical and mechanical indices used in design calculation where possible. Subsoil with the same properties as surrounding ground shall be adopted as backfill after full and uniform compaction and consolidation. When backfilling is also required around the structure, the design materials shall be the same soil materials as the surrounding ground where possible, and their physical and mechanical indicators shall be maintained within the range given in the investigation report, so that their seismic internal force response conforms to the design calculation conditions, and provides sufficient resistance.

12.2 Seismic response calculation

12.2.1 Since a cut-and-cover tunnel is generally a long underground structure, the seismic calculation method based on the plane strain of the cross section is applicable to underground structures at a distance of more than 1.5 times the structural span from the tunnel portal or irregular cross section. The stress and deformation of structures at tunnel portals or irregular cross sections are relatively complicated, so it is more appropriate to use dynamic time history analysis method for seismic calculation with a spatial structural model.

When the cross-sectional shape of the tunnel changes abruptly or the tunnel forms a whole with adjacent buildings or structures, it is generally necessary to consider horizontal earthquake effects in both the transverse and longitudinal directions.

When retaining walls are used for foundation pit excavation during construction of cut-and-cover tunnels, the retaining walls, especially those overlapped with tunnel structures, will deform together with cut-and-cover tunnels under earthquake effect, so they may be considered as a whole with tunnel structures in seismic calculation.

12.2.2 Generally, the tunnel is longitudinally long, with basically the same transverse structural type and configuration. Therefore, in general, it can be treated as a plane strain problem for cross-sectional seismic design under horizontal ground motion effects. The tunnel structure is regarded as a frame structure on an elastic foundation, with the tunnel lining modeled by beam elements and the stratum-structure interaction modeled by springs. For tunnels with complex shapes and longitudinally crossing non-homogeneous ground, longitudinal and vertical earthquake effects also need to be considered due to the complex topographic and geological conditions.

12.3 Seismic check

12.3.2 According to Article 3.1.3 herein, when a highway tunnel is subjected to an earthquake equivalent to the seismic fortification intensity in the region, its main structure shall not sustain damage or remains operational without the need for repair. Therefore, the internal force analysis under E1 earthquake effect is the most basic requirement for checking the seismic response and cross-section bearing capacity of cut-and-cover tunnel structures. Corresponding to this, in view of the analysis method of the seismic response of the structure under the E1 earthquake, the seismic calculation is based on the theory of linear elasticity. Therefore, this section suggests that for checking the cross-section bearing capacity of cut-and-cover tunnel structures, it is assumed that structure and components are under elastic stress.

12.3.4 When the cut-and-cover tunnel is built in the ground with potential for liquefaction, attention shall be paid to check its stability against buoyancy, and taking measures to stabilize the ground when necessary to prevent liquefaction of the ground around the structure during earthquake. Since the dynamic characteristics of the ground will change after the stabilization measures are taken, this article requires that the liquefaction reduction coefficient be determined based on the ratio of the measured SPT blow count to the critical blow count, for calculation of the frictional resistance of the diaphragm wall and uplift pile.

12.3.5 When a good foundation pit enclosure such as diaphragm wall is adopted in the construction of cut-and-cover tunnels, the underlying foundation soil is surrounded by the enclosure, effectively inhibiting the shear deformation of the ground. The liquefiable soil layer contained therein is generally unlikely to liquefy during an earthquake, however, the influence of liquefaction of the surrounding soil layer needs to be considered in checking its strength and stability against buoyancy.

12.4 Seismic measures

12.4.1 Rectangular reinforced concrete structures are generally adopted for cut-and-cover tunnel structures, and their seismic structural measures may follow those used for similar aboveground structures. It should be noted that shear deformation of surrounding ground places higher requirements on the deformation capacity of cut-and-cover tunnels. In addition, since the excavated tunnels are located below the groundwater level in many cases, their weak areas in waterproofing need to be strengthened. The size of members of an underground reinforced concrete frame structure is often larger than that of a similar aboveground structure. However, because the requirements for frame structures with different functions vary, this article specifies only the minimum size of the members shall at least conform to the provisions for similar aboveground structure members, without giving specific sizes.

12.4.2 Paragraph 1 of this article provides measures to appropriately strengthen frame columns under the design concept of "strong column and weak beam". The rest of the provisions all specify stronger measures than the aboveground slab-column structure, with a view to balancing the need for safe load carrying and ease of construction. In order to speed up the construction progress and reduce the exposure time of foundation pit, beamless rib structures are often adopted for the base slab, roof slab and floor slab of underground structures, making the load-carrying system of the base slab, roof slab and floor slab no longer a slab-beam system, so they should be strengthened by providing hidden beams in the slab belt on top of the column.

In order to strengthen the integrity of the floor structure, Paragraph 3 specifies measures to strengthen the structural connection between the surrounding walls and the floor slab.

Under the horizontal earthquake effect, the openings in the side wall, roof slab and floor slab of underground structures will all affect the seismic bearing capacity of the structural system. Therefore, it is necessary to properly limit the opening area and to strengthen the members around the openings with necessary measures.

12.4.3 Compared with the aboveground structure, the underground reinforced concrete frame structure requires more steel bars. As the structural configuration of a cut-and-cover tunnel is generally regular, its reinforcement configuration requirements may follow those for the aboveground reinforced concrete frame structure. In the case of high requirements for seismic measures, the main bar shall be designed for overall length, and the standard for arrangement of transverse steel bars also need to be raised accordingly.

12.4.5 Technical measures such as grouting and soil replacement can effectively eliminate or reduce liquefaction hazards posed by liquefiable soil layers in the surrounding soil and ground.

Where no measures are taken to address the liquefiable soil layer, its potential for floatation shall be considered. See Section 12.3 herein and the relevant contents in the previous sections herein for checking methods and requirements. Floatation resistance measures shall be taken when necessary. When the ground contains a thin interlayer of liquefiable soil, it is better to strengthen the underground structure than to stabilize the ground.

If a diaphragm wall deeper than 20m is used as enclosure in the excavation of foundation pit, the soil mass in the pit will be encased by the diaphragm wall to form better site conditions, and liquefaction is generally unlikely happened under earthquakes. In the case that liquefiable soil exists in the surrounding soil, the influences of factors such as increased soil pressure and decreased frictional resistance caused by liquefaction of the surrounding soil layer shall still be considered in checking bearing capacity and stability against buoyancy.

13 Tunnel Portals

13.1 General requirements

13.1.3 According to the investigation on tunnel damage caused by the "5.12" Wenchuan earthquake, many tunnel portals were buried due to landslides, collapses and falling rocks, which also led to cracking, deformation, sliding and settlement of the retaining structure of the side and front slope, and thus interrupt the traffics. It can also be concluded that portal sections with poor geological conditions and lack in stability of side and front slope are susceptible to landslide disasters during earthquake. Tunnel portals in steep sections are vulnerable to blockage by collapse and falling rocks which have experienced long-term weathering and denudation during earthquake, and thus jeopardizing traffic safety. Therefore, it is required to take measures such as extended cut-and-cover tunnel, increased thickness of cut-and-cover tunnel backfill and providing active or passive protection nets to mitigate earthquake damage at the tunnel portal.