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JTG D50—2017（EN）

Specifications for Design of Highway Asphalt Pavement

Issue date: March 20, 2017　　Effective date: September 1, 2017

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Editing organization in charge of Chinese version: CCCC Road and Bridge Consultants Co., Ltd.
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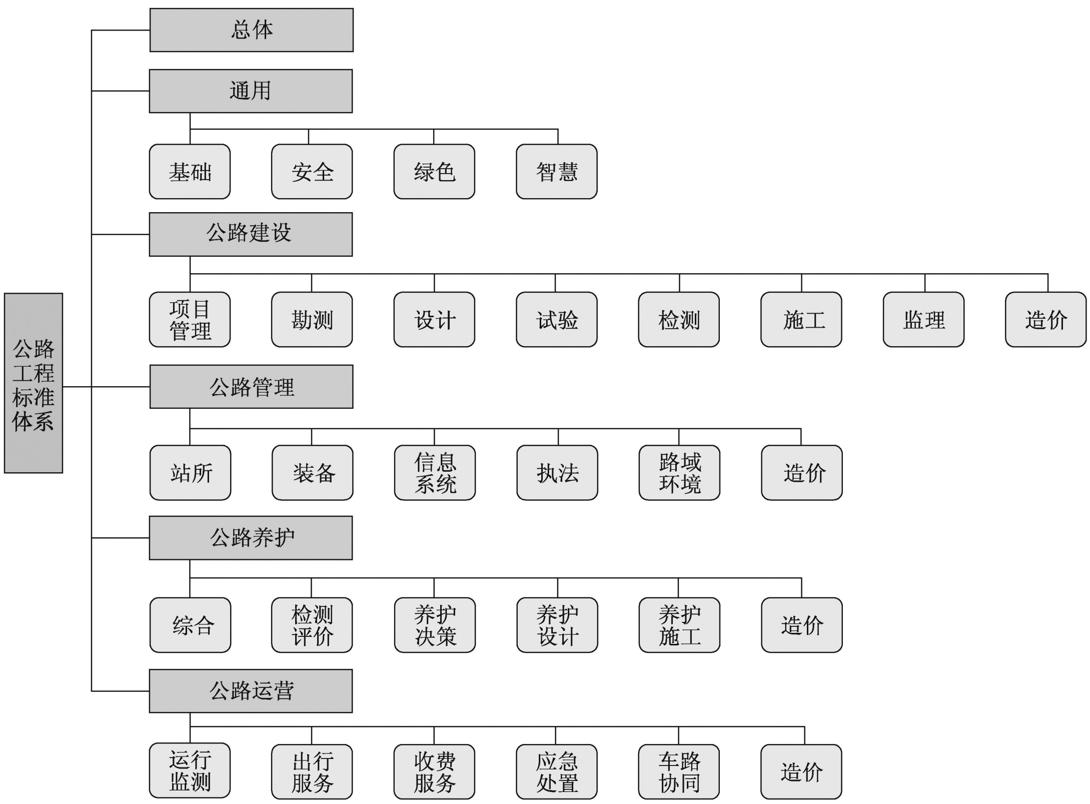
英文版编译出版说明

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。



中国政府历来高度重视交通基础设施建设，不断完善公路基础设施设计相关的标准规范。20 世纪 80 年代，中国在原《公路工程技术标准》（JTJ 01-81）基础上，开始制订公路路线、路基、路面、桥梁、涵洞等专业技术规范；1986 年，中国颁布实施了《公路柔性路面设计规范》（JTJ 014-86），并在 1997 年的第一次修订中将名称更改为《公路沥青路面设计规范》（JTJ 014-97），尔后，经历了 2006 年的第二次修订（JTG D50-2006）以及 2017 年的第三次修订（JTG D50-2017），经过近四十年的技术发展，建立了内容较为完整的公路沥青路面设计技术规范体系。本次编译的《公路沥青路面设计规范》（JTG D50-2017）中文版于 2017 年 3 月修订发布，并于 2017 年 9 月 1 日实施。

中国疆域辽阔，气候复杂多变，从北向南分别处于寒带、温带和热带，从西部的青藏高原到东部沿海地区，高程相差 4000m 以上，不同地区的经济发展存在一定差距，造成交通量差异较大。不同的自然条件、车辆车型、车辆轴型，决定着不同地区、不同等级的路面结构设计和路面材料选择存在着复杂性和差异性。同时路面的造价在道路工程总造价中的占比较高，在路面设计中除了考虑自然因素、工程因素之外还要考虑经济因素。近年来，随着中国公路建设的高速发展，中国工程技术人员在公路路面工程的设计、施工和养护等方面积累了丰富的工程经验，完善和发展了路面设计理论。这些经验与成果在《公路沥青路面设计规范》（JTG D50-2017)中得到了充分的体现.。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，为其他国家类似地形地质条件的公路建设提供参考借鉴。

本英文版的编译工作由中华人民共和国交通运输部委托中国路桥工程有限责任公司主持完成，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白琦峰先生、冯德成先生、韦金城先生在本英文版编译与审定期间给予的指导与支持。

如在执行过程中发现问题或有任何修改建议，请函告英文版主编单位（地址：北京市东城区安定门外大街丙 88 号中路大厦，邮政编码：100011，电子邮箱：kjb@crbc.com），以便修订时研用。

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The People's Republic of China
Ministry of Transport
Public Notice

No.10

Public Notice on Issuing the *Design Specifications for Highway Asphalt Pavement*

Hereby the *Specifications for Design of Highway Asphalt Pavement* (JTG/T D50-2017) is issued as one of the Industry Standards for Highway Engineering, and shall become effective on September 1, 2017. The former edition of the *Specifications for Design of Highway Asphalt Pavement* (JTG D50-2006) shall be superseded from the same date.

This edition of the *Specification for Design of Highway Asphalt Pavement* was drafted and compiled by CCCC Road and Bridge Consultants Co., Ltd. The general administration and final interpretation of the Standard shall belong to Ministry of Transport, while particular interpretation for application and routine administration of this Standard shall be provided by the CCCC Road and Bridge Consultants Co., Ltd.

Comments, suggestions and inquiries are welcome and should be addressed to the CCCC Road and Bridge Consultants Co., Ltd. (88D Andingmenwai Dajie, Jiangsu Building, Beijing, Postal Code: 100011). The feedback will be considered in future revisions.

It is hereby announced.

Ministry of Transport of the People's Republic of China

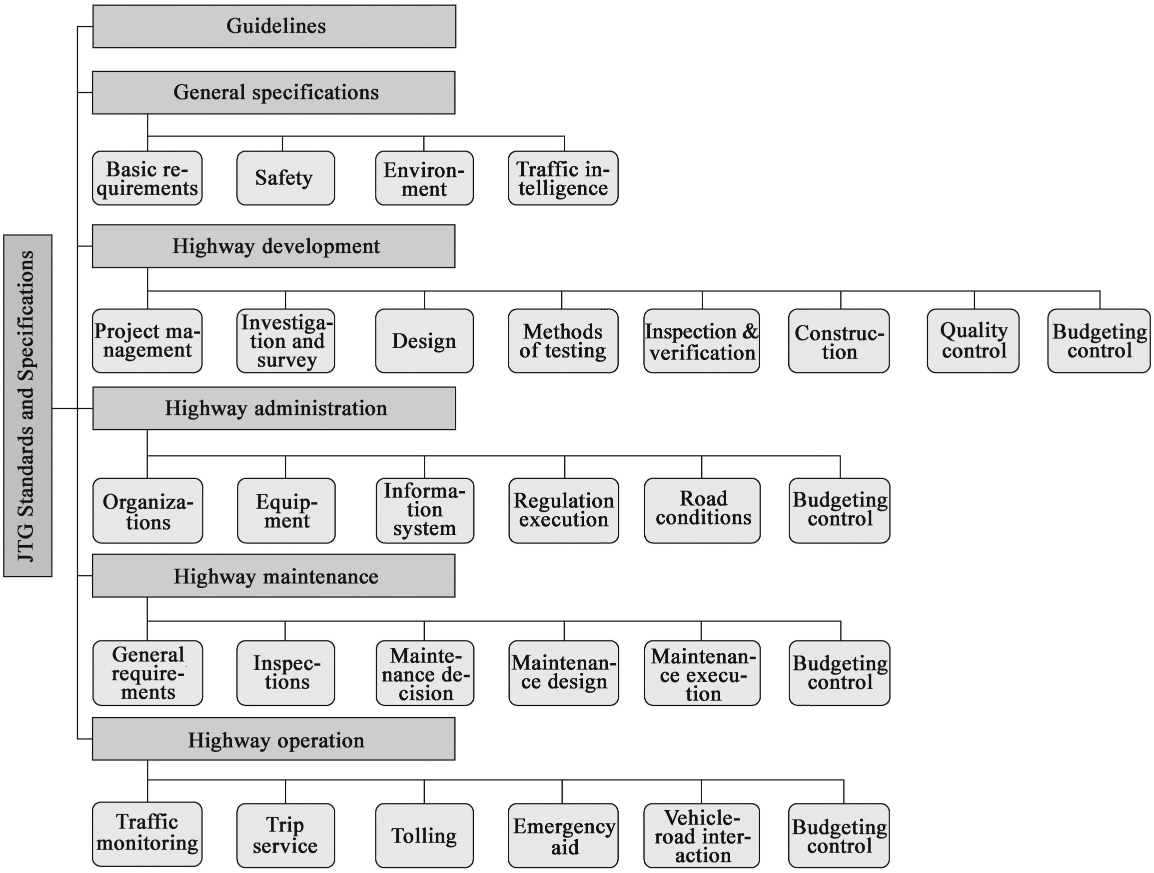
March 20, 2017

General Office of Ministry of Transport Printed on March 22, 2017

Foreword

Standards reflect the achievement of civilization, provide common language for technical communications, and improve global connectivity. In recent years, the Chinese government has been proactively implementing a strategy on standardization to stimulate innovation, coordination, greening, opening up and sharing for reciprocal development in China and worldwide. In the light of mutual development along the Silk Road economic belt and the 21st-century maritime Silk Road (so called ‘one belt one road’ initiative), the Ministry of Transport of the People’s Republic of China organized translation and published an international version of the Chinese transportation industry standards and specifications to cater for the increasing demands for international cooperation in world transportation, achieve interconnected development and promote knowledge dispersion and sharing experience.

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



Transportation plays a pioneering role in the national economic development. The technologies and industry standards for highway infrastructure have been innovatively developed and continuously updated and improved. In the early 1980's, the ***Technical Standards for Highway Engineering (JTJ 01-81)*** was formulated, followed by a series of technical specifications covering highway planning, geometry, earthworks, pavements and structural work. The ***Specifications for the Design of Highway Flexible Pavement (JTJ 014-86)*** was published and implemented in 1986, and then renamed as ***Specifications for Design of Highway Asphalt Pavement (JTJ 014-97)*** in its first revision in 1997. The second revision (JTG D20-2006) was conducted in 2006. A sophisticated system for the design of highway asphalt pavement has been fully established based on 40-years of technical development. The third revision (JTG D20-2017) was issued in September 2018 and put into effective use on January 1, 2018.

The territory of China is extensive with a great variety in climate, which crosses a freezing zone, temperate zone and tropical zone from the north to south, with an altitude difference as much as 4000 meters from the Tibet Plateau in the west to coastal plain in the east. The economic development level in China, and thus highway traffic volumes, are also different in areas across the country. The variety in environment, types of vehicles and vehicle axle combinations requires different pavement design and material selection. Because of the high contribution of pavement cost in road construction, economic considerations are always taken into account in addition to the local conditions and engineering issues. In recent years, wide experience has been gained during the rapid development of highway transportation infrastructure in China, based on which, the pavement design theories have been improved and further developed. These have been summarized and absorbed in these *Specifications for Design of Highway Asphalt Pavement* (JTG D50-2017). The English version of these specifications are therefore issued for the purpose of sharing Chinese experience with the professionals in the world and provide references for highway pavement design for similar geography, climate and geology of other countries.

The English translation of these specifications was conducted by China Road and Bridge Corporation under the authorization of the Ministry of Transport and approved by the Highway Administration of the Ministry of Transport.

The content of the English version are exactly the same as those in the Chinese version. In event of any ambiguity or discrepancies, the Chinese version should be referred to and accepted.

Gratitude is given here to Mr. Bai Qifeng, Mr, Feng Decheng and Wei Jincheng, the Editors in charge of the Chinese version, for the valuable assistance and suggestions during the translation 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: China Road and Bridge Corporation (Address: 88C AndingmenwaiDajie, Postal Code: 10011, E-mail: kjb@crbc.com). The feedback shall be considered in future revisions.

Organization in charge：	China Road and Bridge Corporation
Lead editors:	Yao Haidong, Lu Shan
Associate editors:	Zhang Huiyu, Zeng Feng, Zhang Qinghong
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Preface to Chinese Version

The highway infrastructure, especially the design and construction of asphalt pavement has been rapidly developed in last decades. The application of new pavement materials and new pavement structures have widened engineering choice and enriched implementation practice. These raised needs for revising and updating the industrial standard JTG D50-2006, 'Specifications of Design for Highway Asphalt Pavement', (refer to as 'the previous edition' hereinafter). Authorized by Ministry of Transport of China, the mission was undertaten and fulfilled by CCCC Road and Bridge Consultants.

Supported by domestic and international technical researches and engineering practice and based upon the principles of heritance and development, this edition has made updating and revision in the traffic and emvironmental factors, design parameters, design criteria and relevant performace models.

This edition of the specifications comprises 8 chapters and 7 appendices, which cover design criteria, structural design, requirements for material design parameters, calculation and verification of pavement structures, design for rehabilitation, design for bridge deck pavement, etc. In comparison with the former edition, major changes made in this Edition of the Specifications are as follows:

1 the approaches to surveying and analyzing axle load spectra and other traffic parameters have been regulated.

2 the temperature shift factors and equivalent temperature have been introduced.

3 the design parameters for pavement materials have been altered and corresponding approaches for calibration and validation have been adjusted.

4 New provisions have been added in terms of permanent deformation of asphalt bournd layer, vertical compressive strains at the top of subgrade and low temperature thermal cracking index. Improvement has been made to the prediction model of fatigue cracking of asphalt bound layers and chamecally stabilized layers. Criteria of road surface deflection for pavement design have been abolished.

5 Chapters and sections have been reorganized with highlighting on the requirements for pavement structural design. Terms and symbols have been further regulated.

Chapter 1, 2 and 3 of this edition of the specifications were written by Liu Boying, Chapter 4 by Meng Shutao, Chapter 5 by Niu Kaimin, Chapter 6 and Appendix B by Bai Qifeng, Chapter 7 by Cao Rongji, Chapter 8 and Appendices D and E by Wang Lin; Appendix A waw drafted by Zhao Yanqing, Appendix C by Yang Xueliang, Appendix F by Sun Liqun, Appendix G by Tan Zhimin, The provisions on deflection analysis was written by Tang Bomin, provisions on pavment low-temporature thermal cracking was written by Feng Decheng, the provisions on mechanistic parameters for asphalt bourn materials and the fatigue analysis for asphalt bournd layers was written by Yu Jiangmiao. Zhao Duijia partially participated in the drafting for chapter 5, and Tai Diancang participated in the drafting of Appendix C.

Questions and comments are welcome and should be addressed to the administration team of these specifications, contact person: Liu Boying (mail address: D88 Andingmenwai Dajie, Beigjin China, Post code:100011; Tel. 8610-82016537; Fax:8610-82016573; E-mail: goodpave@163.com; Website: goodpave.com; Wechat: goodpave.) Feedbacks shall be considered in future updating.

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1 GENERAL PROVISIONS

1.0.1 These specifications are formulated to cope with the needs for development of highway industry and highway infrastructure, to improve design quality and service performance of asphalt pavement so as to ensure safety, durability, economical, and fitness to purposes of the highway works.

1.0.2 These specifications are applicable to asphalt pavement design for new construction and rehabilitation of classified highways.

1.0.3 Pavement structures, pavement materials and pavement thicknesses shall be designed in accordance with the classification of the highway, the required pavement service performance and the traffic volume to be accommodated, considering physical factors such as climate, hydrology, geology, materials, and taking construction and maintenance conditions, relevant practical experiences and environmental protection requirements into account. A design strategy shall be selected through technical and economical evaluation on various alternatives.

1.0.4 Highway subgrade shall be in compliance with the lowest criteria for resilient modulus and shall be maintained at a suitable moisture level. Site investigations shall be carried out to identify local geotechnical conditions and moisture levels, based on which an integrated design for both subgrades and pavements shall be conducted.

1.0.5 Newly developed technologies, structures, materials and construction methods are encouraged and shall be adopted in a proactive but prudent way based upon local environments and practical experiences.

1.0.6 A pavement design for the areas of desert, expansive soil or salty soil shall not only comply with the provisions in these specifications, but also embrace specific technical measures based on local practical experience and research achievement to effectively solve the soil problems unique to the region.

1.0.7 Besides these specifications, asphalt pavement design shall comply with relevant provisions in prevailing national and industrial standards.

2 Terms and Symbols

2.1.1 asphalt pavement

The pavement with an asphaltic surface course.

2.1.2 Reliability

The probability that a pavement structure will perform its intended function over a specified period of time and under certain conditions. Target reliability is defined as the reliability required for a designed structure to achieve.

2.1.3 Reliability index

A numerical criterion on the reliability of pavement structures. A target reliability index is defined as the one specified in these specifications to be used as basic data for pavement structural designs.

2.1.4 Pavement design life

The predetermined length of time for which a pavement performs in normal conditions of design, construction, operation and maintenance until structural rehabilitation is required.

2.1.5 Design axle loads

Calculating axle load adopted for pavement structural design

2.1.6 Equivalent single axle loads

The number of design axle loads that are equivalent in damaging effect on a pavement to a given vehicle or axle loading

2.1.7 Cumulative equivalent single axle loads（Cumulative ESAL）

The sum of equivalent single axle loads (ESAL) applied on the design lane for the period of design life.

2.1.8 seal coat

A function layer in pavement structure, which prevents downward water infiltration.

2.1.9 Tack coat

A function layer or membrane providing bond in a pavement structure.

2.1.10 prime coat

A function layer or membrane applied on the surface and penetrated into a certain depth of a non-asphalt material layer, to improve integration of non-asphlt layer and asphalt bound layer.

2.1.11 drainage layer

The function layer for draining away the water infiltrated into a pavement structure.

2.1.12 frost protection layer

The function layer for preventing frost within a pavement structure.

2.1.13 subgrade equilibrium moisture

The subgrade moisture that reaches to stable equilibrium state with surrounding environmental moisture.

2.1.14 low temperature thermal cracking index

The indicator that describes the level of shrinkage cracking at low temperature.

Cl——low temperature thermal cracking index

E——modulus

G^*——binder complex shear modulus

h——Thickness

l——deflection

N——number of axle loads

P——axle loads

R——strength

R_a——permanent deformation of asphalt mixture

S_t——asphalt stiffness modulus

s——standard deviation

T——temperature

ε—— strain

σ——stress

β——target reliability index

3 Design Criteria

3.0.1 The target reliability and target reliability index of a pavement structure shall not be less than the criteria specified in Table 3.0.1.

Table 3.0.1 Target Reliability and Target Reliability Index

Highway Classification	Motorway	Class-1 Highway	Class-2 Highway	Class-3 Highway	Class-4 Highway
Target Reliability （%）	95	90	85	80	70
Target Reliability Index β	1.65	1.28	1.04	0.84	0.52

3.0.2 The design life of a new asphalt pavement shall be determined in accordance with the highway classification, economics and traffic load class and shall not be shorter than the criteria listed in Table 3.0.2

Table 3.0.2 Pavement Design Life (in years)

Highway Classification	Design Life	Highway Classification	Design Life
Motorway, Class-1 highway	15	Class-3 highway	10
Class-2 highway	12	Class-4 highway	8

3.0.3 Pavement design shall use a 100kN single axle-dual tire load as the design axle load. Calculation parameters shall be determined in accordance with Table 3.0.3 below. The cumulative equivalent single axle load (cumulative ESAL) shall be determined by the pavement design life in accordance with Appendix A of these Specifications.

Table 3.0.3 Parameters of design loads

Design loads (kN)	Tire contact pressure (MPa)	Equivalent diameter of single tire contact area（mm）	Dual Tire spacing（mm）
100	0.70	213.0	319.5

3.0.4 The traffic loads applied on pavement structure shall be classified as shown in Table 3.0.4.

Table 3.0.4 Classification of design traffic load

Traffic load classes	Extra heavy	Very heavy	heavy	medium	light
Cumulative Bus and Truck applications （$\times 10^6$，vehicle）in the design lane during design life	≥50.0	50.0~19.0	19.0~8.0	8.0~4.0	< 4.0

Note: Buses and trucks are the vehicles Class 2 to Class 11 as listed in Table A.1.2 of Appendix A of these Specifications.

3.0.5 Asphalt pavement design shall provide controls on fatigue cracking of asphalt bound material layers or chemically stabilized material layers, permanent deformation in asphalt bound layers, vertical compressive strain on subgrade, and low temperature thermal cracking in seasonal frost regions.

3.0.6 The criteria for pavement performance design shall comply with the following requirements.

1 The fatigue life of an asphalt bound layer or a chemically stabilized layer, which is predicted in accordance with Appendices B.1 or B.2, shall not be less than the cumulative ESAL during the design life.

2 For asphalt bound material layers, the permanent deformation calculated in accordance with Appendix B.2 of the Specifications, shall not greater than the allowable amount of permanent deformation.

Table 3.0.6 Allowable total permanent deformation of asphalt bound layers

Base	Allowable total permanent deformation of asphalt bound layers	
	Motorways, Class-1 highways	Class-2 and class-3 highways
Chemically stabilized base, Cement concrete base and asphalt bound base on chemically stabilized subbase.	15	20
Other types of base	10	15

3 The vertical compressive strain on the subgrade shall not be greater than the allowable values calculated in accordance with Appendix B.4.

4 Low temperature thermal cracking indices, calculated in accordance with Appendix B.5 of the Specifications for seasonal frost regions, should not be greater than the values listed in Table 3.0.6-2.

Table 3.0.6-2 Low Temperature Thermal Cracking Indices

Classified Highways	Motorway and Class-1	Class-2	Class-3 and Class-4
Low temperature thermal cracking indices *CI*	3	5	7

Note: Low temperature thermal cracking indices Cl – number of transverse cracks within a 100m survey unit to be measured during taking-over inspection. A transverse crack that crosses over the whole roadway is counted as one crack, whereas a crack that does not cross over the roadway but is longer than the width of one traffic lane is counted as 0.5 crack; where the length is shorter than one lane width, the crack shall not be counted.

3.0.7 For motorways, class-1 highways, and the class-2 or class-3 highways in mountainous or hilly terrains, taking-over inspections shall ensure the skid resistance of pavement in compliance with the criteria in Table 3.0.7.

Table 3.0.7 Criteria for skid resistance

Annual average rainfall (mm)	Criteria for taking-over inspection	
	Side-way force coefficient SFC_{60}[a]	Texture depth TD[b] (mm)
>1000	≥54	≥0.55

500~1000	≥50	≥0.50
250~500	≥45	≥0.45

Note: a. Sideway force coefficient (SFC60)——to be measured by Sideway-force Coefficient Routine Investigation Machine at speed of 60±1km/h.

b. Texture depth (TD) ——to be measured by sand patch method.

4 PAVEMENT STRUCTURAL DESIGN

4.1.1 The structural design of a pavement shall be based on a full understanding of the characteristics of various pavement structures in terms of mechanistic responses, service functions and long-term deterioration and damage processes, and shall be guided by the concept of systematic approach for the subgrade-pavement interaction in order to provide a pavement with safety and durability and life-cycle cost effectiveness.

4.1.2 A pavement structure may be a combination of surface course, base course, subbase course and selected or functional layers as required. Where the upper asphalt layer is paved separately in layers of different materials, the surface course may be further divided into wearing course, intermediate course and binder course.

4.1.3 Within the period of design life, the pavement shall have no structural damage by fatigue, for which only the repairing of surfacing functions may be conducted.

4.1.4 A tack coat shall be applied between any two asphalt bound layers. A seal coat, with or without prime coat, shall be applied between an asphalt bound layer and any other material layer.

Note of English version: a penetrating prime coat will inhibit strength gain of a stabilized layer and should not be used."

4.1.5 Waterproofing or drainage facilities shall be installed to prevent or reduce the infiltration of precipitation into pavement structures.

4.2.1 A pavement structure shall be selected in accordance with traffic load classification, subgrade conditions and other relevant factors, and by taking the pavement material properties and structural characteristics into account.

4.2.2 Asphalt pavement structures may be classified into four types in terms of the material properties of the pavement base, that is, asphalt pavement with chemically stabilized base, asphalt pavement with unbound granular base, asphalt pavement with asphalt bound base and asphalt pavement with cement concrete base.

The selection of pavement structures should comply with the following criteria:

1 An asphalt pavement with a chemically stabilized base is suitable for all classified traffic loads;

2 An asphalt pavement with an unbound granular material base is suitable for heavy or lower classified traffic loads

3 An asphalt pavement with an asphalt bound base is suitable for all classified traffic loads.

4 An asphalt pavement with a cement concrete base is suitable for the heavy, very heavy and extra-heavy classified traffic loads.

4.2.4 An unbound material base or an additional subgrade improvement layer of unbound material should be selected where the subgrade is in wet or moderately wet status.

4.2.5 In high rainfall regions, special measures shall be arranged for controlling pumping and the resultant void formation and other water initiated distresses of pavements where chemically stabilized materials or cement concrete slabs are used as pavement base,

4.2.6 If chemically stabilized materials are selected as pavement base, one or several of the following measures may be selected for controlling shrinkage cracking in the base course and resultant reflection cracking:

1 to select the chemically stabilized materials with high cracking resistant properties as base course;

2 to increase the thickness of asphalt bound layers or to add a layer of either asphalt treated crushed stones or graded unbound aggregates on top of the chemically stabilized base;

3 to place an additional layer of modified asphalt as a stress absorption membrane interlayer (SAMI) or a layer of geosynthetic material on top of the chemically stabilized base.

4.2.7 After the pavement structure is selected, the thickness of each structural layer may be determined in accordance with traffic loads classification as described in Appendix C of these Specifications.

4.3.1 The highway subgrade shall be stable, well compacted and uniform, and shall have sufficient bearing capacity.

4.3.2 For road sections in earth or soil cuttings and cuttings with severely weathered geology in high rainfall areas, special attention shall be paid to the drainage design of transition segments between cutting and embankment fill for the purpose of improving moisture conditions of the subgrade.

4.3.3 For rock cuttings or rock fill sections, a leveling layer about 200~300mm shall be placed beneath the subbase course.

4.3.4 The roadbed of a new highway shall be in dry or moderately wet condition. Specific measures shall be taken to prevent subgrade from infiltration of either surface water or ground water.

4.4.1 The base and subbase of a pavement shall have sufficient bearing capacity and fatigue resistance, as well as sufficient durability and moisture resistance. In addition, the asphalt bound base or chemically stabilized base shall have sufficient resistance to permanent deformation.

4.4.2 Base or subbase materials may be selected by referring to Table 4.4.2.

Table 4.4.2 Materials suitable for Traffic Loads and Structural Layers

Type	Material	Suitable Traffic Loads and Structural Layers
Chemically stabilized	Cement stabilized graded aggregate Cement-fly ash stabilized graded aggregate Lime-flyash stabilized graded aggregate	Base and subbase for all classes of traffic loads
	Cement stabilized all-in aggregate Lime-flyash stabilized all-in aggregate Lime stabilized all-in aggregate	Base for light traffic loads Subbase for all classes of traffic loads
	Cement, lime or lime-flyash stabilized soil	Base for light traffic loads Subbase for all classes of traffic loads
Unbound granular	Graded crushed stone	Base for heavy or lighter traffic loads Subbase for all classes of traffic loads
	Graded gravel All-in aggregates, natural gravels	Base for medium or light traffic loads Subbase for all classes of traffic loads

	Macadam	
Asphalt bound	Dense graded, asphalt treated crushed stone Semi-open graded, asphalt treated crushed stone Open-graded, asphalt treated crushed stone	Base for extra, very heavy or heavy traffic loads
	Bitumen penetration macadam	Base for heavy or below heavy traffic loads
Cement concrete	Cement concrete or lean concrete	Base for extra or very heavy traffic loads

4.4.3 Recycled asphalt bound materials and recycled chemically stabilized materials may be used as base or subbase for all classes of traffic loads. The recycled asphalt bound materials to be used as base course for extra heavy, very heavy or heavy class of traffic loads should be processed by plant hot mixing.

4.4.4 A layer of graded unbound crushed stone or a layer of open graded or semi-open graded asphalt treated crushed stone may be placed between the chemically stabilized layer and the asphalt bound layer.

4.4.5 The thickness of base and subbase of various materials should comply with the criteria listed Table 4.4.5.

Table 4.4.5 Thickness of base and subbase.

Material	Norminal maximum aggregate size (NMAS) (mm	Minimum thickness(mm)
Dense graded, asphalt treated crushed stone, Semi-open graded, asphalt treated crushed stone, Open graded, asphalt treated crushed stone	19.0	50
	26.5	80
	31.5	100
	37.5	120
Bituminous penetration macadam	/	40
Lean concrete	31.5	120
Chemically stabilized materials	19.0、26.5、31.5、37.5	150
	53.0	180
Graded crushed stone Graded gravel All-in aggregates, natural sand and gravel	26.5、31.5、37.5	100
	53.0	120
Macadam	37.5	75
	53.0	100
	63.0	120

4.4.6 The cement concrete base of an asphalt pavement shall comply with relevant provisions of the current JTG D40 Design Specifications for Highway Cement Concrete Pavements.

4.5.1 The surface course shall have properties of evenness and resistance against rutting, fatigue, low temperature thermal cracking and water damaging. The asphalt mixture materials of a wearing course shall have the properties of skid resistance and wearing resistance, while a dense graded asphalt mixture of wearing course shall have the property of low permeability.

4.5.2 The types of surfacing materials should be selected in accordance with Table 4.5.2.

Table 4.5.2 Surfacing Materials suitable for Traffic Loads and Structural Layers

Type of Material	Suitable Traffic Loads and Structural Layers
Continuously graded asphalt mixture	Surface layer, intermediate layer or binder layer for all classes of loads
Stone matrix asphalt (SMA) mixture	Surface layer for extra heavy, very heavy or heavy class of traffic loads, or Surface layer for special skid resistance requirements
Plant hot recycled asphalt mixture	Surface layer, intermediate layer or binder layer for all classes of loads
Hot mix on bituminous macadam	Surface course for medium or light class of traffic loads
Bituminous surface dressing	Wearing courser for medium or light class of traffic loads

4.5.3 An open graded bituminous mixture may be used as surface layer for specific requirements such as skid resistance, drainage or noise reduction. A seal coat layer shall be placed under the porous surface layer, which may be made of modified bituminous emulsion or modified asphalt with small chips. (NOTE for EN)

4.5.4 The layer thickness of an asphalt mixture for various nominal sizes shall be in compliance with the criteria specified in Table 4.5.4. The layer thickness of either continuously graded asphalt mixture or stone matrix asphalt (SMA) mixture should not be smaller than 2.5 times its nominal maximum aggregate size (NMAS). The layer thickness of an open graded asphalt mixture should not be less than 2.0 times its nominal maximum aggregate size (NMAS).

Table 4.5.4 Layer thickness of asphalt mixtures for different nominal size

Type of asphalt mixture	Minimum layer thickness of asphalt mixture for nominal maximum aggregate size (mm)					
	4.75	9.5	13.2	16.0	19.0	26.5
Continuously graded	15	25	35	40	50	75
Stone matrix	-	30	40	50	60	-
Open graded	-	20	25	30	-	-

4.5.5 The thickness should be 40～80mm for a layer of bitumen penetration macadam and not more than 50mm for a layer of bituminous emulsion penetration macadam. The thickness of hot mix asphalt laid on top of a bituminous penetration layer should not be less than 25mm.

4.5.6 Asphalt surface dressing may be in one, two or three layers. The thickness should be 10～15mm for single surface dressing, 15～25mm for double surface dressing, and 25～30mm for triple surface dressing.

4.6.1 In seasonal frost regions, an additional layer for frost protection shall be placed if the pavement thickness does not satisfy the frost requirements. A frost protection layer should be made of coarse sand, sand gravel, crushed stone or other granular materials.

4.6.2 For the road sections with a high water table and unfavorable drainage conditions or for the rock sections in cutting with fracture groundwater, springs or other adverse hydraulic conditions, an additional layer of granular materials shall be placed between base or subbase and subgrade if such a base or subbase is made of non-granular materials. This unbound granular layer shall be extended to subgrade edges or connected with the filter trenches beneath side drains.

4.6.3 A seal coat should be placed between the chemically stabilized or cold recycled material layer and the asphalt bound layer. A seal coat can be a single surface dressing or slurry seal. A seal coat may not be placed in the case where there is a modified asphalt layer for stress absorption in the pavement structure.

4.6.4 Modified bituminous emulsion, penetration grade bitumen or modified bitumen should be selected and used as tack coats for extra heavy, very heavy or heavy class of traffic loads; while bitumen emulsion may be used as tack coats for medium or light class of traffic loads; modified asphalt should be used as the tack coat between a cement concrete slab and an asphalt surface course.

4.6.5 Modified asphalt, penetration grade bitumen or bituminous emulsion may be used as the binder for seal coals on a single surface dressing layer. Rubberized bitumen should be used in the stress absorbing membrane interlayer (SAMI) beneath a modified asphalt pavement.

4.6.6 A prime coat should be provided on the surface of either a granular base or a chemical stabilized base. The bitumen used for prime coating shall have good penetration, for which cutback bitumen or bitumen emulsion may be used .

4.7.1 The structure and material selected for road shoulders shall be consistent with the pavement of the roadway and shall not impede infiltrated water from flowing out of the pavement structure.

4.7.2 For the classified highways under extra-heavy, very-heavy and heavy classes of traffic loads, or highway sections in frost areas, the materials and thicknesses of the base and subbase of a hard shoulder shall be the same as those of the roadway pavement.

4.7.3 Asphalt bound or granular materials can be used as the hard shoulders of Class-3 or Class-4 highways.

4.8.1 Pavement subsurface drainage shall be connected to the other relevant drainage systems of the highway, and shall comply with the current JTG/T D33 Design Specifications for Highway Drainage.

4.8.2 An open graded asphalt mixture, a layer of granular, open graded or semi-open graded aggregate should be placed for drainage. Such a drainage layer may be either over the full width of the subgrade or connected to the edge drainage system.

5 Material Design Parameters

5.1.1 The design parameters for pavement materials shall be determined in accordance with highway classification, traffic loads, climate conditions, the functional requirements and the properties of local materials and on the basis of technical and economical evaluation.

5.1.2 The requirements for the properties of natural materials and the requirements for the composition and properties of material mixtures for each structural layer of a pavement shall comply with the current *JTG F40 Specifications for Construction of Highway Asphalt Pavement* and *JTG/T F20 Specifications for Construction of Base Course of Highway Pavement*, and shall be determined by taking the project characteristics and the experience of local practice into account.

5.1.3 The determination of design parameters for the materials of pavement structural layers may be conducted at three levels as follows:

1 Level-A: Laboratory Testing

2 Level-B : Empirical Correlation Formula

3 Level-C: Typical Values

5.1.4 For motorways and class-1 highways, Level-A should be adopted at the stage of construction drawing design; either Level-B or Level-C may be adopted for other design stages. For Class 2 and lower class highways, Level-B or Level-C may be adopted.

5.2.1 Resilient modulus of the subgrade shall be in compliance with the current JTG D30 Specifications for Design of Highway Subgrade.

5.2.2 The resilient modulus of the subgrade shall comply with the criteria in Table 5.2.2. Otherwise special treatments, such as changing the fill materials, placing a subgrade improvement layer of granular or chemically stabilized materials, shall be arranged to increase the resilient modulus of the subgrade.

Table 5.2.2 The requirements for the resilient modulus at the top of subgrade(MPa)

Traffic Loads Classification	Extra-heavy	Very-heavy	Heavy	Medium/Light
Resilient Modulus, not less than	70	60	50	40

5.3.1 The values of CBR (California Bearing Ratio) shall be in compliance with the criteria in Table 5.3.1.

Table 5.3.1 Requirements for the soaked CBR of Graded Crushed Stone

Structural layer	Highway Classification	Extra-heavy, Very heavy traffic	Heavy traffic	Medium/Light traffic
Base	Motorways, Class-1 highways	≥200	≥180	≥160
	Class-2 and lower class highways	≥160	≥140	≥120
Subbase	Motorways, class-1 highways	≥120	≥100	≥80
	Class-2 and lower class highways	≥100	≥80	≥60

5.3.2 For graded crushed stone or natural gravels, the CBR value of the base shall not be lower than 80; the CBR value of the subbase shall not be lower than 80 in the case of Extra-heavy, Very-heavy and Heavy traffic loads;

not lower than 60 in the case of Medium traffic loads; and not lower than 40 in the case of Light traffic loads.

5.3.3 For motorways and class-1 highways, the nominal maximum size of a granular material base should not be larger than 26.5mm. The nominal maximum size of a crushed stone subbase or a gravel macadam subbase should not be larger than 32.5mm. The nominal maximum size of a granular material subbase should not be larger than 53.0mm.

5.3.4 The nominal maximum size of the coarse aggregates of macadam should be about 1/2~2/3 of the layer thickness and shall not be larger than 53.0mm where it is used as base, or not be larger than 63.0 mm in the case of subbase.

5.3.5 The nominal maximum size of the gravel or crushed stone used as a frost protection layer shall not be larger than 53.0mm.

5.3.6 In the graded crushed stone or graded gravel, the percentage passing the 0.075mm sieve should not be greater than 5%. Otherwise a part of the fine aggregates may be replaced with natural sand.

5.3.7 For structural calculation and verification, the resilient modulus of the granular material layer to be used shall be obtained as the laboratory resilient modulus of the granular material multiplied by a moisture adjustment factor. The moisture adjustment factor may be taken in a range of 1.6~2.0. The resilient modulus of granular material shall be taken from the results of laboratory tests under the conditions of optimum moisture content and the corresponding dry density for required compaction. The required compaction shall comply with the relevant criteria in *JTG/T Guidelines for Construction of Highway Pavement Base Course*.

5.3.8 The resilient modulus of granular materials under optimum moisture content and dry density corresponding to required compaction shall be determined by the corresponding level as specified in Clause 5.1.4 of this edition of the Specifications.

1 Level-A: Values are measured by means of a repeated load triaxial compressive test as described in Appendix D, and the average value of the test results is used as resilient modulus.

2 Level-C: the value of resilient modulus is taken from Table 5.3.8 below in accordance with the type and layer position the material.

Table 5.3.8 Value ranges of resilient modulus of granular material (MPa)

Type and Layer of Material	Under optimum moisture content and dry density corresponding to required compaction	After moisture adjustment
Base of crushed stone macadam	200~400	300～700
Subbase of crushed stone macadam	180~250	190～440
Base of gravel macadam	150~300	250～600
Subbase of gravel macadam	150~220	160～380
Layer of unsieved crushed stone	180~220	200～400
Layer of natural gravels	105~135	130～240

Note: a higher value is taken for the material with good properties, good grading and high compaction, otherwise a lower value should be used.

5.4.1 The nominal maximum size of chemically stabilized material should be not larger than 31.5mm for the base and not larger than 37.5mm for the subbase of a motorway and a class-1 highway. The norminal maximum size should not be larger than 37.5mm for the base and not be larger than 53.0mm for the subbase of class-2 and lower class highways.

5.4.2 The cement content should be 3.0%~6.0% of cement stabilized materials.

5.4.3 The nominal maximum size of the aggregate in lean concrete should not be larger than 31.5mm; the cement content should not be less than 170kg/m^3; the standard value of 28d flexural strength should be controlled within the range of 2.0~2.5MPa.

5.4.4 The representative value of the 7-days unconfined compressive strength of a chemically stabilized material shall comply with the criteria shown in Table 5.4.4 below.

Table 5.4.4 The 7-day Unconfined Compressive Strength of Chemically Stabilized Materials（representative values, in MPa）

Material	Structural layer	Highway Classification	Traffic Load Class		
			Very heavy, Extra heavey	Heavy	Medium, light
Cement Stabilized	Base	Motorway, Class-1 highway	5.0～7.0	4.0～6.0	3.0～5.0
		Class-2 and lower class highway	4.0～6.0	3.0～5.0	2.0～4.0
	Subbase	Motorway, Class-1 highway	3.0～5.0	2.5～4.5	2.0～4.0
		Class-2 and lower class highway	2.5～4.5	2.0～4.0	1.0～3.0
Cement Fly-ash Stabilized	Base	Motorway, Class-1 highway	4.0～5.0	3.5～4.5	3.0～4.0
		Class-2 and lower class highway	3.5～4.5	3.0～4.0	2.5～3.5
	Subbase	Motorway, Class-1 highway	2.5～3.5	2.0～3.0	1.5～2.5
		Class-2 and lower class highway	2.0～3.0	1.5～2.5	1.0～2.0
Lime Fly-ash Stabilized	Base	Motorway, Class-1 highway	≥1.1	≥1.0	≥0.9
		Class-2 and lower class highway	≥0.9	≥0.8	≥0.7
	Subbase	Motorway, Class-1 highway	≥0.8	≥0.7	≥0.6
		Class-2 and lower class highway	≥0.7	≥0.6	≥0.5
Lime Stabilized	Base	Class-2 and lower class highway	-	-	≥0.8[a]
	Subbase	Motorway, Class-1 highway	-	-	≥0.8
		Class-2 and lower class highway	-	-	0.5～0.7[b]

Note: a. In the areas where native soil has low plasticity indices, the 7-day unconfined compressive strength of cement stabilized gravel and crushed stone shall be higher than 0.5MPa (measured by 100g balanced cone).

b. The lower values should be used for the clay with a plasticity index lower than 7, the higher values for those with a PI equal to or greater than 7.

5.4.5 Flexural strength and resilient modulus of chemically stabilized materials shall be determined in accordance with the corresponding level and in compliance with Clause 5.1.4 of the Specifications.

1 Level-A is measured by the uniaxial compressive test as described in Appendix E of the Specifications.

Measurement of flexural strength and resilient modulus shall comply with JTG E851 Method of Test for Chemically Stabilized Materials for Highway Works, T0851. At the time of measurement, the age of the specimens shall be 90 days for cement or cement-flyash stabilized materials, and 180 days for lime or lime-flyash stabilized materials. The average value of measured data shall be taken for flexural strength and resilient modulus.

2 Level-C is to determine flexural strength and resilient modulus by referring to Table 5.4.5.

Table 5.4.5 Flexural Strength and Resilient Modulus (MPa) of Chemically Stabilized Materials

Materials	Flexural strength	Resilient Modulus
Cement stabilized, Cement-flyash stabilized and Lime stabilized granular materials	1.5~2.0	18000~28000
	0.9~1.5	14000~20000
Cement stabilized, Cement-flyash stabilized and Lime-flyash stabilized soil	0.6~1.0	5000~7000
Lime stabilized soil	0.3~0.7	3000~5000

Note：Higher values should be taken if the stabilizer content is comparatively high, the properties and grading of the material to be stabilized are good, and/or the compaction density of stabilized material is high, otherwise lower values should be taken.

5.4.6 For structural calculation and verification, the resilient modulus of chemically stabilized materials shall be multiplied by 0.5 that is the modulus adjustment factor of the structural layer.

5.4.7 Where the lime-flyash stabilized base course is adopted in motorways and class-1 highways in frozen ground regions, the frost-resistance shall be tested in accordance with the relevant provisions of the current JTG E851 Test Methods of Chemically Stabilized Materials for Highway Works, T0858. The ratio of retained compressive strength after the freeze-thaw test shall comply with the criteria in Table 5.4.7.

Table 5.4.7 Frost-resistance of Lime-Flyash Stabilized Materials

Climate Zone	Heavy Frost Zone	Medium Frost Zone
Ratio of Retained Compressive Strength (%)	≥70	≥65

5.5.1 The binder of asphalt bound materials shall be the penetration grade bitumen or the processed bituminous products. The type of bitumen to be adopted shall be determined in accordance with highway classification, climate, traffic load class, pavement layer and construction conditions.

5.5.2 For classified highways under extra heavy, very heavy and heavy traffic loads, highways under extreme climates, or long and steep gradient sections, an optimized grading for the mixture using modified asphalt or admixtures should be adopted for the intermediate layer and top layer of the surface course.

5.5.3 High viscosity bitumen or rubberized bitumen should be adopted for an open-grade asphalt mixture used as the upper layer in a surface course, while hydrated lime or cement in an appropriate quantity should be used to substitute mineral fillers.

5.5.4 The nominal maximum size should not larger than 16.0mm for the upper layer, but not smaller than 16.0mm for intermediate layer or binder layer of the surface course. The nominal size for bitumen macadam base should not smaller than 26.5mm.

5.5.5 In the regions of seasonal frozen ground, the performance at low temperature of the top layer of a surface course of motorways and class-1 highways should meet the criteria as follows:

1 The average of the lowest annual temperature records in 10 successive years shall be analyzed and taken as the design temperature for the pavement at low temperature. In laboratory test environment, a test temperature of 10°C above the low temperature design temperature is used. The creep compliance, S_t, determined by bending beam rheometer (BBR) tests, should not be greater than 300MPa, while the slope, m, of the creep curve should not be greater than 0.3.

2 In the case where the creep compliance, S_t falls within the range of 300~600MPa and the slope, m, of the creep curve is greater than 0.3, an additional direct tension test (DTT) is required, in which the fracture strain should not be less than 1%.

3 Where the above is not the case, the bitumen critical fracture temperature shall be determined by the bending beam rheometer (BBR) test and direct tension test which should not be higher than the design temperature for the pavement at low temperature.

5.5.6 For class-2 and higher class highways where asphalt mixture with nominal maximum size is not larger than 19.0mm, beam bending tests should be performed at a temperature of -10°C with a loading rate of 50mm/min. Failure strain of the asphalt mixture should comply with the criteria given in Table 5.5.6.

Table 5.5.6 Failure strain of Asphalt Mixture by Low Temperature Bending Test

Climate and Technical Indicators	Failure Strain (με) for Climate Zone below									Test method
Annual lowest temperature (℃) and climate zone	<-37.0		-21.5～-37.0			-9.0～-21.5		>-9.0		
	1 Extremely cold winter zone		2 Cold winter zone			3 Cool winter zone		4 Winter warm zone		
	1-1	2-1	1-2	2-2	3-2	1-3	2-3	1-4	2-4	
Norma asphalt mixture,not less than	2600		2300			2000				T 0715
Modified asphalt mixture, not less than	3000		2800			2500				

Note: Determination of the climate zone should comply with the current JTG F40 Specifications of Asphalt Pavement Construction for Highway Works.

5.5.7 For expressways and class-1 highways, surface rutting should be measured under the specified environment and in compliance with the criteria in Table 5.5.7. This is also applicable to class-2 highways.

Table 5.5.7 Dynamic Stability of Rutting of Asphalt Mixture (/mm)

Climate and Technical Indicators		Failure Strain (με) for Climate Zone below									Test method
Highest Temperature (℃) in July in Climate Zone		＞30				20～30				＜20	
		1 Extremely Hot Summer				2 Hot Summer				3 Warm Summer	
		1-1	1-2	1-3	1-4	2-1	2-2	2-3	2-4	3-2	
Ordinary asphalt mixture, not less than		800		1000		600	800			600	T 0719
Modified asphalt mixture, not less than		2800		3200		2000	2400			1800	
SMA mixture, not less than	Ordinary asphalt	1500									
	Modified asphalt	3000									
OGFC mixture, not less than		1500(for medium and light traffic loads), 3000(for heavy and higher traffic loads)									

Note： 1. Determination of the climate zone should comply with the current JTG F40 Specifications of Asphalt Pavement Construction for Highway Works.

2. The highest average temperature in the other month should be used if it is not in July.

3.The criteria for dynamic stability may be increased for special circumstances such as for steel bridge decking, long ascending road sections with comparatively large volumes of heavy traffic loads or steep gradients or special purpose industrial roads.

4. For classified highways in very hot regions or with very-heavy or higher traffic load classes, either the test temperature may be appropriately increased or the test loading increased in accordance with climate and traffic conditions.

5.5.8 The penetration strength of an asphalt mixture should be measured by the Method of Testing Uniaxial Penetration Strength of Asphalt Mixture as described in Appendix F of these specifications. For the asphalt pavement with chemically stabilized base course, the asphalt pavement with chemically stabilized subbase course and asphalt bound base course and the asphalt pavement with cement concrete base course, the penetration strength of the asphalt mixture should meet the requirements of Equation (5.5.8-1).

$$R_{\tau s} \geq \left(\frac{0.31 \lg N_{e5} - 0.68}{\lg[R_a] - 1.31 \lg T_d - \lg \psi_s + 2.50} \right)^{1.86} \tag{5.5.8-1}$$

where: $[R_a]$—— permanent deformation (mm) of asphalt mixture, determined in accordance with highway classification and by referring to Table 3.0.6-1;

N_{e5}——Accumulated number of equivalent design axle loads, calculated in accordance with Appendix A of the Specifications, for the months in which monthly average temperatures are above 0°C over the design life or in the period from the traffic opening to the first surface repair for rutting;

T_d——Design temperature (℃), which is the average of monthly temperatures above 0 ℃;

ψ_s——Pavement structure factor, calculated by using Equation (5.5.8-2) ,

$$\psi_s = \left(0.52 h_a^{-0.003} - 317.59 h_b^{-1.32}\right) E_b^{0.1} \tag{5.5.8-2}$$

Where, h_a——thickness of asphalt bound layer.

h_b——thickness of the chemically stabilized layer or the cement concrete layer (mm);

E_b——modulus of the chemically stabilized layer or the cement concrete layer (MPa);

$R_{\tau s}$——overall penetration strength of asphalt bound layers, determined by Equation (5.5.8-3)

$$R_{\tau s} = \sum_{i=1}^{n} w_{is} R_{\tau i} \tag{5.5.8-3}$$

Where $R_{\tau i}$——penetration strength of i-th layer of asphalt mixture, determined by the test method stated in Appendix F. This value is usually 0.4～0.7MPa for ordinary asphalt mixtures and 0.7～1.2MPa for modified asphalt mixtures;

n——number of asphalt bound layers

w_{is}——weighting of i-th asphalt bound layer, which is the ratio of the shear at mid-point of the thickness of i-th layer to the overall shear at mid-point of total thickness of all layers: $w_{is}=\frac{\tau_i}{\sum_{i=1}^{n}\tau_i}$

For a single asphalt layer, w_1 is 1.0; for two asphalt layers, from top down, w_1 may take 0.48 and w2 may take 0.52; for three asphalt layers, from top downwards, w1, w2 and w3 may take 0.35, 0.42 and 0.23 respectively.

5.5.9 For the asphalt pavement with a granular base course or the asphalt pavement with granular subbase course and asphalt bound base course, the penetration strength of the asphalt mixture should meet the requirements of Equation (5.5.9-1)

$$R_{\tau g}\geq\left(\frac{0.35\lg N_{e5}-1.16}{\lg[R_a]-1.62\lg T_d-\lg\psi_g+2.76}\right)^{1.38} \quad (5.5.9\text{-}1)$$

Where ψ_g——pavement structure factor, calculated by using Equation (5.5.9-2).

$$\psi_g=20.16h_a^{-0.642}+820916\,h_b^{-2.84} \quad (5.5.9\text{-}2)$$

$R_{\tau g}$——overall penetration strength of asphalt bound layers, determined by Equation (5.5.9-3).

$$R_{\tau g}=\sum_{i=1}^{n}w_{ig}R_{\tau i} \quad (5.5.9\text{-}3)$$

w_{ig}——weighting of i-th asphalt bound layer, which is the ratio of the shear force at mid-point of the thickness of i-th layer to the overall shear at mid-point of total thickness of all layers.

$$w_{ig}=\frac{\tau_i}{\sum_{i=1}^{n}\tau_i}$$

For a single asphalt layer, w_1 takes 1.0; for two asphalt layers, w_1 may take 0.44 and w2 may take 0.56 from top down,; for three asphalt layers, w1, w2 and w3 may take 0.27，0.36 and 0.37 respectively from top downwards.

Other symbols are defined the same as in Equatio (5.5.8-1) ~Equation（5.5.8-3）.

5.5.10 The retained soaked Marshall stability and the ratio of retained strength of the freeze-thaw splitting tests shall be measured for water resistance of the asphalt mixtures. These two parameters shall comply with the criteria in Table 5.5.10. If the water resistance is inadequate, the hydrated lime, cement or an anti-strip admixture may be added, or the aggregate may be replaced.

Table 5.5.10 Water resistance of Asphalt Mixtures

Type of Asphalt Mixture		Requirements corresponding to Annual Rainfall as below		Test method
		≥500	<500	
Retained stability of soaked Marshall test (%)				
Ordinary Asphalt Mixture, not less than		80	75	T0709
Modified Asphalt Mixture, not less than		85	80	
SMA Mixture, not less than	Ordinary asphalt	75		
	Modified asphalt	80		
Strength ratio of freeze-thaw splitting test (%)				
Ordinary Asphalt Mixture, not less than		75	70	T0729
Modified Asphalt Mixture, not less than		80	75	
SMA Mixture, not less than	Ordinary asphalt	75		
	Modified asphalt	80		

5.5.11 Dynamic compressive modulus shall comply with the provisions of Clause 5.1.4 and determined in accordance with a corresponding level below.

1 Level-A is to determine the dynamic compressive modulus which shall comply with the current JTG E20 Standard Test Methods of Bitumen and Bituminous Mixtures for Highway Engineering. The average values shall be taken at a test temperature of 20°C, a loading frequency of 10Hz for the surface course asphalt mixure and 5Hz for an asphalt bound base course.

2 Level-B is to calculate dynamic compressive modulus by using Equation (5.5.11), applicable to the asphalt mixture with penetration grade bitumen and regular gradingfor road paving;

$$\begin{aligned}\lg E_a &= 4.59 - 0.02f + 2.58G^* - 0.14P_a \\ &- 0.041V - 0.03VCA_{DRC} - 2.65*1.1^{\lg f} G^* \cdot f^{-0.06} \\ &- 0.05*1.52^{\lg f} VCA_{DRC} \cdot f^{-0.21} + 0.0031 f \cdot P_a + 0.0024V\end{aligned} \qquad (5.5.11)$$

Where：E_a——dynamic compressive modulus of asphalt mixture

f——test frequency (Hz)

G^*——dynamic shear complex modulus (kPa) at 60ºC and 10rad/s;

P_a——bitumen-aggregate rate (%)

V——voids in compacted asphalt mixture (%);

VCA_{DRC}——voids in coarse aggregate after dry rodding (%)

3 Level-C is the dynamic compressive modulus from Table 5.5.11.

Table 5.5.11 Value Range of Dynamic Compressive Modulus of Asphalt Mixture at 20℃

Type of Asphalt Mixture	Grade of Bitumen			
	No. 70 Bitumen for Road Paving	No. 90 Bitumen for Road Paving	No. 110 Bitumen for Road Paving	SBS Modified Bitumen
SMA10/SMA13/SMA16	-	-	-	7500~12000
AC10/ AC13	8000~12000	7500~11500	7000~10500	8500~12500
AC16/ AC20/AC25	9000~13500	8500~13000	7500~12000	9000~13500

ATB25	7000~11000	-	-	-

Note: 1 the figures for ATB25 asphalt mixture are the dynamic compressive modulus under loading frequency of 5 Hz. The other figures are all under a loading frequency of 10Hz.

2 Higher values are taken if the viscosity of the bitumen is high, the grading of material is good and the voids are comparatively small. In contrary situations lower values should be used.

5.6.1 The Poisson's Ratio shall be selected by referring to Table 5.6.1.

Table 5.6.1 Value Range for Poisson's Ratio

Type of Material	Subgrade material	Granular material	Chemically stabilized material	Dense graded asphalt mixture	Open graded asphalt mixture Semi-openly graded asphalt mixture
Poisson's Ratio	0.40	0.35	0.25	0.25	0.40

6 CALCULATION AND VERIFICATION OF PAVEMENT STRUCTURES

6.1.1 The calculation on the mechanistic parameters of a pavement structure is based on the theory of an elastic multi-layer continuous system under the action of two circular uniformly distributed vertical loads.

6.1.2 A trial structure of the pavement shall be initially proposed, on which calculation and verification shall be performed in conformance with Appendix B of the Specifications. Then the final selection of the pavement structure shall be conducted based on practical experience and economic analysis. For class-2 and lower class highways for medium or light traffic load class, a pavement structure may be selected by referring to existing structures adopted in the region.

6.2.1 The design parameters shall be selected in accordance with the proposed pavement structure by referring to Table 6.2.1.

Table 6.2.1 Design Parameters for Pavements Structures

Type of Base Course	Type of subbase	Design Parameter [a]
Chemically Stabilized	Granular	Horizontal tensile stresses at the bottom of chemically stabilized layers; Permanent deformation of asphalt bound layers
	Chemically stabilized	
Asphalt Bound	Granular	Horizontal tensile strains at the bottom of asphalt bound layers; Permanent deformation for asphalt bound layers; Vertical compressive strain on top of subgrade.
	Chemically stabilized	Permanent deformation of asphalt bound layers; Tensile stresses at the bottom of chemically stabilized layers
Granular [b]	Granular	Horizontal tensile strains at bottom of asphalt bound layers; Permanent deformation of asphalt bound layers; Vertical compressive strains at the top of subgrade.
	Chemically stabilized	Horizontal tensile strains at bottom of asphalt bound layers; Permanent deformation of asphalt bound layers; Tensile stress at bottom of chemically stabilized layers.
Cement concrete [c]	-	Permanent deformation of asphalt bound layers

Note: a. In seasonal frost regions, thermal cracking and frost depth shall be calculated and verified for the asphalt surface course.

b. Fatigue of the asphalt layer shall be calculated and verified where a granular layer is placed between an asphalt bound layer and a chemically stabilized layer.

c. Cement concrete base course shall be designed in accordance with the current JTG D40 Specifications for design of Highway Cement Concrete Pavements.

6.2.2 Design parameters to be selected for calculation and verification of a pavement structure shall be the mechanistic responses in terms of vertical positions as specified in Table 6.2.2.The maximum values of the mechanistic responses are calculated at four computational points A, B, C and D as shown in Figure 6.2.2.

Table 6.2.2 Mechanistic Response and Vertical Position of Design Parameters

Design Parameter	Mechanistic Response	Vertical Position
Tensile strains at the bottom of asphalt bound layers	Max.Horizontal tensile strains along traffic direction	bottom of asphalt bound layer
Tensile stresses at the bottom of chemically stabilized layer	Max.Horizontal tensile stresses along traffic direction	bottom of chemically stabilized layer
Permanent deformation of asphalt bound layer	Vertical compressive stresses	top of each asphalt bound layer
Compressive vertical strains at the top of subgrade	Vertical compressive strains	top of subgrade

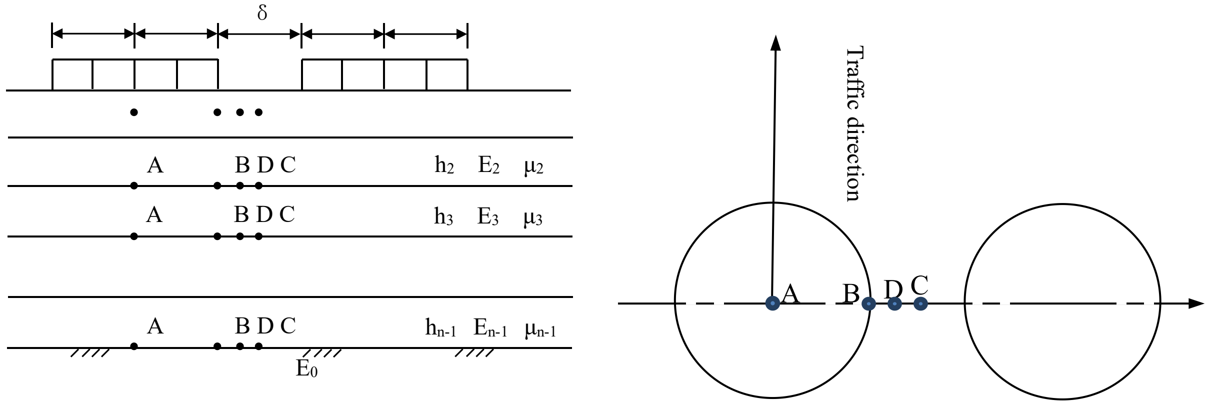


Figure 6.2.2 Position Diagram of computational Points of Mechanistic Response

6.3.1 The cumulative equivalent design axle loads during the design or structur life shall be determined from the results of a survey and analysis of traffic parameters. Calculation procedures are given in Appendix A of the Specifications.

6.3.2 For the calculation and verification of a pavement structure, the modulus of each structural layer shall be taken in compliance with the following provisions:

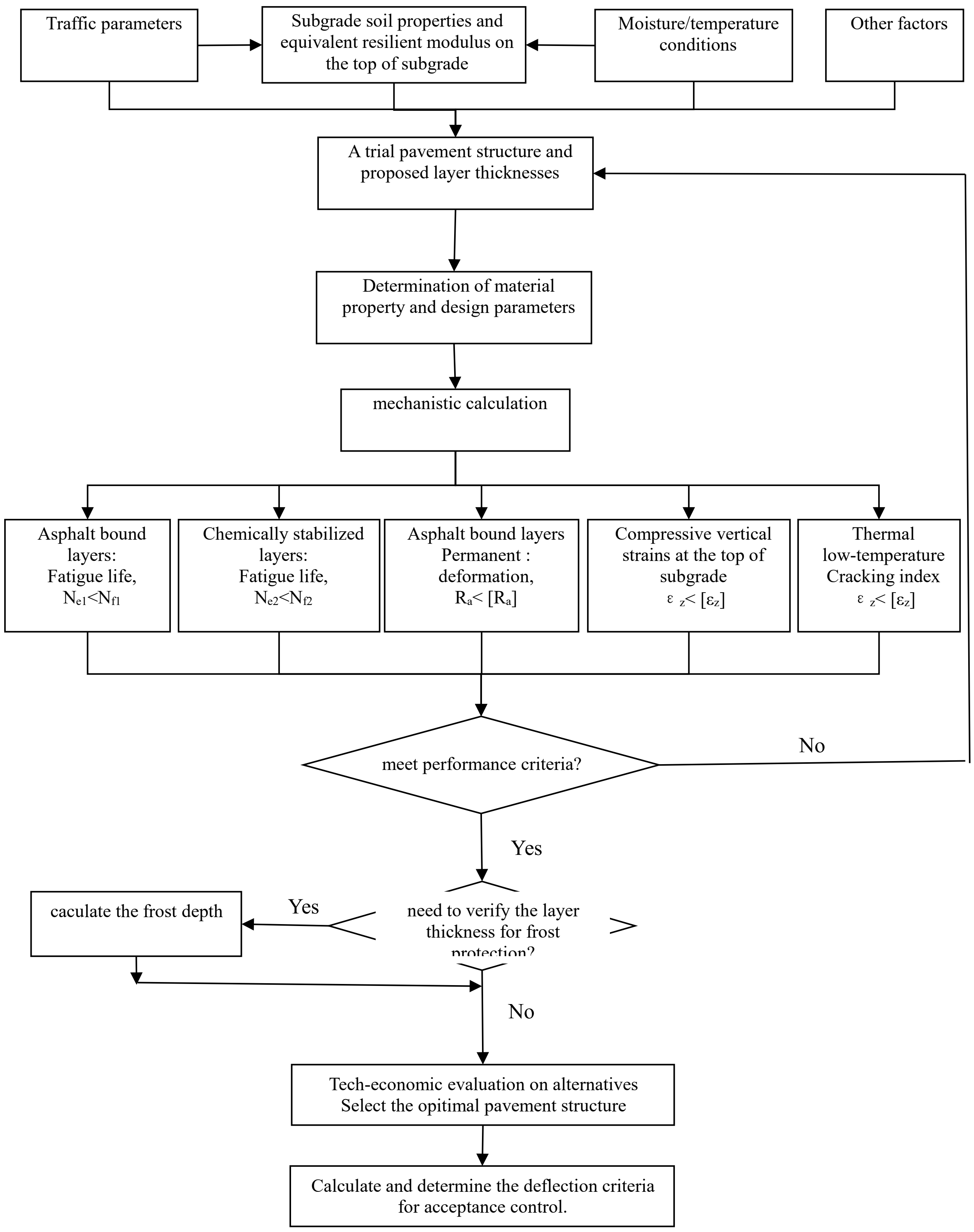
1 Dynamic modulus at 20℃ and 10Hz is adopted for the asphalt surface course; dynamic modulus at 20℃ and 5Hz for asphalt bound base course.

2 Elastic modulus corrected by adjustment factors is adopted for chemically stabilized layers.

3 Moisture adjusted resilient modulus is adopted for granular layers. The equivalent modulus in the top of subgrade in a moisture equilibrium state considering the effects of wet-dry and freeze-thaw cycles is adopted for subgrade.

6.3.3 When the calculation and verification is executed on the fatigue life of asphalt bound layers or on the vertical compressive strains on the top of the subgrade, the temperature shift factor shall be determined in accordance with Appendix G based on local temperature conditions, the pavement structure type and thickness of the structural layer. When the calculation and verification is executed for permanent deformation, the equivalent temperature shall be selected by following Appendix G based on the local temperature regime.

6.4.1 Calculation and verification on a pavement structure shall be performed according to the flow diagram shown in Figure 6.4.1.



Figue 6.4.1 Flowchart of Calculation and Verification of Pavement Structure

1 Investigate and analyze the traffic parameters in accordance with Appendix A of the Specifications, and then

determine the traffic load class in accordance with Clause 3.0.4 of the Specifications.

2 Identify the subgrade moisture condition and moisture level based on subgrade soil type and water table; determine the resilient modulus at the top of subgrade and calculate the necessary subgrade improvement measures in accordance with Clause 5.2.2 of these Specifications and by referring to relevant provisions in the current JTG D30 Design Specification of Highway Subgrade.

3 In accordance with the requirements for the design, collect the pavement structures popularly used and the properties of the materials available in the region; identify and analyze other factors that may influence the pavement structural design; suggest an initial pavement structure and thicknesses of pavement structural layers; and select design parameters accordingly.

4 Determine design parameters such as modulus of each structural layers; pursuant to Chapter 5 of the Specifications, test and obtain the CBR values of granular materials, the unconfined compressive strength of the chemically stabilized materials, and the low temperature thermal performance of bitumen, the low temperature thermal failure strains, the dynamic stability, penetration strength, moisture stability of the asphalt mixtures in accordance with Chapter 5 of the Specifications.

5 Further to the provisions in Appendix G of these Specifications, collect air temperature data of the region; determine the temperature shift factors or equivalent temperatures in accordance with the design parameters.

6 Calculate the pavement responses of various design parameters by using elastic multi-layer system software.

7 Further to Appendix B of these Specifications, carry out the calculation and verification on the pavement structure; the results shall comply with the provisions of Clause 3.0.6; in case of non-compliance, adjust the proposed pavement structure and re-perform the calculation and verification until compliance is achieved.

8 Carry out a tech-economic analysis on the verified pavement structures; identify and decide the optimum pavement structure.

9 Calculate the deflection criteria for acceptance of the pavement structure in accordance with Section B.7 in Appendix B of the Specifications.

6.4.2 Determination of the deflection on the top of subgrade and on the pavement surface for acceptance control shall comply with the provisions in Section B.7in Appendix B of these Specifications.

7 Design of Pavement Rehabilitation

7.1.1 This chapter is applicable to the design of structural strengthening of an asphalt pavement.

7.1.2 A design of pavement rehabilitation shall be developed based on thorough investigations and section-by-section assessments on the existing pavement to identify the real causes of the pavement damages and find out possible solutions to the problems. These possible or alternative solutions shall be further assessed by techno-economic evaluations to formulate the optimal solution thus the design concept for the pavement rehabilitation in accordance with predicted traffic loading class and by referring to the relevant and successful practical experience.

7.1.3 During the design for pavement rehabilitation, utilization of the existing pavement shall be fully taken into account. Material wastage shall be kept to a minimum by reutilizing the materials of the existing pavement in a proactive but prudent way.

7.1.4 A traffic guidance scheme for construction, including temporary safety devices, shall be planned and designed.

7.1.5 A contingency approach shall be adopted in pavement rehabilitation design. During implementation of pavement rehabilitation, the field investigation followed by assessment shall be carried out on each of the road sections, and then the proposed rehabilitation design approach shall be adjusted accordingly.

7.2.1 Investigations and analyses on the existing pavements shall be conducted as follows

1 Collect the information and data about design, construction and historic maintenance activities of the existing pavement and related drainage facilities;

2 Survey and analyze the traffic loading parameters including traffic volumes, axle comfiguration, traffic loads, traffic growth and so forth;

3 Investigate the damage to the pavement and identify the types, the severity, the scope and the quantity of the damaging;

4 Measure and assess the bearing capacity of the pavement structure by using FWD or other type of deflectometer;

5 Detect the thickness, interlayer bonding and damage accumulation of the existing pavement by means of coring, trial pits, ground penetrating radar (GPR) or trenching; perform the sampling and laboratory testing to determine the specimen modulus and strengths; analyze the material composition; and assess the deterioration of the pavement materials.

6 For sections where pavement damage is due to subgrade distress, take samples from the subgrade to determine geotechnical classification of the subgrade soil, moisture content, CBR, and so forth; and assess the subgrade stability and bearing capacity.

7 Investigate the climatic conditions, water table and drainage situation along the highway.

8 Investigate the requirements for bridge clearance and tunnel profile clearance, and other factors which may affect the design of the pavement rehabilitation.

7.2.2 Assessment on damage accumulation of the existing pavements shall be comply with the relevant provisions of prevailing *JTG H20 Assessment Criteria for Highway Technical Conditions, and GTJ H10 Technical Specifications for Highway Maintenance*. Additional assessments in terms of spacing of transverse cracks, rate of longitudinal cracks, the rate of crazing areas, the rate of patched areas or other indicators subject to the characteristics of pavement damaging.

7.2.3 In accordance with the investigations on the existing pavements, the designers shall analyze the distress causes in a comprehensive way to identify the layer position where the distress occurs, assess the damage accumulation and the damaging tendency and to determine the reutilization rate of the existing pavement.

7.3.1 The treatment method for the existing pavement shall be selected in accordance with the pavement conditions and damage accumulation of each pavement section.

7.3.2 Treatment of the existing pavement may be done in part, in full or in combination and shall comply with the requirements as follows:

1 The existing pavement sections with moderate damages and acceptable structural performance may be treated in part by the method of distress patching and then overlaying, in accordance with the prevailing JTJ 073.2 Technical Specifications of Bituminous Pavement Maintenance for Highways.

2 Where an existing pavement section has been seriously damaged or is in structural performance, the whole section should be treated. The depth and scope of the full treatment shall depend on the damage accumulation, the damaged layer position and the treatment process.

7.3.3 Pavement rehabilitation shall utilize the structure and materials of the existing pavement as much as possible. The treatment methods may be selected depending on particular situations, that is, applying one or several overlays directly on the top of the existing pavement after partial treatment of distress, overlaying after milling the existing pavement down to a structural layer or overlaying after milling and reclamation of the existing pavement.

7.3.4 Effective measures shall be taken to control and mitigate reflection cracking if such cracks exist extensively in the existing pavement.

7.3.5 In the case of water damage due to inadequate pavement subsurface drainage, the pavement subsurface drainage system shall be improved or replaced. Interlayer bonding measures such as a tack coat or seal coat shall be placed between the overlay and the existing pavement.

7.3.6 The composition and technical requirements for overlays shall comply with the criteria listed in chapter 5 of these Specifications. Technical requirements for recycled materials shall comply with the prevailing *JTG F41 Technical Specifications of Highway Asphalt Pavement Recycling*.

7.4.1 Traffic loading parameters predicted for the design life shall be investigated and analyzed in compliance with Appendix A of these Specifications, and traffic load classes be determined in compliance with clause 3.0.4 of these Specifications.

7.4.2 The performance of overlays and treated existing pavement structure for the structural design life shall comply with Clauses 3.0.6 and 3.0.7 of these Specifications.

7.4.3 In the case that the direct overlaying or the overlaying after milling the existing pavement down to a certain structural layer is adopted as the method of treatment for the pavement sections with limited damage and acceptable structural performance, structural verification shall be executed on both of the existing pavement layers and the overlays. Design parameters for overlays shall be determined in the same way as those for new construction pavement. Design parameters of structural layers of the existing pavement shall be determined in accordance with the following requirements:

1 The existing pavement and subgrade shall be simplified into a three-layer system comprising of asphalt bound layer, chemically stabilized or granular layer and subgrade. The modulus of each structural layer shall be determined by back-calculation of deflection basin data or by measuring sampled cores.

2 Flexural strength of chemically stabilized layer in the existing pavement should be determined by Equation (7.4.3) based on the unconstrained compressive strength obtained by testing the sample cores, or otherwise may be determined in accordance with the strength of whole pavement, the failure situation of base course and surface course in conjunction with the local experience and practice.

$$R_s = 0.21 R_c \tag{7.4.3}$$

Where R_s——flexural strength of specimen of chemically stabilized material (MPa);

R_c——unconstrained compressive strength of specimen of chemically stabilized material (MPa).

7.4.4 For the sections where the pavement has been seriously damaged and thus structural performance is inadequate, the structural verification shall be executed on either direct overlays or the overlays after milling. Design parameters of overlays shall be determined in the same way as that of a new pavement structure. For the existing pavement or re-utilized pavement after treatment, structural verification is not required but the equivalent resilient modulus on the surface is calculated by equation (7.4.4) below.

$$E_d = \frac{176pr}{l_0} \qquad (7.4.4)$$

Where: E_d——equivalent resilient modulus on the surface of the existing pavement structure;

p——load applied by falling weight deflectometer (FWD)

r——Radius of the loading plate of the falling weight deflectometer(FWD);

l_0——deflection value (0.01mm) at mid-point of loading plate of the falling weight deflectometer (FWD).

7.4.5 Design parameters of recycled materials shall be determined by site sampling and laboratory testing or by referring to the relevant experience in practice.

7.4.6 Structural verification of a rehabilitated pavement shall be carried out in the procedures as shown in Figure 7.4.6. The major activities are as follows:

1 Traffic parameters are investigated and analyzed in accordance with Appendix A, and traffic load class is determined as specified in Clause 3.0.4 of these Specifications.

2 Technical conditions of the existing pavement shall be investigated and analyzed in accordance with Section 7.2 of these Specifications.

3 Segmentation of the existing pavement is executed based on the results of pavement condition investigation。Rehabilitation methods are developed in accordance with section 7.3 of these Specifications and by referring to local practice.

4 The structural layers to be verified and relevant design parameters such as material modulus of the existing pavement and overlays are determined in accordance with Clauses 7.4.3, 7.4.4 and 7.4.5 of these Specifications. CBR values for verification of granular materials in overlays, unconstrained compressive strength for chemically stabilized materials, the requirements of low-temperature thermal performance, the strain at failure due to low-temperature, dynamic stability, penetration strength and water stability of asphalt mixtures are determined in accordance with Section 5 of these Specifications.

5 Temperature data is collected in the region along the project. Temperature factors or equivalent temperatures corresponding to various design parameters shall be determined in accordance with Appendix G of these Specifications.

6 The mechanistic responses of various design parameters are calculated by means of computerized software based on the theory of multi-layer elastic systems.

7 Structural verification of the pavement is executed in accordance with Appendix B, and the results shall be in compliance with Clause 3.0.6 of these Specifications; otherwise the design of the pavement rehabilitation should be revised and the verification should be re-executed until the results are satisfactory.

8 After structural verification, a techno-economic analysis shall be conducted to confirm the design.

9 Deflection values on the pavement surface for acceptance control are calculated in accordance with Appendix B, Section B.7 of these Specifications.

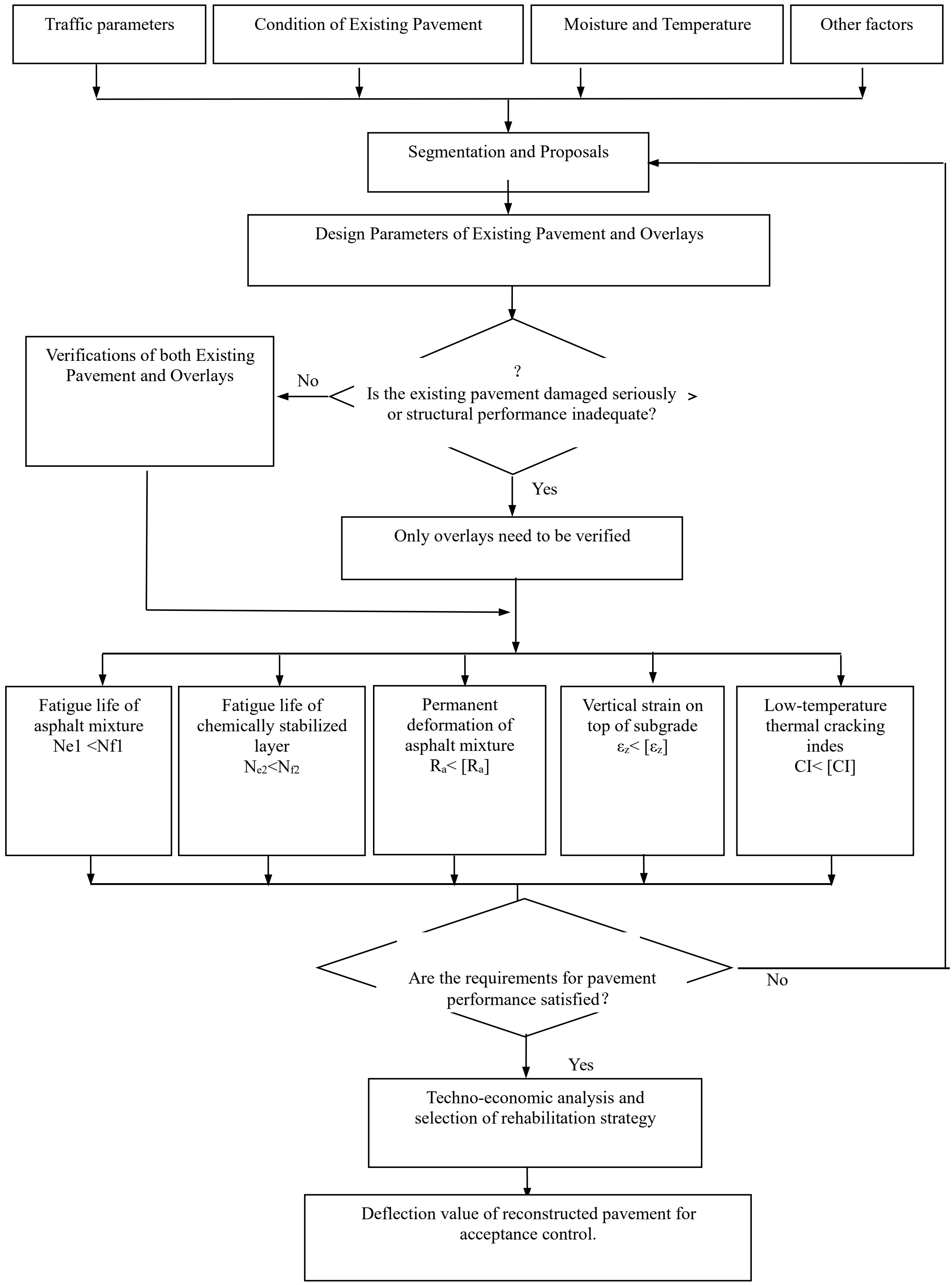


Figure 7.4.6 Flow chart of structural verification of pavement rehabilitation

8 Design of Bridge Deck Pavement

8.1.1 The design of a bridge deck pavement may include designs for bridge deck treatment, drainage, waterproofing, pavement structural layers and interface sealing at the edge and at expansion joints. The bridge type, highway classification, traffic load class, climatic environment and other influencing factors shall be taken into account during the design.

8.1.2 The pavement structure of a bridge deck should be in harmony with the pavement of main route of the highway. A specific pavement design should be conducted for any steel deck or Portland cement concrete deck of large or extra-large bridges.

8.1.3 Sufficient durability is required for the waterproofing system of a bridge deck.

8.2.1 A Portland cement concrete bridge deck should be pre-treated by milling or abrasive blasting. The textile of treated concrete deck surface should be 0.4 ~0.8mm thick.

8.2.2 The concrete leveling layer on a Portland cement concrete deck, if so required, should not be thinner than 80mm and should be reinforced by steel meshes. The concrete strength of a regulation layer shall be the same as that of the bridge beams and shall be well-bonded to the bridge deck.

8.2.3 Waterproofing materials for a Portland cement concrete deck should possess properties of sufficient bond strength, waterproofing capacity, capable of preventing damage during construction and acceptable durability. Hot bitumen and coating film are usually used as waterproofing material.

8.2.4 Rubberized asphalt or SBS modified asphalt are normally used as a hot bituminous waterproof membrane. The membrane thickness may be 1.5~2.0mm and should be covered with 13 mm single sized crushed stone with a coverage about 60%~70%.

8.2.5 For motorways or class-1 highways, the asphalt pavement on a Portland cement concrete bridge deck should not thinner than 70mm, for which a two layer or more than two layer structure may be selected provided that the thickness of the top layer should not be thinner than 30mm. For class 2 or lower class highways, the asphalt pavement on a Portland cement concrete bridge deck should not thinner than 50mm.

8.2.6 A layer of sand asphalt mixture should be placed on the deck of an extra-large bridge. Such a sand asphalt mixture shall possess enough high-temperature thermal stability, water thigh and capable of preventing construction damages, for which modified asphalt mastic sand, cast asphalt concrete are usually used.

8.2.7 An asphalt layer on a bridge deck shall possess low voids, good high-temperature thermal stability and high skid-resistance. Continuously graded asphalt mixture or asphalt mastic macadam should be selected.

8.2.8 The interfaces between curb borders, barriers or expansion joints and the asphalt deck pavements should be sealed and waterproofed by using hot asphalt, sealant strips or sealant fillers.

8.2.9 Longitudinal blind trenches may be placed along the edge strips of a bridge deck pavement, which should be 100~200mm wide and filled with open-graded asphalt bound materials or single sized crushed stones. Blind trenches shall connect to weep holes of the bridge.

8.3.1 Shot blasting shall be applied as a surface treatment of a steel bridge deck. The de-rusting level shall not be lower than Sa2.5. The surface after de-rusting shall be applied with an antirust coat or tack coat.

8.3.2 Waterproofing materials for a steel deck shall be compatible with the material of the deck pavement.

8.3.3 Cast in-situ asphalt concrete, epoxy asphalt concrete, continuously graded asphalt mixture, asphalt mastic crushed stones or the combination of multiple mixtures should be selected as the deck pavement of steel bridges.

Appendix A: Parameter Analysis of Traffic Loadings

A.1 Vehicle Classification

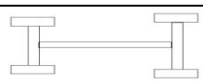
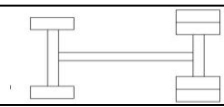
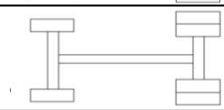
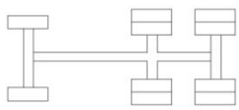
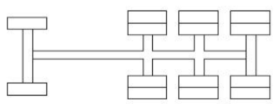
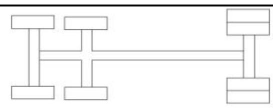
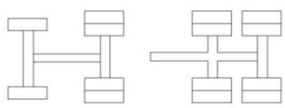
A.1.1 Vehicle axle configurations shall be categorized into 7 axle groups in terms of the number of tires and axles.

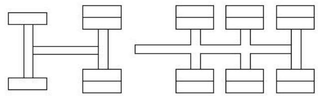
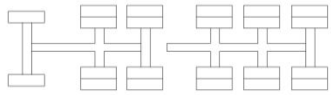
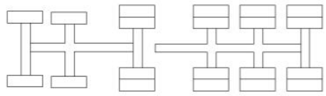
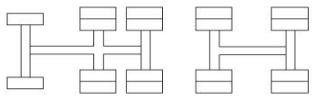
Table A.1.1 Axle Configuration

Axle Code	Axle group
1	Single axle (fitted with single tires)
2	Single Axle (fitted with dual tires)
3	Tandem axles (single tires)
4	Tandem Axles (one axle fitted with single tires, the other with dual tires
5	Tandem Axles (both fitted with dual tires)
6	Tridem axles (all fitted with single tires)
7	Tridem axles (all fitted with dual tires)

A.1.2 Vehicles are classified into 11 classes in terms of the axle configurations as shown in Table A.1.2.

Table A.1.2 Vehicle Classification

Vehicle Class	Vehicle Classification	Vehicle Types and Diagrams		Other vehicle types
Class-1	Vehicle with 2 axles and 4 tires	Type-11 vehicles		
Class-2	Bus with 2 axles, 6 or more tires	Type-12 buses		Type-15 trucks
Class-3	Single-unit truck with 2 axles and 6 tires or more	Type-12 trucks		
Class-4	Single-unit truck with 3 axles (non dual front axles)	Type-15		
Class-5	Single-unit truck with 4 axles, 14 tires	Type-17		
Class-6	Single-unit truck with dual front axles	Type-112 Type 115		Type-117
7 类 Class-7	4 轴及以下半挂货车 （非双前轴） Articulated trailer with 4 or less than 4 axles (non-dual front ales)	Type-125		Type-122

Class-8	Articulated trailer with 5 axles and 18 tires (non-dual front axles)	Type-127 Type-155		
Class-9	Articulated trailer with 6 or more axles (non-dual axles)	Type-157		
Class-10	Articulated trailer with dual front axles	Type-1127		Type-1122 Type-1125 Type-1155 Type-1157
Class-11	Truck trailer	Type-1522 Type-1222		

A.2 Traffic Data

A.2.1 Traffic data shall include traffic volume, traffic growth, directional distribution factors, lane distribution factors, vehicle type distribution, vehicle axle configurations and axle load distribution.

A.2.2 The initial traffic volume and other parameters may be derived from the project feasibility study reports or other prediction information about traffic volumes and by making reference to local traffic observations and statistics, otherwise observation stations shall be established on site.

A.2.3 Annual average traffic growth rates shall be determined by investigation and analysis in accordance with the highway classification and the regional economy and traffic development.

A.2.4 Directional distribution factors should be determined by site traffic counting data on traffic volumes in each direction or selected within a range of 0.5~0.6 if no site-counting data is available.

A.2.5 Lane distribution factors may determined in accordance with one of the three levels as follows. Level 1 shall be adopted for pavement rehabilitation design. Either Level 2 or Level 3 may be adopted for the design of new pavement.

1 Level 1: Based on traffic volume data from site observation, the designers calculate the number of vehicles travelling in each traffic lane, and then determine the lane distribution factors.

2 Level 2: Taking local empirical values.

3 Level 3: Taking recommended values listed in Table A.2.5.

Table A.2.5 Lane Distribution Factors

Number of Lanes on one direction	1	2	3	≥4
Freeway	/	0.70~0.85	0.45~0.60	0.40~0.50
Other Classified Highway	1.00	0.50~0.75	0.50~0.75	/

Note: Low values should be taken where the lane traffic is seriously affected by non-motorized vehicles. High values should be taken in converse situations.

A.2.6 Vehicle Distribution factors may be determined in accordance with one of the three levels, i.e. Level 1 shall be taken for pavement rehabilitation, while Level 2 or Level 3 may be taken for the design of new pavements.

1 Level 1: According to site observations, identify the percentages of Type 2~Type 11 vehicles, and then determine the vehicle type distribution factors.

2 Level 2: Based on historic traffic data or empirical values, determine Truck Traffic Classifications (TTC) group by using Table A.2.6-1, and then select the corresponding local empirical values.

3 Level 3: Based on historical traffic data or empirical values, determine Truck Traffic Classifications (TTC) by using Table A.2.6-1, and then take the Vehicle Type Distribution Factors from Table A.2.6-2.

Table A.2.6-1 Highway Truck Traffic Classifications (TTC)

TTC Group	Percentage of Single-unit Trucks	Percentage of Articulated trailer
TTC1	<40	>50
TTC2	<40	<50
TTC3	40-70	>20
TTC4	40-70	<20
TTC5	>70	-

Note: In the table above, single-unit truck are Type 3 ~ Type 6 vehicles, articulated trailers are Type 7 ~ Type 10 vehicles listed in Table A.1.2.

Table A.2.6-2 TTC Groups and Vehicle Type Distribution Factors (%)

Vehicle Type	Class-2	Class-3	Class-4	Class-5	Class-6	Class-7	Class-8	Class-9	Class-10	Class-11
TTC1	6.4	15.3	1.4	0.0	11.9	3.1	16.3	20.4	25.2	0.0
TTC2	22.0	23.3	2.7	0.0	8.3	7.5	17.1	8.5	10.6	0.0
TTC3	17.8	33.1	3.4	0.0	12.5	4.4	9.1	10.6	8.5	0.7
TTC4	28.9	43.9	5.5	0.0	9.4	2.0	4.6	3.4	2.3	0.1
TTC5	9.9	42.3	14.8	0.0	22.7	2.0	2.3	3.2	2.5	0.2

A.3 Vehicle Conversion to Equivalent Design Axle Loads

A.3.1 Equivalency factors for vehicle conversion may be determined in accordance with one of the three levels listed below. Level 1 shall be adopted for pavement rehabilitation of freeways and Class-1 highways. Level 2 or Level 3 shall be used for all other purposes.

1 Level 1—use weighing equipment to collect data of vehicle classes, axle configurations and axle loads continuously in the design lane, and then determine the equivalency factors for various vehicles by the following steps:

1) count the numbers of Class-2 ~ Class-11 vehicles respectively, calculate the number of axles of each class of the vehicles in terms of single axle-single tire, single axle-dual tire, tandem axle and tridem axle ; sum up the total axles of Class-2 to Class 11 vehicles, and calculate the average number of axles for each vehicle class by using equation (A.3.1-1).

$$NAPT_{mi} = \frac{NA_{mi}}{NT_m} \quad \text{(A.3.1-1)}$$

Where: $NAPT_{mi}$——average number of class-i axles within class-m vehicles.

NA_{mi}——Total number of axles of type-i vehicles within class-m vehicles.

NT_m——Total number of axles of all *m classes of vehicles*

i—— axle group in terms of single axle-single tire, single axle-dual tire, tandem axles and tridem axles.

m——class-2 ~ class-11 vehicles as listed in Table A.1.2.

2) Calculate the proportion of each axle type in different axle load ranges by using Equation (A.3.1-2) and derive axle load distribution factors, that is, the axle load spectrum. In order to develop the axle load spectrum, axle load ranges shall be grouped in 2.5kN, 4.5kN, 9.0kN and 13.5kN intervals for single axle-single tire, single axle-dual tire, tandem axles and tridem axles respectively.

$$ALDF_{mij} = \frac{ND_{mij}}{NA_{mi}} \quad \text{(A.3.1-2)}$$

Where: $ALDF_{mij}$——axle load distribution factor of type-i axles in the axle load range j within m classes of vehicles.

ND_{mij}——number of type-i axles in axle load range j within m classes of vehicles;

NA_{mi}——number of type-i axles within m classes vehicles.

Other symbols have the same meanings as those of Equation (A.3.1-1)

3）Take the median value of each axle load range as the representative axle load, calculate the equivalency factor for various axle types of class-2 ~ class-11 vehicles in each axle load range by using Equation (A.3.1-3). Then calculate equivalency factors for each class of vehicle by using Equation (A.3.1-4).

$$EALF_{mij} = c_1 c_2 \left(\frac{P_{mij}}{P_s} \right)^b \quad \text{(A.3.1-3)}$$

Where: P_s——Design axle loads (kN);

P_{mij}——single axle load (kN) of type-i axle of class-m vehicle; for tandem or tridem axles, refer to the axle mass which is equally allocated to each single axle;

b ——load equivalency exponent. b = 4 for analyzing fatigue or permanent deformation of asphalt bound materials； b = 5 for analyzing permanent deformation of the subgrade; b = 13 for analyzing fatigue of chemical stabilized layers.

c_1——axle configuration factor: take as separate single axles where the spacing between front axle and rear axle is greater than 3m; use the values in Table A.3.1-1 where the axle spacing is less than 3 m.

Table A．3.1-1 Axle Group Factors

Design Parameters	Axle type	c_1 value
Tensile strain at bottom of Asphalt bound layers, Permanent deformation of asphalt bound layers	Tandem	2.1
	Tridem	3.2
Vertical compressive strain on subgrade	Tandem	4.2
	Tridem	8.7
Tensile stress at bottom of chemical stabilized layers	Tandem	2.6
	Tridem	3.8

c_2——tire factor, 1.0 for dual tire, 4.5 for single tire.

$$EALF_m = \sum_i \left(NAPT_{mi} \sum_j (EALF_{mij} \times ALDF_{mij}) \right) \qquad \text{(A.3.1-4)}$$

Where $EALF_m$ —— load equivalency factor for class-m vehicles.

$NAPT_{mi}$ —— average number of type-i axles within class-m vehicles.

$ALDF_{mij}$ —— axle load distribution factor of type-i axles within class-m vehicles

$EALF_{mij}$ ——load equivalency factor for type-i axles within class-m vehicles in axle load range, j, which is determined by using Equation (A.3.1-3)

2 Level 2 and Level 3: determine load equivalency factors for various vehicle classes by using Equation (A.3.1-5). For the proportion of partially loaded/fully loaded vehicles and the load equivalency factors in Equation (A.3.1-5), local empirical values shall be taken where Level 2 is adopted; or the national empirical values, as listed in Table A.3.1-2 and Table A.3.1-3, shall be taken where Level 3 is adopted.

$$EALF_m = EALF_{ml} \times PER_{ml} + EALF_{mh} \times PER_{mh} \qquad \text{(A.3.1-5)}$$

Where $EALF_{ml}$ ——load equivalency factor for partially loaded class-m vehicles.

$EALF_{mh}$ ——load equivalency factor for fully loaded class-m vehicles

PER_{ml} ——percentage of partially loaded vehicles in total class-m vehicles

PER_{mh} ——percentage of fully loaded vehicles in total class-m vehicles

Table A.3.1-2 Percentages of partially loaded and fully loaded Class-2 ~Class-11vehicles

Vehicle Class	Proportion of partially loaded	Proportion of fully loaded
Class-2	0.80~0.90	0.10~0.20
Class-3	0.85~0.95	0.05~0.15
Class-4	0.60~0.70	0.30~0.40
Class-5	0.70~0.80	0.20~0.30
Class-6	0.50~0.60	0.40~0.50
Class-7	0.65~0.75	0.25~0.35
Class-8	0.40~0.50	0.50~0.60
Class-9	0.55~0.65	0.35~0.45
Class-10	0.50~0.60	0.40~0.50
Class-11	0.60~0.70	0.30~0.40

Table A.3.1-3 Load equivalency Factors for Class-2 ~ Class-11 Vehicles

Vehicle class	Tensile strain at bottom of Asphalt bound layers, Permanent deformation of asphalt bound layers		Tensile stress at bottom of chemical stabilized layers		Vertical compressive strain on the top of subgrade	
	Partially loaded	Fully loaded	Partially loaded	Fully loaded	Partially loaded	Fully loaded
Class-2	0.8	2.8	0.5	35.5	0.6	2.9
Class-3	0.4	4.1	1.3	314.2	0.4	5.6
Class-4	0.7	4.2	0.3	137.6	0.9	8.8
Class-5	0.6	6.3	0.6	72.9	0.7	12.4
Class-6	1.3	7.9	10.2	1505.7	1.6	17.1
Class-7	1.4	6.0	7.8	553.0	1.9	11.7
Class-8	1.4	6.7	16.4	713.5	1.8	12.5
Class-9	1.5	5.1	0.7	204.3	2.8	12.5
Class-10	2.4	7.0	37.8	426.8	3.7	13.3
Class-11	1.5	12.1	2.5	985.4	1.6	20.8

A.4 Cumulative Equivalent Design Axle Loads

A.4.1 After the load equivalency factors are determined in accordance with section A.3 of these Specifications, determine the daily average equivalent axle loads per lane in the first year of the design life, N_1.

$$N_1 = AADTT \times DDF \times LDF \times \sum_{m=2}^{11} (VCDF_m \times EALF_m) \qquad \text{(A.4.1)}$$

Where $AADTT$ ——Annual average daily truck traffic of the vehicles with two axles and 6 or more tires in both directions.

DDF ——Directional distribution factor

LDF ——Lane Distribution Factor

m——Vehicle class code

$VCDF_m$ ——Vehicle Class-m Distribution Factor

$EALF_m$ —— Equivalent axle load factor for class-m vehicles

A.4.2 Using the daily average equivalent axle loads in the first year of the design life N1, the cumulative equivalent design axle loads, N_e, over the design life shall be calculated by using Equation (A.4.2).

$$Ne = \frac{\left[(1+\gamma)^t - 1\right] \times 365}{\gamma} N_1 \qquad \text{(A.4.2)}$$

Where N_e——Cumulative number of equivalent design axle loads over the design life.

t ——Design life in years

γ ——Annual average traffic growth during the design life

N_1 ——Daily average equivalent loads in the design lane during the first year after opening to traffic.

APPENDIX B METHOD OF CALCULATION AND VERIFICATION OF PAVEMENT STRUCTURES

B.1.1 The fatigue life of an asphalt bound layer shall be calculated based on the strains at the bottom of an asphalt bound layer, which is obtained from pavement structural analysis by using Equation (B.1.1-1) below.

$$N_{f1} = 6.32 \times 10^{(15.96-0.29\beta)} k_a k_b k_{T1}^{-1} \left(\frac{1}{\varepsilon_a}\right)^{3.97} \left(\frac{1}{E_a}\right)^{1.58} (VFA)^{2.72} \quad \text{(B.1.1-1)}$$

Where N_{f1}——fatigue life (number of repetitions to fatigue cracking) of asphalt bound layers.

β——Target reliability index, taken from Table 3.0.1 in accordance with the highway classification.

k_a——adjustment factor for seasonal frost region, determined by using an interpolated value from Table B.1.1.

k_b——factor of fatigue loading mode, calculated by using Equation (B.1.1-2)

$$k_b = \left(\frac{1 + 0.3E_a^{0.43}(VFA)^{-0.85} e^{(0.024h_a-5.41)}}{1 + e^{(0.024h_a-5.41)}}\right)^{3.33} \quad \text{(B.1.1-2)}$$

Where E_a——dynamic compressive modulus at 20 °C of asphalt mixture (MPa);

VFA——voids filled with asphalt (%), determined in accordance with relevant provisions in the current *JTG F40 Specifications for Construction of Highway Asphalt Pavement.*

h_a——thickness of asphalt bound layer (mm);

k_{T1}——temperature shift factor;

ε_a——microstrain (strain x10^{-6}) at bottom of asphalt bound layer, calculated by Equation (B.1.1-3) at the calculation points selected in accordance with the multi-layered elastic theory and the provisions of Sub-Clause 6.2.2 of these Specifications.

$$\varepsilon_a = p\overline{\varepsilon_a} \quad \text{(B.1.1-3)}$$

$$\overline{\varepsilon_a} = f\left(\frac{h_1}{\delta}, \frac{h_2}{\delta}, \ldots, \frac{h_{n-1}}{\delta}; \frac{E_2}{E_1}, \frac{E_3}{E_2}, \ldots, \frac{E_0}{E_{n-1}}\right)$$

Where: $\overline{\varepsilon_a}$ ——coefficient of theoretical tensile strain

p,δ——contact pressure of tire and equivalent radii under standard axle loads.

E_0 ——resilient modulus st the top of subgrade (MPa);

$h_1, h_2, \ldots, h_{n-1}$ ——thickness of each structural course (mm);

$E_1, E_2, \ldots, E_{n-1}$ ——Modulus of each structural layer (MPa).

Table B.1.1 Adjustment Factor of Seasonal Frost Areas

Frost Area	Heavy frost	Medium frost	Moderate frost	Others

Frost Index F (℃ • day)	≥2000	2000~800	800~50	≤50
k_a	0.60~0.70	0.70~0.80	0.80~1.00	1.00

B.1.2 The fatigue life (i.e. number of repetitions to fatigue cracking) of an asphalt bound layer shall be larger than the cumulative ESAL on the design lane during the design life. Otherwise, the proposed pavement structure shall be adjusted and re-calculated and verified until compliance is reached.

B.2.1 The fatigue life of a chemically stabilized layer shall be calculated by using Equation (B.2.1-1), based on the tensile stress of each and all chemically stabilized layers which are obtained from the pavement structural analysis.

$$N_{f2} = k_a k_{T2}^{-1} 10^{\left(a-b\left(\frac{\sigma_t}{R_s}\right)+k_c-0.57\beta\right)} \qquad \text{(B.2.1-1)}$$

Where N_{f2}——number of repetitions to fatigue cracking in chemically stabilized layer;

k_a——adjustment factor of seasonal frost area, determined by using Table B.1.1;

k_{T2}——temperature shift factor, determined as described in Appendix G of these Specifications;

R_s——Tensile flexural strength (MPa) of chemically stabilized material;

a, b——regression parameters of fatigue tests, determined by using Table B.2.1-1.

k_c——an overall field correction factor, determined by using Equation (B.2.1-2)

$$k_c = c_1 e^{c_2(h_a+h_b)} + c_3 \qquad \text{(B.2.1-2)}$$

Where c_1、c_2、c_3——parameters, taking the values from Table B.2.1-2.

h_a 、 h_b——thickness of asphalt bound layer, and thickness of chemically stabilized layers above the computational point.

β——target reliability index, values taken from Table 3.0.1 in accordance with highway classification.

σ_t——Tensile stress at the bottom of chemically stabilized layers (MPa), in accordance with the multi-layered elastic system theory, at the calculation points selected by referring to Sub-Clause 6.2.2 and by using Equation (B.2.1-3).

$$\sigma_t = p\overline{\sigma_t} \qquad \text{(B.2.1-3)}$$

$$\overline{\sigma_t} = f\left(\frac{h_1}{\delta}, \frac{h_2}{\delta}, \ldots, \frac{h_{n-1}}{\delta}; \frac{E_2}{E_1}, \frac{E_3}{E_2}, \ldots, \frac{E_0}{E_{n-1}}\right)$$

Where $\overline{\sigma_t}$—— Coefficient of calculated tensile stress

Definitions of other symbols are the same as those under Equation (B.1.1-3).

Table B.2.1-1 Parameters for fatigue failure model of chemically stabilized layers

Materials	a	B
Chemically stabilized granular materials	13.24	12.52
Chemically stabilized soil	12.18	12.79

Table B.2.1-2 Relevant Parameters for Overall Field Correction Factor, k_c

	New pavement or reconstruction of existing pavement		Overlay for rehabilitation	
Materials	Chemically stabilized granular materials	Chemically stabilized soil	Chemically stabilized granular materials	Chemically stabilized soil
c_1	14.0	35.0	18.5	21.0
c_2	-0.0076	-0.0156	-0.01	-0.0125
c_3	-1.47	-0.83	-1.32	-0.82

B.2.2 For chemically stabilized layers, fatigue life shall be longer than the cumulative equivalent design axle loads during the design life on the design lane. Otherwise adjustment shall be made either to the pavement structure or to the thickness of the structural layers, followed by re-calculation and reverifiction until the requirements are satisfied.

B.3.1 Each layer of asphalt bound materials shall be further divided into sub-layers, and permanent deformation in each sub-layer shall be calculated as specified below.

1 Surface layer, taking each 10~20mm in thickness as one sub-layer.

2 For second layer of asphalt bound materials, the thickness of each sub-layer shall not be greater than 25mm.

3 For third layer of asphalt bound material, the thickness of each sub-layer shall not be greater than 100mm.

4 Fourth and lower layers of asphalt bound materials shall be taken as one sub-layer.

B.3.2 The permanent deformation obtained from rutting tests of each asphalt bound layer is obtained by referring to the rutting tests under standard conditions, and then the permanent deformation in each sub-layer and total permanent deformation of the asphalt bound materials are calculated by using Equation (B.3.2-1).

$$R_a = \sum_{i=1}^{n} R_{ai} \qquad \text{(B.3.2-1)}$$

$$R_{ai} = 2.31 \times 10^{-8} k_{Ri} T_{pef}^{2.93} \quad p_i^{1.80} N_{e3}^{0.48} (h_i / h_0) R_{0i}$$

Where R_a——total permanent deformation in the asphalt bound layers.

R_{ai}——permanent deformation in the i-th sub-layer(mm);

n——the number of sub-layers;

T_{pef}——equivalent temperature (℃) for permanent deformation in asphalt bound layers, to be determined as described in Appendix G of these Specifications.

N_{e3}——cumulative number of equivalent design axle loads in the design lane during the design life or during the period of operation until the first pavement rehabilitation to repair rutting

h_i ——thickness of the i-th layer;

h_0——thickness of the specimen used in the rutting test

R_{0i}——permanent deformation (mm) in the i-th layer of asphalt bound material at a test temperature of 60℃ and a test pressure of 0.7MPa for 2520 load repetitions, determined in the rutting test.

k_{Ri}——overall correction factor, calculated by using Equations (B.3.2-2) to (B.3.2-4)

$$k_{Ri} = \left(d_1 + d_2 \cdot z_i\right) \cdot 0.9731^{z_i} \qquad \text{(B.3.2-2)}$$

$$d_1 = -1.35 \times 10^{-4} h_a^2 + 8.18 \times 10^{-2} h_a - 14.50 \qquad \text{(B.3.2-3)}$$

$$d_2 = 8.78 \times 10^{-7} h_a^2 - 1.50 \times 10^{-3} h_a + 0.90 \qquad \text{(B.3.2-4)}$$

Where z_i——thickness of the i-th sub-layer of asphalt bound layers, taking 15mm for 1st sub-layer; for other sub-layers, the depth from the pavement surface to the mid-point of the sub-layer is used.

h_a——the thickness of asphalt bound layer (mm); wherever h_a is greater than 200mm, the value of 200mm is used.

p_i——vertical compressive stress (MPa) at the top of i-th sub-layer of asphalt bound layers, calculated by using Equation (B.3.2-5) for the calculation points which are selected in conformance with Sub-Clause 6.2.2, according to multi-layer elastic theory.

$$p_i = p\overline{p_i} \qquad \text{(B.3.2-5)}$$

$$\overline{p_i} = f\left(\frac{\mathbf{h_1}}{\delta}, \frac{\mathbf{h_2}}{\delta}, \ldots, \frac{\mathbf{h}_{n-1}}{\delta}; \frac{E_2}{E_1}, \frac{E_3}{E_2}, \ldots, \frac{E_0}{E_{n-1}}\right)$$

where $\overline{p_i}$—— theoretical coefficient of compressive stress.

Other symbols are defined as for those of Equation (B.1.1-3).

B.3.3 The permanent deformation in asphalt bound layers derived by calculation and verification shall comply with the requirements of allowable permanent deformation as specified in Table 3.0.6-1. Otherwise, the asphalt mix design should be revised until the requirements are satisfied.

B.3.4 Subject to the fulfillment of the requirements for allowable permanent deformation, asphalt bound materials shall also comply with the requirements as specified in Sub-Clause 5.5.7 for dynamic stability determined by standard rutting tests. The dynamic stability corresponding to permanent deformation R_0 may be used as one of the indicators for quality and workmanship control. Standard rutting tests shall be conducted at a testing temperature of 60ºC, testing pressure of 0.7MPa, test specimens are 50mm thick and the number of repetitive loads is 2520, then the dynamic stability (DS) of the asphalt bound material may be calculated by using Equation (B.3.4).

$$DS = 9365R_0^{-1.48} \tag{B.3.4}$$

Where DS—dynamic stability of asphalt bound material (number of repetition/mm)

B.4.1 The allowable vertical compressive strains at the top of subgrade shall be determined by the calculations using Equation (B.4.1)

$$[\varepsilon_Z] = 1.25 \times 10^{4-0.1\beta}(k_{T3}N_{e4})^{-0.21} \tag{B.4.1}$$

Where $[\varepsilon_z]$——allowable vertical compressive strains (10^{-6}) at the top of subgrade;

β——target reliability indicator, read from Table 3.0.1 in accordance with the highway classification.

N_{e4}——cumulative number of design axle loads, calculated in accordance with Appendix A of these Specifications.

k_{T3}——temperature shift factor, determined as described in Appendix G of the Specifications.

B.4.2 The computational points shall be selected in conformance with the provisions of Sub-Clause 6.2.2. Vertical compressive strains at the top of subgrade shall be calculated in accordance with multi-layer elastic theory, by using Equation (B.4.2). The vertical compressive strains at the top of subgrade shall be less than the allowable value of compressive strains. Otherwise, the proposed pavement structure shall be revised and further calculation and verification be conducted until the requirements are satisfied.

$$\varepsilon_z = p\bar{\varepsilon_z} \tag{B.4.2}$$

$$\bar{\varepsilon_z} = f\left(\frac{\boldsymbol{h_1}}{\delta}, \frac{\boldsymbol{h_2}}{\delta} \cdots\cdots \frac{\boldsymbol{h}_{n-1}}{\delta}; \frac{E_2}{E_1}, \frac{E_3}{E_2} \cdots\cdots \frac{E_0}{E_{n-1}}\right)$$

Where $\bar{\varepsilon_z}$——coefficient of calculated vertical compressive strain;

Other symbols have the same meaning as defined for Equation (B.1.1-3)

B.5.1 For the asphalt surface course in a seasonal frost area, low temperature thermal cracking index CI shall be calculated by using Equation (B.5.1).

$$CI = 1.95 \times 10^{-3} S_t \lg b - 0.075(T + 0.07h_a)\lg S_t + 0.15 \tag{B.5.1}$$

Where CI—Thermal cracking index of asphalt surface course;

T—Design temperature of the pavement surface at low temperature, taking the average of the annual lowest air temperatures in 10 successive years.

S_t——asphalt creep stiffness (MPa) of the pavement surface at the test temperature, which is the design temperature of pavement surface at low temperature plus 10℃, under loading for 180 seconds in the test for determining the flexural creep stiffness of asphalt binder using the Bending Beam Rheometer (BBR).

h_a——thickness of asphalt bound layers.

b——coefficient of subgrade type, b=5 for sand, b=3 for silt clay, and b=2 for clay.

B.5.2 The low temperature thermal cracking index of the asphalt surfacing shall comply with the requirements in Table 3.0.6-2. Otherwise, the selected asphalt binder material shall be replaced until the bitumen binder material satisfies the requirements.

B.6.1 Where the subgrade is in a moderate moisture or wet level in a seasonal frost region, the multi-year maximum frost depth of the highway shall be determined by using Equation (B.6.1.

$$Z_{max} = abcZ_d \tag{B.6.1}$$

Where $Z_{\max}$——multi-year maximum frost depth of the highway;

Z_{d}——multi-year frost depth of the ground, determined by referring to historic records;

a——relative thermo coefficient of each layer of the pavement and subgrade materials within the range of ground frost penetration, determined by using Table B.6.1-1;

b—subgrade moisture content, determined by using Table B.6.1-2;

c——coefficient of subgrade cross-sectional type, determined by interpolation from Table B.6.1-2.

Table B.6.1-1 Relative Thermo l Properties of Pavement and Subgrade Materials

Subgrade materials	Clay	Silt	Silty sand	Silt sand, clayey sand	Gravel with fine sand
Relative thermo coefficient	1.05	1.10	1.20	1.30	1.35
Pavement materials	Cement concrete	Asphalt bound materials	Graded macadam	Lime-flyash or cement stabilized granular materials	Lime-flyash or cement stabilized soils
Relative thermo coefficient	1.40	1.35	1.45	1.40	1.35

Table B.6.1-2 Subgrade Moisture Coefficient b

Moisture Level	Dry	Moderate Moisture	Wet
Moisture Coefficient	1.0	0.95	0.90

Table B.6.1-3 Coefficient of Subgrade Cross-sectional Type, c

Height of fill or Depth of cutting	Height of Embankment Fill					Depth of Cutting			
	Zero fill	<2m	2~4m	4~6m	> 6m	<2m	2~4m	4~6m	> 6m
Coefficient of Cross-sectional Type	1.0	1.02	1.05	1.08	1.10	0.98	0.95	0.92	0.90

B.6.2 Based on the multi-year maximum frost penetration of the highway, the highway frost protection depth shall be calculated and verified as specified in Table B.6.2. If the pavement structural thickness is less than the minimum frost protection depth as specified in Table B.6.2, an additional frost protection layer shall be placed to meet the requirements for frost protection depth.

Table B.6.2 Minimum Thickness of Asphalt Pavement Structure(mm)

Subgrade	Base and Subbase	Maximum Frost Depth Z_{max}（mm）and Minimum Frost Protection Thickness							
		Moderate moisture				Wet			
		500 ~1000	1000 ~1500	1500 ~2000	＞ 2000	500 ~1000	1000 ~1500	1500 ~2000	＞ 2000
Clay, fine sandy soil	Granular materials	400~450	450~500	500~600	600~700	450~550	550~600	600~700	700~800
	Cement or lime stabilized materials, cement concrete	350~400	400~450	450~550	550~650	400~500	500~550	550~650	650~750
	Cement-flyash or lime-flyash stabilized materials, Asphalt bound materials	300~350	350~400	400~500	500~550	350~450	450~500	500~550	550~700
Silty soil	Granular materials	450~500	500~600	600~700	700~750	500~600	600~700	700~800	800~1000
	Cement or lime stabilized materials, Cement concrete	400~450	450~500	500~600	600~700	450~550	550~650	650~700	700~900
	Cement-flyash or lime-flyash stabilized materials, Asphalt bound materials	300~400	400~450	450~500	500~650	400~500	500~600	600~650	650~800

Note: 1 The frost protection thickness may be reduced by 15%~20% in the areas where moisture level is less than 0.5 and in the dry areas of Zone II, III or IV, referring to the current *JTJ003 Highway Natural Zoning Standards;*

【En Note】Moisture level (K) is the ratio of annual precipitation R(mm) to annual evacuation Z(mm)， K = R/Z.

2 Frost protection depth shall be reduced by 5%~10% for a sandy soil subgrade in Zone II.

3 The upper values should be taken where the highway multi-year maximum frost penetration is large, the lower values taken in converse situations.

4 In the cases where different types of materials are used in the base and subbase, the frost protection thickness should be determined based on the material that is comparatively thicker.

B.7.1 The deflection value for acceptance inspection at the top of subgrade, l_g, shall be calculated by using Equation (B.7.1)

$$l_g = \frac{16500}{E_0} \qquad l_g = \frac{176pr}{E_0} \tag{B.7.1}$$

Where l_g——deflection value for acceptance at the top of subgrade (0.01mm)；

p——pressure applied by falling weight deflectometer (FWD) (MPa);

r——Radius of the loading plate of the falling weight deflectometer(FWD) (mm);

E_0——resilient modulus at the top of subgrade at moisture equilibrium (MPa)$E_d = \frac{176pr}{l_0}$$E_d = \frac{176pr}{l_0}$ $E_d = \frac{176pr}{l_0}$

B.7.2 The falling weight deflectometer should be used for subgrade acceptance inspection. The falling weight should be 50kN; the radius of the loading plate should be 150mm. The representative deflection value measured at the top of subgrade shall comply with the requirements of Equation (B.7.2-1).

$$l_0 \leq l_g \qquad \text{(B.7.2-1)}$$

Where l_0——representative value (0.01mm) measured at the top of subgrade in the road segment under assessment, which is about 1～3km long, calculated by using Equation (B.7.2-2)

$$l_0 = (\bar{l}_0 + \beta \cdot s)\ K_1 \qquad \text{(B.7.2-2)}$$

where $\bar{l}_0$——average of the deflection values measured on top of the subgrade in the road segment under consideration (0.01mm);

s——standard deviation of the deflection values measured on top of the subgrade within the road segment under consideration;

β——target reliability index, given in Table 3.0.1 in terms of the highway classification;

K_1——moisture adjustment factor for deflection at the top of subgrade, determined by local experience.

B.7.3 The deflection value at the top of the road surface for acceptance l_a shall be calculated by using Equation (B.7.3) in the light of designed pavement structure and in accordance with the multi-layered elastic system theory. The parameters of the pavement structural layers shall be the same as the ones used for calculation and verification of the pavement structure. The resilient modulus at the top of subgrade to be used shall be the resilient modulus at the top of subgrade in moisture equilibrium multiplied by a modulus adjustment factor K_l.

$$l_a = p\overline{l_a} \qquad \text{(B.7.3)}$$

$$\overline{l_a} = f\left(\frac{\boldsymbol{h_1}}{\delta}, \frac{\boldsymbol{h_2}}{\delta}, \dots, \frac{\boldsymbol{h}_{n-1}}{\delta}; \frac{E_2}{E_1}, \frac{E_3}{E_2}, \dots, \frac{k_l E_0}{E_{n-1}}\right)$$

where l_a——Deflection value at the top of road surface for acceptance control (0.01mm).

$\overline{l_a}$—— calculated deflection factor;

k_l——adjustment factor of resilient modulus at the top of subgrade: for an asphalt pavement with chemically stabilized base course and an asphalt pavement with cement concrete base course, use 0.5; for pavements with granular base course or asphalt bound basecourse, use 0.5 if the subbase is chemically stabilized materials, otherwise use 1.0.

E_0——resilient modulus at the top of subgrade in moisture equilibrium status (MPa).

Other symbols have the same meaning as defined for Equation (B.1.1-3).

B.7.4 Inspection and tests of deflection at the top of road surface shall be conducted for acceptance control or on construction completion of the pavement. The representative deflection values at the central sensor of a falling weight deflectometer (FWD) shall comply with the requirements of Equation (B.7.4-1).

$$l_0 \le l_a \qquad \text{(B.7.4-1)}$$

Where: l_0 ——representative deflection value at the top of road pavement (0.01mm) within the road segment under consideration, which is about 1 to 3km long , calculated by using Equation (B.7.4-2).

$$l_0 = (\overline{l}_0 + \beta \cdot s)\ K_1 K_3 \qquad \text{(B.7.4-2)}$$

Where: $\overline{l}_0$——average of the deflection values measured on the road surface within the road segment under consideration;

s —— standard deviation of deflections measured on the road surface within the road segment under consideration (0.01mm);

β—— target reliability index, taken from Table 3.0.1 according to the highway classification;

K_1——factor of moisture influence on the deflection at the top of road surface, which is determined as follows: firstly derive the subgrade modulus by back calculation using measured defletion values, followed by revisions of the subgrade modulus to calculate the value of structural modulus, and then obtain the factor K_1. *Alternatively K_1 may be obtained by experience of local practice.*

K_3——factor of temperature influence on deflection at the top of road surface, determined by using Equation (B.7.4.3)

$$K_3 = e^{\left[9\times10^{-6}(\ln E_0 - 1)h_a + 4\times10^{-3}\right](20-T)} \qquad \text{(B.7.4-3)}$$

where T—measured temperature at mid-point of asphalt bound material when deflection is measured (°C) or predicted from air temperature;

h_a——thickness of asphalt bound material (mm);

E_0——resilient modulus at the top of subgrade in moisture equilibrium (MPa).

APPENDIX C ASPHALT PAVEMENT STRUCTURES

C.0.1 In accordance with the traffic loads classification, the thickness of each structural layer of an asphalt pavement may be either selected by referring to Tables C.0.1-1 ~ C.0.1-6; or determined based on experience with local engineering practice.

TableC.0.1-1 Layer thicknesses of a pavement with chemically stabilized base and unbound granular subbase (in mm)

Traffic loads classification	Very heavy or Extra heavy	Heavy	Medium	Light
Surface course	250～150	250～150	200～100	150～20
Base course (chemically stabilized materials)	600～350	550～300	500～250	450～150
底基层（粒料类）Subbase course (Unbound granular materials)	200～150			

Table C.0.1-2 Layer thicknesses of a pavement with chemically stabilized base and chemically stabilized subbase (in mm)

Traffic loads classification	Very heavy or Extra heavy	Heavy	Medium	Light
Surface course	250～120	250～100	200～100	150～20
Base course (chemically stabilized materials)	500～250	450～200	400～150	500～200
Subbase course (chemically stabilized materials)	200～150			-

Table C.0.1-3 Layer thicknesses of a pavement with unbound granular base and unbound granular subbase (in mm)

Traffic loads classification	Heavy	Medium	Light
Surface course	350～200	300～150	200～100
Base course (unbound granular)	450~350	400~300	350~250
Subbase course (unbound granular)	200～150		

Table C.0.1-4 Layer thicknesses of a pavement with asphalt bound base and unbound granular subbase (in mm)

Traffic loads classification	Heavy	Medium	Light
Surface course	150～120	120～100	80～40
Base course (asphalt bound materials)	250～200	220～180	200～120
Subbase course (unbound granular materials)	400～300	400～300	350～250

Table C.0.1-5 Layer thicknesses of a pavement with asphalt bound base and chemically stabilized subbase (in mm)

Traffic loads classifications	Very heavy or Extra heavy,	Heavy	Medium	Light
Surface course	120～100	120～100	100～80	80～40
Base course (asphalt bound materials)	180～120	150～100	150～100	100～80
Subbase (chemically stabilized materials)	600～300	600～300	550～250	450～200

Table C.0.1-6 Layer thicknesses of a pavement with asphalt bound base and unbound granular or chemically stabilized subbase (in mm)

Traffic loads classifications	Very heavy or Extra heavy	Heavy	Medium	Light
Surface course	120～100	120～100	100～80	80～40
Base course (asphalt bound materials)	240～160	180～120	160～100	100～80
Subbase (unbound granular materials)	200～150	200～150	200～150	200～150
Subbase (unbound granular materials)	400～200	400～200	350～200	250～150

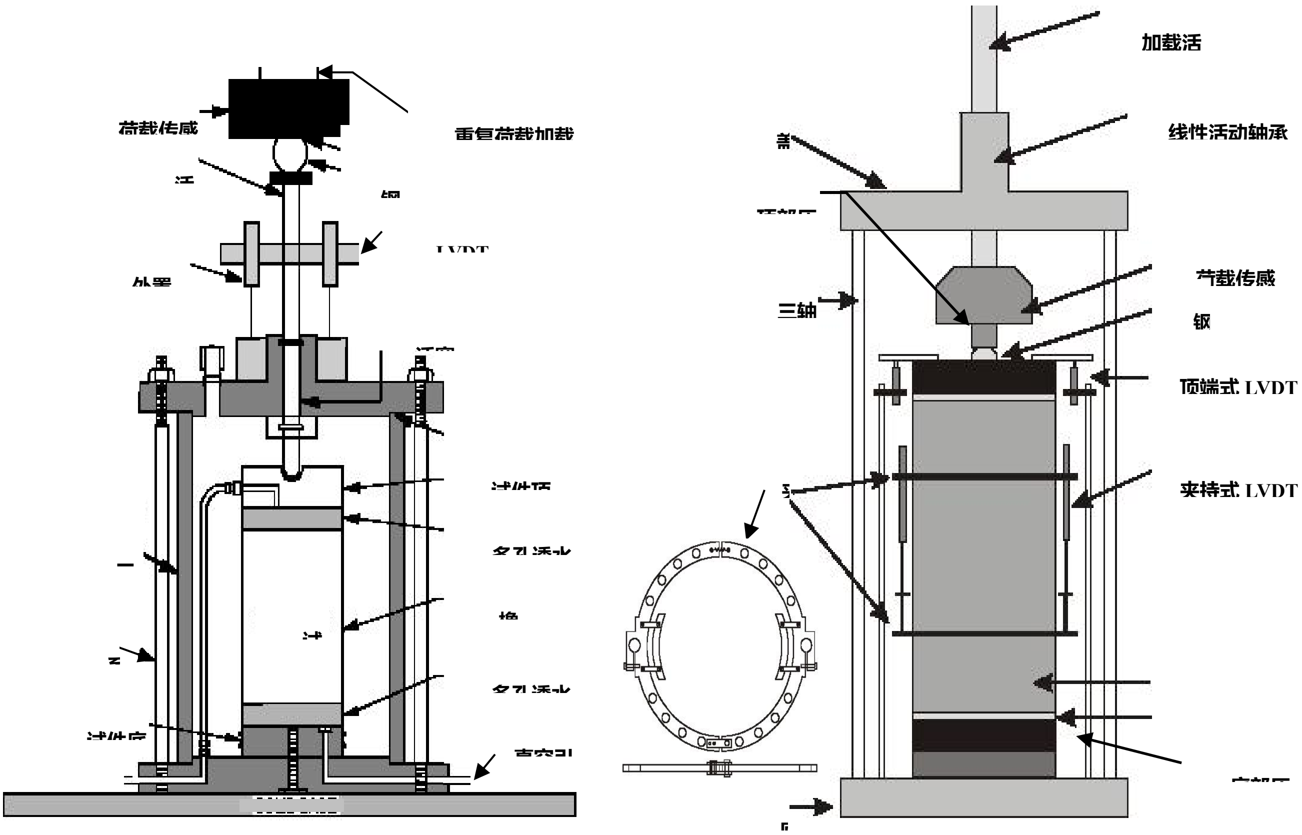
C.0.2 The thickness of a structural layer shall be related to the traffic load classification, the bearing capacity of the subgrade and other relevant factors. Where the traffic load classification is comparatively high or the bearing capacity of subgrade is rather low, an upper value of the thickness range, or the value in the range for one traffic load classification class higher shall be selected. Otherwise, the lower value in the thickness range or the value in the range for one load classification class lower may be used.

APPENDIX D STANDARD METHOD OF TEST FOR MEASURING THE RESILIENT MODULUS OF UNBOUND GRANULAR MATERIAL

D.1.1 The method is applicable to the repeated load triaxial compression test for determining the resilient modulus of unbound granular material.

D.1.2 Reconstituted specimens may be compacted by drop rammer, static loading, vibrating hammer or other methods.

D.2.1 There are two types of repeated load triaxial machines, namely internal type and external type, in terms of the position of deflection measuring instruments. Note that the internal type is preferred as end effects are eliminated.



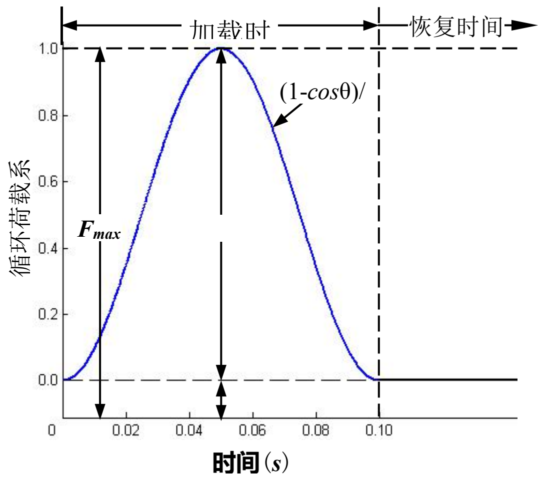
活塞 Piston	Ball seat Steel ball		盖板 Cover plate 顶部压盘 Top plate	线性活动轴承 LINEAR BEARING
外置 LVDT External LVDT	LVDT 托架 LVDT solid bracket		三轴室 Tri-axial chamber	荷载传感器 LOAD ACTUATOR 钢球 STEEL BALL
三轴室 Tri-axial chamber	Piston sleeve O rings		夹具 Attachment frames	顶端式 LVDT Top LVDT
系杆 Tie rods	Sample cap		底座 Solid Base	夹持式 LVDT Attached LVDT
试件底座 Specimen base	porous stone Specimen membrane porous stone			试件 SPECIMEN
	Vacuum inlet			多孔透水石

				POROUS STONE
试件 Specimen				Bottom plate
a) 外置式 a) External Type			b) 内置式 b) Internal Type	
c)				

图 D.2.1 动三轴试验仪
Figure D 2.1 Repeated load triaxial machine

D.2.2 The tri-axial chamber shall be made of polycarbonate, acrylic or other “see through” material, in which air shall be used for providing the confining pressure.

D.2.3 The loading device shall be a top-loading type, equipped with a closed loop, electro-hydraulic or electro-pneumatic actuators and with a function generator which is capable of applying repeated cycles of haversine-shaped load pulse of the durations as shown in Figure D.2.3.



加载时间	Loading duration	恢复时间	Rest period
循环荷载系数	Cyclic load factor	时间	Time

图 D. 2. 3 半正矢形加载波
Figure D.2.3 Haversine-shaped load pulse

D.2.4 Axial loads shall be measured by using electronic load cells with load capacity and accuracy as defined in Table D.2.4.

Table D.2.4 Requirements for load capacity and accuracy of load cells

Specimen Diameter (mm)	Load capacity (kN)	Accuracy (N)
100	≥9.0	±18.0
150	≥22.0	±22.0

D.2.5 Axial deformation may be measured by optical deformation measurement devices, non-contacting approaching transducers or linear variable differential transducers (LVDTs). Each shall comply with the following requirements.

1 Optical deformation measurement devices shall use analog digital output signals, reading output shall not be less than 0.005mm, frequency response not less 200Hz, linearity not greater than ±1%, range of displacement measurement not shorter than 12.7mm; and the measurable length shall be within the range of 63.5~127.0mm.

2 Both non-contacting approach transducers and linear variable differential transducers shall comply with the requirements of Table D.2.5.

Table D.2.5 Technical requirements for non-contacting approach transducers and Linear Variable Differential Transducers (LVDTs)

Specimen diameter（mm）	Minimum range of measurement (mm)	Resilient displacement (mm)*	Minimum. A.C. output (mV)	LVDT Minimum sensitivity（mV/V）	Non-contacting approach transducer Minimum sensitivity（mV/V）
150	±6.0	0.025	6	2.1	—
100	±2.5	0.015	5	2.8	5

Note: *refers to the minimum resilient displacement at a height equivalent of 2 times the diameter of the specimen.

D.2.6 The data collection system shall be capable of automatic data processing and should be equipped with signal excitation, conditioning and recording devices. The accuracy of measurement shall not be less than ±0.02%; non-linearity not greater than 0.5%.

D.2.7 The tri-axial chamber shall be equipped with and controlled by a pressure conditioning device with pressure capacity not less than 210kPa and accuracy not less than 1.0kPa. The pressure shall be monitored with conventional pressure gauges, manometers, or pressure transducers accurate to 1.0kPa.

D.2.8 Miscellaneous apparatus for the test shall include calipers, micrometer gauge, steel ruler calibrated to 0.5 mm, rubber membranes 0.25 to 0.79 mm thick, rubber O-rings, vacuum source with bubble chamber and regulator, membrane expander, porous stones (subgrade materials), porous bronze discs (base/subbase), scales, moisture content cans and report forms, as required.

D.3.1 Specimen dimensions for unbound granular materials whose maximum nominal size is larger than 19mm: diameter x height (mm) = Ø150×300 and the particles larger than 26.5mm shall be removed when the specimens are prepared. Specimen dimensions for unbound granular material whose maximum nominal size is smaller than 19mm: diameter x height (mm) = Ø100×200.

D.3.2 The target moisture content of laboratory compacted specimens shall be the optimum moisture content determined from impact compaction tests. Deviation of the moisture for laboratory compacted specimens shall not exceed ±0.5% of the target moisture content.

D.3.3 The dry density corresponding to the requirements for site compaction shall be used for laboratory compacted specimens. In the case of lack of site compaction, 95% of the maximum dry density in impact tests may be used and the deviation of laboratory compaction to the target compaction shall not exceed ±1.0%.

D.4 Test Procedure

D.4.1 The testing system shall be calibrated prior to testing each group of specimens.

D.4.2 Specimens shall be prepared as follows:

1 Place the bottom plate and then a moist porous stone under the specimen. In the case of clogging problems in the porous stone, a moist filter paper shall be placed between the specimen and the porous stone.

2 Place the membrane on a membrane expander and apply a vacuum to the membrane expander. Then place the membrane over the specimen, on which a moist porous stone and the top plate are placed. Then remove the vacuum and the membrane expander.

3 Fold up the membrane, and seal the membrane edges, at the top plate and the bottom plate with O-rings or some other pressure seals. The rubber membrane and the specimen shall remain in direct contact with no air permeating. In case of air permeating vacuum oil may be applied on the contact surfaces of the top or bottom plate.

D.4.3 Place the "specimen assembly" on the base plate of the triaxial chamber. Connect the specimen's bottom drainage line to the vacuum source through the medium of a bubble chamber. Apply 35kPa vacuum. If bubbles are present, check and eliminate the cause of the leakage. Leakage through holes in the membrane can frequently be eliminated by coating the surface of the membrane with liquid rubber latex or by using a second membrane. When leakage has been eliminated, disconnect the vacuum supply and carefully clean the O-ring seals and their contacting surfaces.

D.4.4 The equipment shall be assembled as follows :

1 Place the cover plate on the chamber and insert the loading piston to obtain a firm connection with the load cell. Connect the piston rods to the load cell. Tighten the chamber tie rods firmly by using a spanner to fix the triaxial chamber firmly onto the base plate. Check the top plate of the triaxial chamber by means of a well-calibrated leveling instrument to make sure that it is level.

2 For a fixed triaxial chamber: Place the specimen under the axial loading device. Connect the loading cell and the specimen. When the piston contacts the steel ball tightly with a little load applied on the specimen, slowly roll the steel ball to center the position so as to ensure the steel ball and the piston are in line while slightly moving the

specimen to obtain axial loading.

3 For a movable triaxial chamber: Slide the chamber into position under the axial loading device to accurately position it so as to ensure axial loading to be applied on the specimen.

4 Install the measuring system to monitor axial displacement.

D.4.5 Open all drainage valves leading into pressure supply and the triaxial chamber. Apply the pre-conditioning confining pressure of 105.0 kPa to the test specimen. Apply 1000 conditioning repetitions of a maximum haversine-shaped axial load of 231kPa load pulse, for a loading duration of 0.1s and a rest period of 0.9s.

D.4.6 Following the loading sequence shown in row 1 of Table D.4.6, adjust the maximum axial stress to 14.0kPa and set the confining pressure to 20.0kPa. Apply 100 repetitions of the corresponding cyclic axial stress using a haversine-shaped load pulse for a loading duration of 0.1s and a rest period of 0.9s. Record the average recovered displacement for each LVDT separately for the last five cycles. After the completion of loading sequence 1, continue the test by changing the stress level, as for the sequence 2 to the sequence 25, one after another, and record the average recovered displacement. If at any time the permanent strain of the sample exceeds 5 percent during testing, stop the test and report the result.

Tabel D.4.6 Loading Sequence

Sequence No.	Confining pressure, σ_3 (kPa)	Contact stress 0.2σ_3 (kPa)	Cyclic stress σ_d (kPa)	Max. axial stress σ_{mac} (kPa)	No. of loads applications
0	105	21	210	231	1000
1	20	4	10	14	100
2	40	8	20	28	100
3	70	14	35	49	100
4	105	21	50	71	100
5	140	28	70	98	100
6	20	4	20	24	100
7	40	8	40	48	100
8	70	14	70	84	100
9	105	21	105	126	100
10	140	28	140	168	100
11	20	4	40	44	100
12	40	8	80	88	100
13	70	14	140	154	100
14	105	21	210	231	100
15	140	28	280	308	100
16	20	4	60	64	100
17	40	8	120	128	100
18	70	14	210	224	100
19	105	21	315	336	100
20	140	28	420	448	100
21	20	4	80	84	100
22	40	8	160	168	100
23	70	14	280	294	100
24	105	21	420	441	100
25	140	28	560	588	100

D.4.7 At the completion of the triaxial test, reduce the confining pressure to zero and remove the sample from the triaxial chamber. Remove the rubber membrane. Test the moisture content of the specimen and record the result.

D.5.1 The resilient modulus value is computed for each of the last 5 cycles of each loading sequence. These values are subsequently used to calculate the results of all loading sequences.

D.5.2 The model parameters k_1, k_2 and k_3 shall be determined by a nonlinear regression technique and in accordance with the measured data and the constitutive model as expressed by Equation (D.5.2).

$$M_R = k_1 p_a \left(\frac{\theta}{p_a}\right)^{k_2} \left(\frac{\tau_{oct}}{p_a} + 1\right)^{k_3} \tag{D. 5. 2}$$

Where, M_R ——resilient modulus (MPa);

θ ——bulk stress,

$$\theta = \sigma_1 + \sigma_2 + \sigma_3,$$

σ_1, σ_2, σ_3 — principal stresses（MPa）;

τ_{oct} ——octahedral shear stress

k_i ——Regression coefficients $k_1, k_2 \geq 0$， $k_3 \leq 0$

p_a ——normalizing stress (atmospheric pressure)（MPa）。

D.6.1 Test specimen information to be included in a laboratory report shall include: number of specimens, size, density and moisture content and whether or not the permanent deformation during testing reached 5%.

D.6.2 The testing data to be included in a laboratory report shall include: the confining pressure in each loading sequence, the nominal maximum axial stress, the axial load, the axial stress, the recoverable deformation, the resilient modulus and the standard deviations of resilient modulus, regression coefficients k_1, k_2 and k_3 for the constitutive model, the ratio of standard estimated error and the standard deviation, the squared error of relevant factors etc.

APPENDIX E STANDARD METHOD OF TEST FOR DETERMING THE RESILIENT MODULUS OF CHEMICALLY STABILIZED MATERIALS

E.1.1 This method is applicable to testing the uniaxial compressive resilient modulus along the height of a chemically stabilized material specimen.

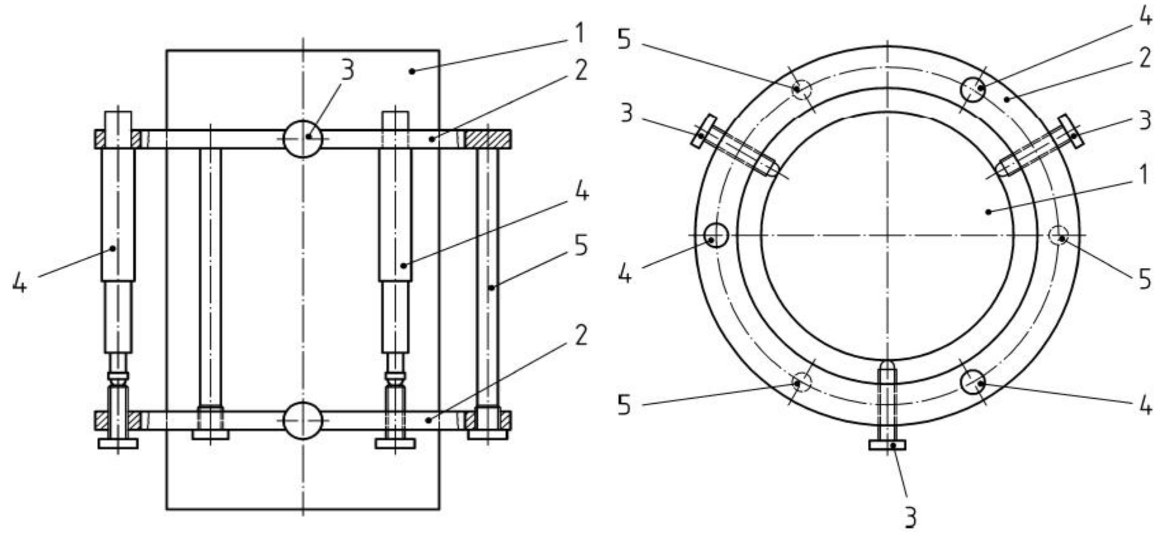
E.1.2 Specimens may be either molded in laboratory or drilled on-site.

E.2.1 The test machine shall be equipped with a servo hydraulic or servo pneumatic system. Measurement accuracy shall not be less than ±1%. The loading rate shall be computer controlled capable of continuous loading and unloading or maintaining a constant load. The maximum measurement range shall not be less than 300kN.

E.2.2 The loading platens may be hard steel plates or high tensile aluminum plates with a diameter not be smaller than the diameter of the specimen, shall be placed on top and under the specimen.

E.2.3 Axial displacement, Δl, shall be measured at the cylindrical side of a specimen by use of a displacement sensor. The measurement range of the displacement sensor shall not be shorter than 5mm with accuracy up to 1μm. Measurement points shall be on three paralleled straight lines on the diameter at an angle 120° to each other. The spacing L between any two measuring points shall not be less than 4 times the maximum size of the aggregate. The spacing from a measuring point to either end face of the specimen shall not be shorter than 15mm.

E.2.4 Displacement sensors shall be positioned as shown in Figures E.2.4a and E.2.4b: the rigid yokes shall be fixed on the cylindrical side of the specimen by bolts with hemispherical heads. Three displacement sensors shall be fixed on one of the rigid yokes.



a) Front view b) Top view

1- Specimen; 2- rigid yoke; 3- bolt; 4- sensor; 5 removable bolts

Figure E.2.4 Sensor Positions

E.2.5 A computer controlled data collecting system shall be used for recording the loads and specimen axial deformation at every 0.01 seconds.

E.2.6 Double faced saw discs shall be used for specimen cutting unless it can be guaranteed that the top and bottom faces are parallel.

E.3.1 Test specimens may be either molded cylindrical specimens prepared in accordance with T0843 of the current *JTG E51 Test Methods of Materials Stabilized with Inorganic Binders for Highway Engineering* or cored specimens drilled either from a laboratory prepared beam or from the pavement on site. Test specimens shall be of consistent shape with smooth and flat face. Three specimen sizes may be used: diameter × height (mm) = 100 Ø ×150, 150 Ø ×150 or 150 Ø ×300.

E.3.2 Cut both ends of the specimen by using a cutting machine and ensure that the height of the specimen shall be 150±2.5mm or 300±2.5mm. The allowance is ±0.05mm for the depth difference of the texture on the diameter. Either end face shall be perpendicular to the axis of the specimen with a tolerance of ±1° otherwise the specimen shall be discarded.

E.3.3 Measure the diameter of the specimen at three positions, one at the midheight and other two at 1/3 height of the specimen from each end face. Take two readings at each measurement position. Rotate the specimen 90° after the first measurement and measure it again. Calculate the average value and standard deviation of the six diameter measurements. Any specimen that fails to satisfy the allowable standard deviation, that is 2.5mm, shall be discarded. The average figure of the six measurements shall be taken as the diameter of the specimen, which satisfies the criteria accurate to 0.1mm.

E.3.4 Both end faces of the specimen shall be carefully finished with cement mortar. Place the specimen standing on a table. Apply a thin film of rapid hardening cement mortar on the top face of the specimen and then apply a little 0.25~0.5mm fine sand on the mortar. Place a round steel plate, diameter is slightly larger than the specimen, on the top face, press and rotate the steel plate to make a flat and smooth top surface. Remove the steel plate while rotating. If there is cement mortar stuck to the steel plate surface, the mortar finishing shall be redone. Allow the specimen to stand for 4 hours after one of the end faces is finished, and then repeat the process on the other face. The size of the finished specimens shall conform to the requirements in E.3.2 of the Specifications.

E.3.5 Curing of the test specimens shall be executed in accordance with the provisions in T0843 of the current *JTG E51 Test Methods of Materials Stabilized with Inorganic Binders for Highway Engineering.*

E.3.6 The number of specimens shall not be less than 9 if the maximum nominal size of granular material in the chemically stabilized material is not larger than 26.5mm; or 15 specimens if the maximum nominal size is greater than 26.5.

E.4.1 Specimens shall be submerged for 24 hours, and then taken out of the water bath, wipe off surface water and weighed. Mass difference of the specimen after curing and after preparation shall not be greater than 2%, otherwise the specimen fails. The duration from the time when specimens are taken out of the water bath to the test completion shall be as short as possible.

E.4.2 Apply a little amount of 0.25~0.50mm fine sand on top face of the specimen, place loading platten on the top face, press and rotate to fill any unevenness on the top face with fine sand and allow surplus to flow off. Place the specimen in the compression machine and carefully align the axis of the specimen with the center of the loading platen.

E.4.3 The compression machine shall apply the loading continuously and uniformly at a loading rate of 1mm/min until the specimen fails.

E.4.4 Strain ε shall be obtained by calculating the average of the deformation values measured by the three displacement sensors. Data shall be record during the test and the ‘load-strain’ curve shall be plotted as shown in Figure E.4.3. In the event that the starting point the ‘load-strain’ curve is not exactly at point zero or the initial stage of the curve fluctuates, adjustment shall be done to the starting point of the curve by using the tangent to the curve to provide a new zero point.

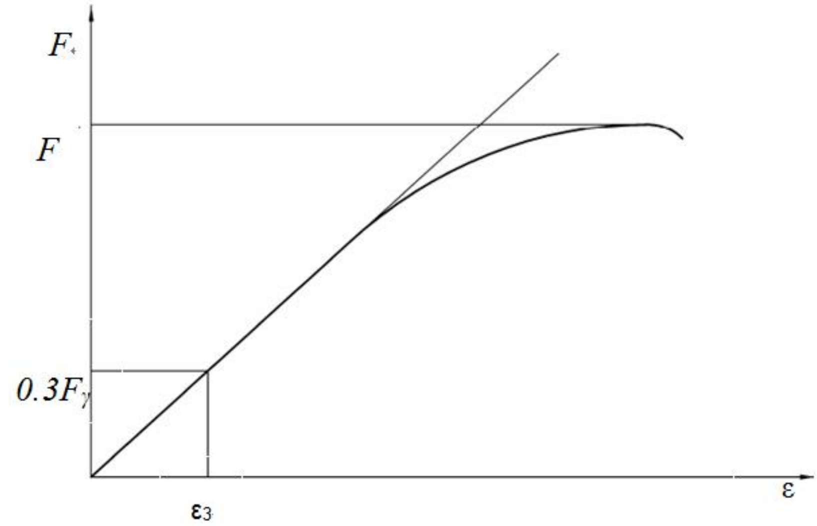


Figure E.4.4 'Load-Strain' curve

E.4.5 The resilient modulus shall be calculated by equation (E.4.5) in accordance with the maximum load and the compressive strain corresponding to 0.3 times of the maximum load.

$$E = \frac{1.2F_r}{\pi \cdot D^2 \cdot \varepsilon_3} \qquad \text{(E.4.5)}$$

Where, E——resilient modulus (MPa)

F_r——Maximum load (N)

D——diameter of specimen (mm)

ε_3——vertical strain at $0.3F_r$,: $\varepsilon 3=\Delta l/L$

E.5.1 The resultant resilient modulus shall be rounded up to an integer value.

E.5.2 Abnormal values that exceed the mean value of a group by three times the standard variation will be removed.

E.5.3 The allowable coefficient of variation of the test results shall not be greater than 10% in the case where the nominal maximum size of the aggregate in chemically stabilized materials is not larger than 26.5mm; or not greater than 15% if the nominal maximum size is larger than 26.5mm. Otherwise, additional tests are required and the additional results and original ones shall be used to calculate the coefficient of variation until the above requirement is satisfied.

E.5.4 The test report shall include the method of specimen preparation, the specimen sizes, the mass after curing and after preparing, the curing and storing environments, age of the specimen, test date, the value of resilient modulus of each specimen, the minimum and maximum values, average values, standard deviation and coefficient of variation of the test results.

APPENDIX F STANDARD METHOD OF TEST FOR DETERMINING THE UNIAXIAL PENETRATION STRENGTH OF ASPHALT MIXTURE

F.1.1 This method is applicable for testing the penetration strength of asphalt mixtures for mix design or for checking the high temperature stability of asphalt mixtures after completion.

F.1.2 The method is applicable to the penetration strength tests on either laboratory prepared specimens or in-situ drilled cores. A typical test temperature shall be 60℃ but other temperatures may be used as required.

F.1.3 The method uses cylindrical specimens of asphalt mix, which are 100±2.0mm or 150±2.0mm in diameter, and 100±2.0mm high. Cylindrical specimens with other lengths may be used as required.

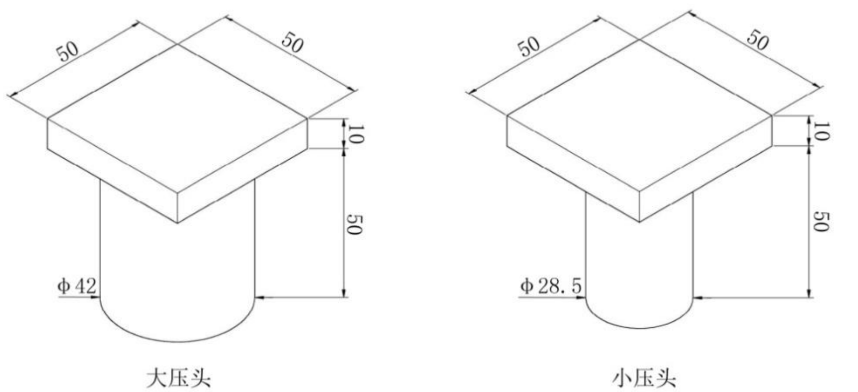
F.2.1 A universal material testing machine should be used. Other types of pavement material testing machines may be used subject to the following requirements:

1 The applied load shall not exceed 80% or be less than 20% of the measurement range, which should be 10kN maximum and have a 10N scale.

2 A temperature control chamber shall be able to control temperature to an accuracy of ±0.5℃.

3 The loading rate shall be maintained at 1mm/min as required. The testing machine should be equipped with a servo system to keep loading at a constant speed.

F.2.2 The material of penetrating top units shall be Q235 stainless steel with Rockwell hardness between 10~30. The upper part of the top unit is a 50mm×50mm×10mm thin plate; the lower part is a cylinder, either 42 Ø mm×50mm for 150mm diameter specimens, or 28.5 Ø mm×50mm for 100mm diameter specimens; as shown in Figure F2.2.



大压头 Large top-unit	小压头 Small top unit

Figure F2.2. Top Units for Penetration Test

F.2.3 Specimens should be prepared and shaped by using rotary kneading compaction apparatus. Voids of the asphalt mixture specimens should be controlled to the actual voids in the pavement.

F.2.4 Other apparatus shall include oven, calipers, balance to weigh submerged specimens, etc.

F.3.1 Specimens of an asphalt mixture shall be prepared as required for the test. Standard diameter of a specimen shall be 150mm, or may be 100mm for the mixture which nominal maximum size is equal to or smaller than 16mm. The height of a specimen shall be 100mm.

F.3.3 The asphalt mixture sample during field sampling shall be stored in a thermo box and shall be prepared into specimens before cooling to or below the temperature required for specimen compaction. Reheating is not allowed.

F.3.3 Laboratory made specimens shall be prepared in accordance with *JTG E20 Method of Testing Bitumen and Asphalt Mixture for Highways, Test T076*. Other specimen compaction methods may be used provided that the specimen dimensions comply with the relevant requirements.

F.3.4 After being compacted in laboratory, a specimen of ordinary asphalt mixture shall be stored at ambient temperature for 12 hours or longer; a specimen of modified asphalt mixture shall be stored for 48 hours but not longer than 7 days. After storing as required, the specimens shall be tested for void content in accordance with *JTG E20 Method of Testing Bitumen and Asphalt Mixtures, Test T 0705*.

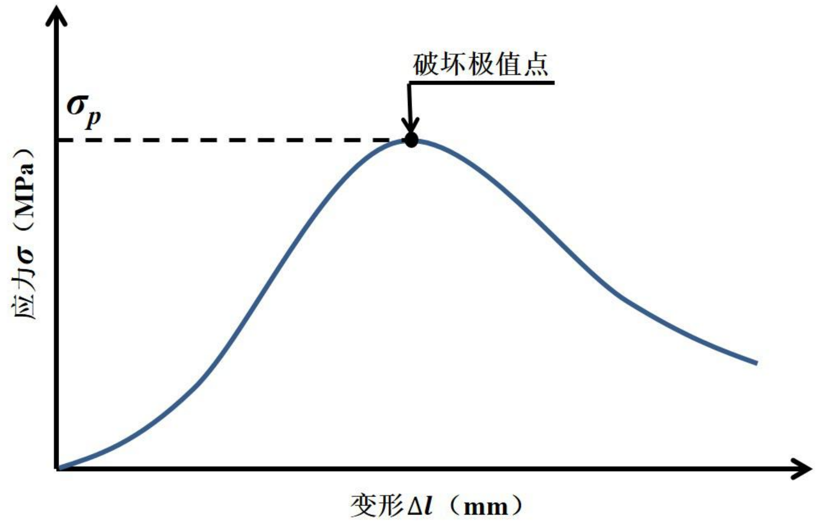
F.3.5 When specimens are drilled by coring, the cores shall have a consistent shape of a cylinder. Uneven end surfaces shall be cut off by using cutting blades.

F.3.6 Specimens should be placed in an environmental chamber at a constant temperature of 60±0.5℃ for 5 to 6 hours. In case that an environmental chamber is not available or has insufficient space, the specimens may be store in an oven at the same temperature. The top unit shall be placed into an environment box for maintaining at test temperature.

F.4.1 Place a specimen on the test deck. The surface for the penetration test shall be the one in contact with the top unit and was the upper surface during specimen preparation. Adjust position of the specimen to align the top unit with the center of the specimen.

F.4.2 Adjust the top unit to a position 1mm away from the specimen surface, then adjust the position of the top unit to touch the specimen surface with a contact load up to but not exceeding 0.05kN.

F.4.3 Initiate the loading at a loading rate of 1mm/min, record the compressive force and displacement. Stop the test when the stress falls to 90% of the maximum value. Take the maximum stress value at failure as the penetration strength of the specimen.



应力σ(MPa) Stress σ(MPa)	破坏极值点 Limit point at failure
	变形 Δl(mm) Displacement Δl(mm)

Figure F.4.3 Typical Stress-Displacement Relation in Uniaxial Penetration Test

F.5.1 Take the reading of P, the maximum load during penetration, with an accuracy to 1N.

F.5.2 Compute the penetration strength of an asphalt mixture with standard height.

$$R_\tau = f_\tau . \sigma_p \tag{F.5.2-1}$$

$$\sigma_p = \frac{P}{A} \tag{F.5.2-2}$$

Where, R_τ——Penetration strength（MPa）;

σ_p——Penetration stress（MPa）;

P——Maximum load on specimen at failure（N）;

A——cross-sectional area of top unit（mm^2）;

f_τ——factor of penetration stress: $f_\tau = 0.35$ for specimens 150mm in diameter，$f_\tau = 0.34$ for specimens 100mm in diameter.

F.5.3 For specimens in which the thickness is not 100mm, the factor of penetration stress shall be adjusted in the following circumstances.

1 For 150mm diameter specimens, the factor of penetration stress shall be calculated by Equation (F.5.3-1) where the thickness of the specimen is in the range 38mm≤h＜100mm

$$f_{\tau}=0.0023h + 0.12 \tag{F.5.3-1}$$

2 For 100mm diameter specimens, the factor of penetration stress shall be calculated by Equation (F.5.3-2) where the thickness of the specimen should be in the range: 38mm≤h＜100mm

$$f_{\tau}=0.0012h + 0.22 \tag{F.5.3-2}$$

F.5.4 For the specimens obtained by coring in the field, the calculated penetration strengths shall be multiplied by 1.15, which is a correction factor.

F.6.1 For a particular type of asphalt mixture or for a particular section of pavement, a group of replicate tests should include 5 to 6 specimens, of which the average value should be taken as the result of the testing. If one of the measured values in a group is greater than the average by k times the standard deviation, such data shall be discarded and the average of the rest shall be taken as the test result as long as there are at least four effective data points. The value of k is 1.67 or 1.82 corresponding to the number of test specimens, n = 5 or 6.

F.6.2 The specimen dimensions, the method for obtaining the specimens , the density, void of the specimens, the type of loading equipment, and the test results shall be recorded in the test report.

APPENDIX G TEMPERATURE FACTORS AND EQUIVALENT TEMPERATURE

G.1.1 The asphalt surface course, base course or subbase course of a pavement structure may comprise two or more layers of different materials. Such pavement structural layers shall be converted to either an equivalent asphalt surface course or equivalent base by means of equations (G.1.1-1) and (G.1.1-2). Asphalt bound base course shall be converted to equivalent asphalt surface course. In the case of more than two layers of different materials in an asphalt course, the conversion shall be conducted one layer after another downwards by using Equations (G.1.1-1) and (G.1.1-2) repeatedly so as to obtain a simplified structure comprising equivalent asphalt surface course, equivalent base course (including subbase course) and subgrade.

$$h_i^* = h_{i.1} + h_{i.2} \qquad \text{(G.1.1-1)}$$

$$E_i^* = \frac{E_{i.1}h_{i.1}^3 + E_{i.2}h_{i.2}^3}{\left(h_{i.1} + h_{i.2}\right)^3} + \frac{3}{h_{i.1} + h_{i.2}}\left(\frac{1}{E_{i.1}h_{i.1}} + \frac{1}{E_{i.2}h_{i.2}}\right)^{-1} \qquad \text{(G.1.1-2)}$$

Where h_i^* and E_i^* —— equivalent thickness (mm) and equivalent modulus (MPa); the subscript *i=a for asphalt surface course*，*i=b for base course.*

G.1.2 A temperature shift factor is obtained by shifting the damage to a reference pavement structure at various temperatures to the damages to the reference pavement structure at reference temperature. Table G.1.2 gives temperature shift factors for pavement structural design in some regions of China. The factors for other regions may be taken by referring to those regions where climate conditions are similar. The temperature data should be the average of the data for the last 10 successive years.

Table G.1.2 Temperature Shift Factors and Equivalent Temperatures

Locality	Province，Autonomous Region or Municipalities	Average Temperature in hottest month (℃)	Average Temperature in coldest month (℃)	Annual Average Temperature (℃)	Temperature Shift Factor		Reference Equivalent Temperature (℃)
					Tensile strain at the bottom of the asphalt layer Tensile stress at the bottom of the chemically stabilized layer	Compressive strain at top surface of subgrade	
Beijing	Beijing	26.9	-2.7	13.1	1.23	1.09	20.1
Jinan	Shandong	28.0	0.2	15.1	1.32	1.17	21.8
Rizhao	Shandong	26.0	-2.0	12.7	1.21	1.06	19.4

Taiyuan	Shanxi	23.9	-5.2	10.5	1.12	0.98	17.3
Datong	Shanxi	22.5	-10.4	7.5	1.01	0.89	15.0
Houma	Shanxi	26.8	-2.3	13.0	1.23	1.08	19.9
Xi-an	Shaanxi	27.5	0.1	14.3	1.28	1.13	20.9
Yan-an	Shaanxi	23.9	-5.3	10.5	1.12	0.98	17.3
Ankang	Shaanxi	27.3	3.7	15.9	1.35	1.19	21.7
Shanghai	Shanghai	28.0	4.7	16.7	1.38	1.23	22.5
Tianjin	Tianjin	26.9	-3.4	12.8	1.22	1.08	20.0
Chongqing	Chongqing	28.3	7.8	18.4	1.46	1.31	23.6
Taizhou	Zhejiang	27.7	6.9	17.5	1.42	1.26	22.8
Hangzhou	Zhejiang	28.4	4.5	16.9	1.40	1.25	22.8
Hefei	Anhui	28.5	2.9	16.3	1.37	1.22	22.6
Huangshan	Anhui	27.5	4.4	16.6	1.38	1.23	22.3
Fuzhou	Fujian	28.9	11.3	20.2	1.55	1.40	24.9
Jian-ou	Fujian	28.2	8.9	19.1	1.49	1.35	24.1
Dunhuang	Gansu	25.1	-8.0	9.9	1.10	0.97	17.6
Lanzhou	Gansu	22.9	-4.7	10.5	1.12	0.98	17.0
Jiuquan	Gansu	22.2	-9.1	7.8	1.02	0.90	15.0
Guangzhou	Guangdong	28.7	14.0	22.4	1.66	1.52	26.5
Shantou	Guangdong	28.6	14.4	22.1	1.64	1.50	26.1
Shaoguan	Guangdong	28.5	10.3	20.4	1.56	1.42	25.2

Heyuan	Guangdong	28.4	13.1	21.9	1.63	1.49	26.1
Lianzhou	Guangdong	27.6	11.0	20.3	1.55	1.40	24.8
Nanning	Guangxi	28.4	13.2	22.1	1.64	1.51	26.3
Guilin	Guangxi	28.0	8.1	19.1	1.49	1.35	24.2
Guiyang	Guizhou	23.7	4.7	15.3	1.31	1.15	20.1
Zhengzhou	Henan	27.4	0.6	14.7	1.30	1.15	21.2
Nanyang	Henan	27.3	1.7	15.2	1.32	1.17	21.4
Gushi	Henan	28.1	2.6	16.0	1.36	1.21	22.3
Heihe	Heilongjiang	21.5	-22.5	1.0	0.80	0.77	10.7
Mohe	Heilongjiang	18.6	-28.7	-3.9	0.67	0.73	6.4
Tsitsihar	Heilongjiang	23.0	-19.7	3.5	0.88	0.81	13.0
Shenyang	Liaoning	24.9	-11.2	8.6	1.06	0.94	16.9
Dalian	Liaoning	24.8	-3.2	11.6	1.16	1.02	18.2
Chaoyang	Liaoning	25.4	-8.7	9.8	1.10	0.97	17.7
Erenhot	Inner Mongolia	24.0	-17.7	4.8	0.92	0.84	14.2
Dongsheng	Inner Mongolia	21.7	-10.1	6.9	0.98	0.87	14.2
Ejin Banner	Inner Mongolia	27.4	-10.3	9.5	1.10	0.97	18.2
Hailar	Inner	20.5	-24.1	0.0	0.77	0.76	9.8

	Mongolia						
Keyoqian Banner	Inner Mongolia	20.8	-16.7	3.0	0.86	0.79	11.4
Tongliao	Inner Mongolia	24.3	-12.5	7.3	1.01	0.90	15.7
Silinhot	Inner Mongolia	21.5	-18.5	3.3	0.87	0.80	12.2
Shijiazhuang	Hebei	26.9	-2.4	13.3	1.24	1.10	20.3
Chengde	Hebei	24.4	-9.1	9.1	1.07	0.95	16.8
Handan	Hebei	26.9	-2.3	13.5	1.25	1.10	20.5
Wuhan	Hubei	28.9	4.2	17.2	1.41	1.27	23.3
Yichang	Hubei	27.5	5.0	17.1	1.40	1.25	22.7
Changsha	Hunan	28.5	5.0	17.2	1.41	1.26	23.1
Changning	Hunan	29.1	6.0	18.1	1.45	1.31	23.9
Xiangxi	Hunan	27.2	5.3	16.9	1.39	1.24	22.4
Changchun	Jilin	23.6	-14.5	6.3	0.97	0.87	14.9
Yanji	Jilin	22.2	-13.1	5.9	0.95	0.86	13.9
Nanjing	Jiangsu	28.1	2.6	15.9	1.35	1.20	22.1
Nantong	Jiangsu	26.8	3.6	15.5	1.33	1.17	21.2
Nanchang	Jiangxi	28.8	5.5	18.0	1.45	1.30	23.8

Ganzhou	Jiangxi	29.1	8.3	19.6	1.52	1.38	25.0
Yinchuan	Ningxia	23.8	-7.5	9.5	1.08	0.95	16.8
Guyuan	Ningxia	19.6	-7.9	6.9	0.97	0.86	13.2
Xining	Qinghai	17.3	-7.8	6.1	0.94	0.84	11.9
Haibei	Qinghai	11.3	-13.6	0.0	0.74	0.74	5.5
Golmud	Qinghai	18.2	-8.9	5.7	0.93	0.83	11.9
Yushu	Qinghai	12.9	-8.0	3.5	0.85	0.78	8.2
Guolo	Qinghai	9.9	-12.9	-0.3	0.73	0.74	4.7
Chengdu	Sichuan	25.5	5.8	16.5	1.37	1.21	21.5
E-meishan	Sichuan	11.7	-5.8	3.4	0.84	0.77	7.4
Garz	Sichuan	13.9	-4.6	5.7	0.92	0.82	10.0
Aba	Sichuan	11.0	-10.0	1.7	0.79	0.75	6.4
Luzhou	Sichuan	27.0	7.6	17.9	1.43	1.28	22.9
Mianyang	Sichuan	26.2	5.5	16.7	1.38	1.22	21.9
Panzhihua	Sichuan	26.4	12.8	20.8	1.57	1.42	24.6
Lhasa	Tibetan	16.2	-0.9	8.4	1.01	0.88	12.5
Aqsu	Xinjiang	24.2	-7.7	10.6	1.13	0.99	18.0
Altay	Xinjiang	22.0	-15.4	5.0	0.92	0.84	13.4
Qomul	Xinjiang	26.3	-10.0	10.1	1.12	0.99	18.5
Khotan	Xinjiang	25.7	-4.1	12.9	1.22	1.08	20.0
Kashgar	Xinjiang	25.4	-5.0	11.9	1.18	1.04	19.1
Ruoqiang	Xinjiang	27.9	-7.2	12.0	1.19	1.06	20.2

Tacheng	Xinjiang	23.3	-10.0	7.7	1.02	0.90	15.3
Turpan	Xinjiang	32.3	-6.4	15.0	1.34	1.21	24.1
Urumqi	Xinjiang	23.9	-12.4	7.4	1.01	0.90	15.7
Yanqi	Xinjiang	23.4	-11.0	8.9	1.06	0.94	16.8
Yining	Xinjiang	23.4	-8.3	9.4	1.08	0.95	16.8
Kunming	Yunnan	20.3	8.9	15.6	1.30	1.13	18.7
Tengchong	Yunnan	19.9	8.5	15.4	1.29	1.12	18.5
Mengzi	Yunnan	23.2	12.7	18.8	1.46	1.29	21.9
Lijiang	Yunnan	18.7	6.2	12.8	1.18	1.02	16.1
Jinhong	Yunnan	26.3	17.2	22.7	1.66	1.51	25.6
Haikou	Hainan	28.9	18.4	24.6	1.77	1.65	27.9
Sanya	Hainan	29.1	22.0	26.2	1.85	1.74	28.8
Xisha	Hainan	29.3	23.6	27.0	1.89	1.79	29.3

G.1.3 Temperature shift factors shall be calculated by using Equations (G.1.3-1) ～ (G.1.3-15).

$$k_{Ti} = A_h A_E \hat{k}_{Ti}^{\,1+B_h+B_E} \qquad \text{(G.1.3-1)}$$

Where k_{Ti}——temperature shift factor; subscript i=1 corresponds to the analysis of fatigue cracking in asphalt bound layer; i=2 corresponds to the analysis of fatigue cracking in chemically stabilized layers; and i=3 corresponds to the analysis of vertical compressive strain on the top surface of the subgrade;

$\hat{k}_{Ti}$——temperature shift factor for reference pavement structure, taken from Table G.1.2 by referring to the location;

A_h、B_h，A_E、B_E—— functions in relation to the thickness and modulus of surface course or base course, calculated by using Equations (G.1.3-2) ~ (G.1.3-13):

For fatigue cracking in asphalt bound layer:

$$A_E = 0.76(\lambda_E)^{0.09} \quad \text{(G.1.3-2)}$$

$$A_h = 1.14(\lambda_h)^{0.17} \quad \text{(G.1.3-3)}$$

$$B_E = 0.14\ \ln(\lambda_E / 20) \quad \text{(G.1.3-4)}$$

$$B_h = 0.23\ \ln(\lambda_h / 0.45) \quad \text{(G.1.3-5)}$$

For fatigue cracking in chemically stabilized layers:

$$A_E = 0.10\lambda_E + 0.89 \quad \text{(G.1.3-6)}$$

$$A_h = 0.73\lambda_h + 0.67 \quad \text{(G.1.3-7)}$$

$$B_E = 0.15\ \ln(\lambda_E / 1.14) \quad \text{(G.1.3-8)}$$

$$B_h = 0.44\ \ln(\lambda_h / 0.45) \quad \text{(G.1.3-9)}$$

For vertical strain on the top surface of the subgrade:

$$A_E = 0.006\lambda_E + 0.89 \quad \text{(G.1.3-10)}$$

$$A_h = 0.67\lambda_h + 0.70 \quad \text{(G.1.3-11)}$$

$$B_E = 0.12\ \ln(\lambda_E / 20) \quad \text{(G.1.3-12)}$$

$$B_h = 0.38\ \ln(\lambda_h / 0.45) \quad \text{(G.1.3-13)}$$

Where in Equation (G.1.3-2) to Equation (G.1.3-13)

λ_E——ratio of equivalent moduli of the surface course to the base course, calculated by Equation (G.1.3-14).

$$\lambda_E = E_a^* / E_b^* \quad \text{(G.1.3-14)}$$

λ_h——ratio of equivalent thicknesses of the surface course to the base course, calculated by Equation (G.1.3-15).

$$\lambda_h = h_a^* / h_b^* \quad \text{(G.1.3-15)}$$

G.2.1 For analyzing permanent deformation, the equivalent temperature of an asphalt bound layer shall be obtained by using Equation (G.2.1).

$$T_{pef} = T_\xi + 0.016h_a \quad \text{(G.2.1)}$$

Where, T_{pef}——Equivalent temperature of asphalt bound layer (℃);

h_a——thickness of asphalt bound layer (mm);；

T_ξ——reference equivalent temperature, obtained by referring to Table G.1.2.

Wording Explanation for the Specifications

The strictness in execution of the Specifications are 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.

1 GENERAL PROVISIONS

1.0.6 Specific soil and climate characteristics raise special requirements for highway pavement. These special requirements shall be carefully considered in pavement design for such problem susceptible regions in addition to the regular requirements for pavement performance in normal areas

3 Design Criteria

3.0.2 Values listed in Table 3.0.2 are the minimum requirements for the design life of asphalt pavements of new highways. For upgrading projects, the design life is usually required to be the same for both the rehabilitated pavement that received an overlay and the new pavement on the road widening. For pavement strengthening projects, the pavement design life may be determined differently, namely either as the remaining life by referring to the original design life, or for an extended design life that is the remaining life of the existing pavement plus the additional life extended by the strengthening.

3.0.3 The previous edition took the 100kN single axle-dual tire load as the design load (standard loads). According to an analysis on the data of axle loads collected from more than 10 provinces nationwide in 2012, fewer than 10% of the trucks on motorways were over-loaded by 30% or more; and fewer than 5% of the trucks were overloaded by 50% or more. Furthermore, a part of overloading effect is considered in the equivalency conversion of axle loads. Therefore, the design load remains unchanged in this edition of the Specifications.

3.0.4 Besides the cumulative ESAL, traffic load classification was introduced in the previous edition, which divides traffic loads into four classes by taking the cumulative ESAL and daily average truck traffic volume as indicators.

Multiple design indicators are adopted in this edition of the Specifications. Different design indicators were based on different axle conversion factors, thus corresponding to different cumulative ESAL. Had cumulative ESAL been used for traffic load classification, different classification criteria would have been required for design indicators, which would have inconvenienced practical application. Taking all the above into account, the traffic loads are classified by the cumulative traffic volumes of large buses and large trucks.

Recognizing particular characteristics in terms of axle loads, traffic composition and other respects along the container or coal transportation corridors, an extra-heavy class was introduced in addition to the four traffic load classes stated in the previous edition.

In this edition of the Specifications, five criteria have been adopted for controlling different types of damage to pavements. The prediction model for fatigue cracking in asphalt bound material layers and chemically stabilized material layers have been revised. Three criteria have been introduced respectively for permanent deformation of asphalt layers, for vertical strain on the subgrade and for low temperature cracking in seasonal frost regions.

3.0.6 Referring to JTG H20-2007 Highway Performance Assessment Standards, SHELL pavement design methods, Asphalt Institute (AI) Pavement Design Methods and AASHTO Mechanistic – Empirical Pavement Design Guide (MEPDG), this edition of the Specifications provides the criteria for permanent deformation in asphalt bound material layers as shown in Table 3.0.6-1.

The requirements for low temperature thermal cracking indicators specified in Table 3.0.6 are the criteria for taking-over of pavement works on completion. Only pavement low temperature thermal cracking is taken into account. Neither reflective cracking nor longitudinal cracking is included.

4 PAVEMENT STRUCTURAL DESIGN

4.1.1 There are significant differences between various pavement structures in terms of mechanistic properties, service functions, long-term deterioration and damage processes. All these factors need to be taken into account when a pavement structural design is conducted.

An integrated subgrade-pavement design requires that the subgrade shall have sufficient bearing capacity and shall be in a suitable moisture status compatible with the pavement structure.

Lifecycle cost effectiveness means the optimal solution for the pavement structure, which is identified and designed by taking into account not only the initial construction costs but also the maintenance and rehabilitation costs during the design life.

4.1.2 In China, a pavement structure was usually divided into surface course, base course, subbase course and bedding or selected layers. In the previous edition of the Specifications, some of the specific functions such as resilient modulus improvement, frost protection or subsurface drainage were assigned to bedding layers, which often caused confusion in design and practical application. In this edition of the Specifications, these functions have been specifically identified and categorized.

In some cases, an unbound granular layer or a chemically stabilized layer may be placed to increase subgrade resilient modulus or to improve subgrade moisture resistance. Regarding such layers as a part of the pavement structure may cause some conflicts with the fundamental requirements of a subgrade. In the United States and in European countries, this type of layers is taken as a part of the subgrade. Therefore, in this Edition of the Specifications, these subgrade-related layers are categorized as part of the subgrade and are termed subgrade improvement layers while the others, such as frost-protection layers and sub-drainage layer, are taken as the functional layers of a pavement structure.

4.2.1 to 4.2.3 For the selection of pavement structures, attention needs to be drawn to the material properties, the structural characteristics, the damage models and the performance deterioration process of various potential pavement structures. The main damage models of an asphalt pavement are as shown in Table 4-1.

Table 4-1 Main damage models of asphalt pavements

Structural type	AP with unbound granular base AP with unbound granular subbase and asphalt bound base			AP with chemically stabilized base AP with chemically stabilized subbase and asphalt bound base	
Thickness of asphalt bound materials (mm)	⩾150	150~50	⩽50	⩾150	<150
Damage model	Permanent deformation in asphalt mixture Fatigue cracking in asphalt mixture	Fatigue cracking in asphalt mixture; Permanent deformation in asphalt mixture	Rutting	Rutting; Fatigue cracking in base; Reflection cracking in surface course	.Fatigue cracking in base; Reflection cracking in surface course
Seasonal frost regions	Low temperature thermal cracking in surface course				

AP – asphalt pavement

The asphalt pavement with a chemically stabilized base has a high bearing capacity and is thus suitable for all classes of traffic loads. The main distresses in this type of pavement are fatigue cracking in the base course and reflection cracking in the surface course. Rain and melting snow water may infiltrate through reflection cracks and cause distresses like pumping, which leads to interlayer voids under the base course， and other damages. Using unbound granular materials as subbase layer or subgrade improvement layer may reduce reflection cracks and consequently pumping and voids.

The asphalt pavement with an unbound granular base has no problem of reflection cracks. However its asphalt surface course bears comparatively large flexural stress due to the bending action, thus fatigue of asphalt surface course is the major damaging model (note by EN: which can be restored by sealing maintenance operations). In addition, permanent deformation may occur in an asphalt surface course, unbound granular base and subgrade of this type of asphalt pavement structure, thus rutting is the other problem to which attention needs to be paid.

The asphalt pavement with asphalt bound base course is suitable for all classes of traffic loads. Where the subbase is made of chemically stabilized materials, the performance of this type of pavement structure is similar to the asphalt pavement with chemically stabilized base. Because the asphalt layer is comparatively thick, this type of pavement has higher bearing capacity and better capability of mitigating reflection cracks. Where the subbase is made of unbound granular materials, the performance of this type of pavement structure is similar to the asphalt pavement with unbound granular base.

The asphalt pavement with a cement concrete base has comparatively higher bearing capacity, and is thus suitable for heavy to extra-heavy traffic load classes. In addition to the regular damage to the cement concrete pavement, the most significant distress is the reflection cracking at cement concrete slab joints and the permanent deformation in asphalt surface course.

4.2.5 Asphalt pavements with a chemically stabilized or cement concrete base have a high potential for reflection cracking. In high rainfall regions these reflection cracks will rapidly develop into pumping, interlayer voids and other distresses, which may accelerate the deterioration of the asphalt pavement. Therefore, it will be necessary to take specific reflection cracking control measures to mitigate the damaging of pumping and interlayer voids, such as to place a drainage layer or a subgrade improvement layer under the unbound granular base or the cement concrete base.

4.2.6 Reflection cracking is the most common distress in an asphalt pavement with a chemically stabilized base. Therefore, materials with good crack resistance shall be selected for the chemically stabilized base. Increasing the thickness of asphalt mixture surface course and placing a functional layer of stress absorption or a layer of reinforcement may provide positive effects on reducing or mitigating reflection cracking.

4.3 Subgrade

4.3.4 According to the depth of the water table affected by ground water or constantly standing surface water which may affect the moisture within the subgrade influence zone of vehicle loads, subgrade moisture status is classified into three types: the wet status as a result of ground water, the dry status because of climatic factors and the moderate status subject to both ground water and climate.

4.4 Base and Subbase

4.4.3 In recent years the practice of pavement recycling, cold recycling is one of the effective methods for either reclamation or in-place recycling of the materials removed from an existing asphalt pavement by road milling. The cold recycled material can satisfy the requirements for a pavement base course under all classes of traffic loads. It is recognized that hot recycled pavement materials produced in a central plant have similar properties for road use to those of newly mixed asphalt but the cost is higher than cold recycled asphalt. Therefore it is recommended to use hot recycling for the base course under heavy and higher class of traffic loads.

4.4.4 In order to reduce or mitigate reflection cracking, a layer of graded unbound crushed stone or a layer of asphalt treated crushed stone, either open graded or semi-open graded, may be placed between the chemically stabilized layer and the asphalt bound layer. If an unbound crushed stone layer is used, the tensile strain at bottom of the asphalt mixture layer needs to be verificated.

4.5.3 The wearing course of an open graded asphalt mixture is permeable. Precipitation infiltrates through the wearing course and is then drained off along the top surface of the layer beneath. However, the layer beneath wearing course is usually made of an asphalt mixture with medium or coarse nominal size, which may not be water proof (due to particle segregation or other causes). Therefore, in order to prevent the lower layers from water damage, a waterproofing membrane is necessary under an open graded wearing course, as specified herein.

4.5.4 In order to ensure compaction and reduce segregation during construction, this edition specifies the minimum rate of layer thicknessl to the norminal maximum size of the asphalt bound material, and the minimum layer thickness of a asphalt bound layer in terms of nominal maximum aggregate sizes.

4.6.2 Granular layer provides drainage function. One is to prevent the pavement moisture content from being affected by a wet subgrade, fracture groundwater or capillary water; the other is to drain the infiltrated water away from the pavement as soon as possible so as to avoid infiltrated water entering into the subgrade.

4.6.3 Considering that cold recycled materials are sensitive to moisture, a seal coat should be placed between a cold recycled material layer and an asphalt bound layer.

4.6.4 Pavements for extra-heavy, very-heavy or heavy class of traffic loads require high interlayer bond strength. For this reason, it is specified that modified bituminous emulsion, penetration grade bitumen or modified asphalt should be used as tack coats. However, due to difference in material properties, it is difficult to form an effective bond at the interface between a cement concrete slab and asphalt surface course so that only modified asphalt should be used in this case.

4.6.5 The binder of a stress-absorbing membrane interlayer (SAMI) should have good tensile and bonding strength. Rubberized asphalt is popularly adopted in practice. In recent years, the popularity of rubber-asphalt production equipment encourages the application of rubberized asphalt as stress absorbing membrane interlayer. Therefore this edition specifies that rubberized asphalt should be used as the binder of a stress absorbing membrane interface.

4.8.2 In order to drain the infiltrated water out of the pavement structure effectively and promptly, a layer with drainage function may be placed either by extending the width of the layer to be the same as that of subgrade, or by providing an drainage system at pavement edges.

5 Material Design Parameters

5.1.3 and 5.1.4 In this Edition adjustments have been made to the methods of calibration of design parameters for pavement materials. Considering that a certain time is needed for obtaining the testing equipment and for performing the tests, different levels for the determination of design parameters are provided.

Level-A is to determine the material parameters by tests in a laboratory. Laboratory equipment is needed and the cost of design is thus comparatively high. Therefore, it is specified to use Level-A at the stage of construction drawing design. Level-B is to determine the material parameters by using empirical formulae. Currently only an empirical relationship in terms of the dynamic modulus of asphalt mixtures is available. Level-C is to determine the parameters by referring to the typical data values recommended in this edition of the specifications. This level is applicable to all the design stages for Class-2 and lower class highways, or for the preliminary design stage of motorways and Class-1 highways.

5.2.2 The resilient modulus of the subgrade is referred to as that at the equilibrium moisture condition and taking periodical wet-dry and freeze-thaw effects into account.

5.3.6 In addition to the requirements for bearing capacities (such as the CBR value and modulus), the layer of granular material should have good water permeability to provide its drainage function. Therefore, the proportion of the particles smaller than 0.075mm needs to be controlled.

The fine aggregate in crushed stone macadam is usually the material passing the fine sieves, in which the proportion of the particles smaller than 0.075mm is comparatively high and varies over a large range. In the case where the particles smaller than 0.075mm cannot be well-controlled in a design mix, a certain amount of coarse natural sand may be used to replace the chippings so as to reduce the proportion of particles smaller than 0.075mm.

5.3.7 According to relevant researches, the moisture in a granular material layer after completion of construction will progressively reduce until it reaches a state of moisture equilibrium. Referring to the Mechanistic- Empirical Pavement Design Guidelines (MEPDG) developed by AASHTO of U.S., the modulus-moisture adjustment factor is introduced to reflect the effects of moisture condition on the modulus in a granular material layer after opening the highway to traffic.

5.3.8 In the previous edition of these Specifications, design parameters were determined by taking the representative values in terms of guaranteed factors for various highway classifications. In this edition of these Specifications, average values are taken as the design parameters when a performance model is established. Therefore it is stipulated that the average value of actually measured data is taken when test and measurement method is adopted for determining the material parameters of structural layers.

5.4.2 Over dosed cement content would cause shrinkage cracking in chemically stabilized materials, and further cause an increase in the reflection cracking in the pavement surface. Therefore the cement content needs to be strictly controlled.

5.4.4 The influence of structural layer thickness needs to be taken into account in addition to highway classification, traffic load class and structural layer position. Both practice and engineering analysis have confirmed that the risk of fatigue cracking of chemically stabilized layer will be significantly increased if thinner thickness is selected. Therefore, high strength materials shall be used if the designed pavement structure has only 1-2 layers of chemically stabilized materials.

Subject to design requirements, attention shall be paid to the fact that for chemically stabilized materials, higher unconfined compressive strength may imply more shrinkage cracks which in turn may cause more reflection cracking.

5.4.5 In The Research on Multi-Parameter Based Methodology for Asphalt Pavement Structural Design, which was one of MoT technical research projects for transportation infrastructure development in western China, comparisons were conducted between various test methods, such as dynamic compressive resilient modulus, dynamic flexural modulus, EN Compressive Resilient Modulus, Mid-section Method of Uniaxial Compressive Resilient Modulus, and Top Surface Method of Resilient Modulus suggested by the earlier edition of the Specifications. This resulted in the selection of the methods required in these Specifications.

[En version note: Mid-section method refers to the test mothed in which resilient modulus are measured along the height of a test specimen]

5.4.6 In The Research on Multi-Parameter Based Methodology for Asphalt Pavement Structural Design, the laboratory measured resilient modulus was compared with the structural layer modulus back-calculated from deflection basin measured by Falling Weight Deflectometer (FWD) for chemically stabilized materials. It was concluded that the value of the former was double the value of the latter. Therefore, a modulus adjustment factor is introduced to adjust the laboratory resilient modulus to the pavement structural layer modulus.

5.5.4 There were misunderstandings that the larger were the nominal maximum size the higher the rut resistance would be. However, for a particular aggregate material, grading, bitumen content and compaction have far more impact than nominal maximum size on the rutting resistance of an asphalt mixture. Skid resistance is mainly subject to material grading, depth of surface macrotexture, aggregate abrasion and so forth, which has no much relation to nominal maximum size. Furthermore, the larger the nominal maximum size is, the more segregation may occur during construction thus the higher will be the risk of partial damages by water. Taking the above into account, it is stipulated that the nominal maximum size of the top layer of surface course should not be larger than 16.0mm.

5.5.5 Critical fracture temperature indicates not only the effects of temperature stress accumulated in the pavement but the tensile strength of bitumen as well. This indicator is adopted in AASHTO Standard Practice PP42-07 for evaluating low temperature performance of modified asphalt. In this edition of the Specifications, the indicator is used as supplementary to two indicators of low-temperature thermal performance, namely the creep compliance and the slope of creep curve, both of which are determined by bending beam rheometer test.

Critical cracking temperature is obtained by calculating the results of bitumen bending beam rheometer (BBR) tests and direct tension test (DTT). The calculation procedure is that: (1) develop creep stiffness-time curves at different temperature S(t) measured by BBR tests; (2) obtain the Stiffness Master Curve in accordance with the principle of time-temperature equivalency; (3) convert stiffness S(t) to creep compliance D(t), and then further to relaxation modulus E(t); and (4) calculate thermal stress at different temperatures based on relaxation modulus. Then a relation curve of failure strain and failure stress is developed from DTT. The temperature corresponding to the intersection point of the temperature stress curve and the failure strength curve is the bitumen critical cracking temperature T_{cr}. By using the program provided with the test equipment, the temperature stress curve and failure strength curve can be generated automatically, and thus T_{cr} is obtained.

5.5.8 and 5.5.9 specify penetration strength of asphalt mixtures in order to control asphalt surface rutting. In the MoT technical research project on The Loading Standards for Asphalt Pavements, the study and research was conducted to develop a relationship between penetration strength and permanent deformation of asphalt mixtures of different pavement structures under different traffic conditions and in different climatic environments. Based on the study an expression for checking the penetration strength of asphalt mixtures was obtained.

Equation (5.5.8-1) and Equation (5.5.9-1) take the average vehicle speeds into account. On some of the road segments, such as steep and long grade segments, where the average vehicle speed is much different from those on normal road segments, penetration strength of an asphalt mixture may be checked by Equation (5-1) and Equation (5-2). Equation (5-1) is used for an asphalt pavement with either chemically stabilized base course or an asphalt pavement with chemically stabilized subbase and asphalt bound base course; Equation (5-2) is applicable to an asphalt pavement with granular base course and an asphalt pavement with granular subbase course and asphalt bound base course.

Where v_e——Average speed of trucks（km/h）;

Other symbols are defined as in Equation (5.5.8-1) and Equation (5.5.9-1).

5.5.10 Moisture induced damage is one of the major types of distresses in early life of asphalt pavement. Adding hydrated lime, cement or anti-stripping admixture, or treating aggregate by saturated limewater may improve the adhesion between aggregate and bitumen and enhance the resistance against moisture induced damage.

5.5.11 The upper surface method was adopted in previous editions to determine the resilient modulus at temperatures of 15 ℃ or 20 ℃. This could not represent the dependency of modulus to temperature and loading time accurately. The problem is solved by using the dynamic uni-axial compressive modulus as adopted in this edition. Temperature reflects climate conditions in different areas. Loading time can reflect the influence of vehicle speeds and the thickness of asphalt mix.

Loading time is presented by the loading frequency during test. Taking vehicle loading distribution with the depth of pavement into account, for a particular operating speed, the closer to pavement surface, the shorter is the loading time, which in turn means higher frequency. Conversely, the longer the loading time means a lower frequency. Referring to relevant research abroad, 10 Hz loading frequency is adopted for asphalt bound surface course, and 5 Hz for asphalt bound base course in this edition of the Specifications.

The test temperature is taken as 20℃. In calculation and verification of pavement structures, different design indicators and temperature conditions in different regions are reflected by temperature shift factors.

6 CALCULATION AND VERIFICATION OF PAVEMENT STRUCTURES

6.1.2 A number of typical pavement structures compatible to local environment have been developed and are recognized through extensive engineering and construction practice and during a long period of operational experience. For highways with a lower classification and low traffic volumes, a pavement structure may be determined by referring to local experience. However, for highways with a higher classification and high traffic volumes or in the case where an abnormal type of pavement structure is to be used, a performance analysis needs to be conducted to ensure reliability of the pavement structure.

6.2.1 Adjustments have been made to the design indicators and parameters in this edition, including the number of design parameters that have been slightly increased compared to the earlier edition.

6.3.2 Layered modulus, which is the laboratory measured modulus of the material of a structural layer multiplied by a corresponding modulus correction factor, is used in mechanistic calculation and verification of a pavement structure.

According to the correlation analysis, there is little difference between the laboratory tested modulus and field measured modulus of asphalt mixtures; for chemically stabilized materials, the laboratory tested modulus is twice the field measured modulus.

For granular materials, the resilient modulus at moisture equilibrium, which is the modulus under normal conditions multiplied by a moisture adjustment factor, is used.

The modulus at the top of subgrade needs to be adjusted for moisture and corrected in terms of wet-dry and freeze-thaw cycles. Determination of moisture adjustment factor and wet-dry and freeze-thaw reduction factor shall comply with the provisions of the current JTG D30 Design Specification of Highway Subgrade.

6.3.3 Temperature is one of the important external factors affecting pavement performance. Based on the data obtained during a one-year monitoring of pavement temperature and related climatic conditions, which was carried out in seven districts including Guangzhou, Ningbo, Datong, Kumul, Tsitsihar, Zhengjiang and Jinan, and according to the linear partial differential equation of heat conduction and the function of thermal flux on the road surface, a prediction model for pavement temperature based on field and road surface temperature was developed to predict temperatures at different depths in a pavement. According to this research, this edition of the Specifications uses temperature shift factors to reflect the influence of the climatic conditions in different regions on the fatigue cracking in pavement structural layers and the vertical compressive strains, and adopts an equivalent temperature to reflect the influence on permanent deformation in asphalt bound layers.

7 Design of Pavement Rehabilitation

7.1.1 These Specifications concern the design of strengthening for the existing pavement whose structural performance is insufficient. For regular pavement maintenance including distress patching and other small-scale repair activities, reference shall be made to JTJ 073.2 Specification of Maintenance Techniques for Highway Asphalt Pavement.

7.1.2 After a period of trafficking, pavement conditions of various sections in an existing highway could be quite different from one another. Therefore it is necessary for the designers to assess the pavement conditions of the existing highway and to develop suitable solutions for each section. During the design of the pavement rehabilitation, designers shall pay attention to and learn from the previous experience of practice in addition to a theoretical analysis, because of the variety of the factors that influence pavement rehabilitation and the difficulty in terms of performance assessment and the prediction of remaining lives of the pavement sections.

7.1.3 Utilization of the existing pavements, which is one of the most important tasks in pavement rehabilitation design, needs to be assessed and designed to make use of the remaining structural capacity of the existing pavement and to minimize un-necessary stripping or milling of the existing pavement materials. Reclaimed materials shall be reused in a proactive and proper way, or repaved by appropriate technology.

7.1.4 Traffic management during pavement rehabilitation has a significant impact on construction activities and traffic safety. Therefore, designers need to ensure that the traffic management during the construction period and relevant safety devices have been effectively planned and designed.

7.1.5 Due to complexity of the existing pavement conditions, the data derived in design stage may not be able to reflect the site conditions accurately and adequately. These data need to be validated and updated during construction. The real conditions of each road section shall be further investigated in detail and the situations of the existing pavement shall be reviewed and double checked. Wherever the actual site conditions differ from the data collected at the design stage, the design of the pavement rehabilitation for the particular section shall be adjusted accordingly.

7.2.1 Investigation of the existing pavements are to assess the structural performance and material properties of the existing pavement structural layers, identify the root causes of pavement distress, and consider possible technical solutions to eliminate distress or mitigate deterioration. Due to the variety and complexity of real situations of the existing pavements, it is difficult or even impossible to provide standardized methodologies of investigations and assessment for the existing pavements all over different areas. Therefore, this clause only covers major activities that should be carried out for a pavement investigation. Details of the investigation shall be planned and determined in accordance with the conditions of specific project.

7.2.2 The ratio of longitudinal cracks is the length of longitudinal cracks to the length of the traffic lane. The percentage area of crocodile cracking or the ratio of patched area is the percentage of the enveloped rectangular area of the crocodile cracking or the patches respectively to the area of the traffic lane. These indicators need to be measured and calculated in sections and by each of lanes.

7.2.3 The cause of pavement distress, the layer position where distress occurs, the damage accumulation, the damage progress and the reutilization of remaining capacity of the existing pavement are the basic factors for determining the methods of treatment of the existing pavement. The designers shall analyze the causes and development trend of the pavement distress and systematically analyze and identify the layer position and accumulation of the distress in accordance with the pavement conditions and relevant influencing factors, and then determine whether the existing pavement structural layers can be reutilized and how to be reutilized.

7.3.2 A rehabilitation design comprises the design of the existing pavement treatment and the design of overlaying. Partial treatment, which is the treatment that is only applied wherever necessary in terms of positions and types of distress, may be adopted for the pavement with no structural damage but limited pavement distress. In cases where the occurrence of distress is high, the workload of spot patching would be high and the resulting pavement performance may be reduced significantly if only partial treatment is adopted. Consequently, where pavement structural damage occurs over a rather long section, the treatment over the whole section should be adopted. Major methods of the treatment over a full section include but ardc not limited to applying a thick asphalt overlay or overlays directly on the top of the existing pavement, overlaying after milling the existing pavement to the damaged structural layer or overlaying after in-place reclamation of the existing pavement materials.

Table 7-1 shows the statistics in relation to the methodology of treatment of a full pavement section. The wholly treatment may be adopted as long as the real situation falls in the range of any one of the indicators.

Table 7-1 The pavement condition situations to be treated in full

Code	Indicator	Range
1	Percentage of pavement damage (%)	⩾10
2	Spacing between cracks (m)	⩽15
3	Percentage of crocodile cracked area (%)	⩾10
4	Percentage of patched area (%)	⩾10
5	Deflection of pavement	Greater than the critical value of deflection

In Table 7-1, critical deflection values are used for assessing whether or not the existing pavement has been structurally damaged. These critical values of deflection are to be determined in such a way that： firstly deflections are measured on the existing pavement surface and sample cores drilled at the spots where deflection is measured; secondly the relationship between strength and pavement surface deflection is established based on the integrity of the sample cores and deflection to derive the critical deflection value at failure of the corresponding pavement structural layer. However, only a rough relationship can be established between the critical deflection values and the structural failure of the pavement. At this stage, designers may only be able to roughly identify which pavement sections need to be treated in full. A further decision may need to be made during the construction stage based on field coring and according to integrity or strength of the sample cores to confirm whether the pavement has been structurally damaged.

7.3.4 Reflection cracking is one of the common problems to be solved by rehabilitation. In the case that a number of reflection cracks exist in the existing pavement, effective crack control measures, such as increasing the thickness of the structural layers or adding a rubberized asphalt stress absorbing layer, may be considered to mitigate and retard reflection cracking.

7.3.5 Failure or inadequacy of a pavement drainage system may cause poor subsurface drainage or even water damage leading to pumping, raveling, potholes, etc. In such cases, the replacement or improvement of the existing pavement drainage system shall be included in the rehabilitation design.

It is difficult to form an effective interface bond between the newly paved overlay and the existing pavement, due to the differing properties of the materials and the influence of construction quality. Therefore, the designers shall pay attention to the design of tack coats and seal coats between overlays and the existing pavement.

7.4.1 The conditions for traffic parameter investigation and analysis for a rehabilitation design are much better than those for a new construction design. Accurate data on traffic loading parameters can be derived from tolling-by-weight systems supplemented with site observations.

7.4.2 The pavements after rehabilitation shall comply with the same performance requirements as those for newly constructed pavements.

7.4.3 and 7.4.4 Whether or not a structural verification needs to be executed on the existing pavement shall be determined in accordance with the damage accumulation and overlay design.

For pavement sections with limited damage and acceptable structural performance, utilization of remaining capacity of the existing pavement shall be taken into account in the design of the pavement rehabilitation based on the assessment that no fatigue failure would occur in the existing pavement structure during the remaining structural design life. Structural verification shall be executed on both of the existing pavement layers and the overlays no matter whether direct overlaying, overlaying after milling or overlaying after in-place recycling is adopted as the method of treatment. Design parameters including modulus of each structural layer of the existing pavement shall be determined by back-calculation of the deflection basin data or by measuring sampled cores.

In the case where the pavement is badly damaged and structural strength is too low to insure that the existing pavement structure will remain in an acceptable condition during the design life, the existing pavement structure and the subgrade can be regarded as a semi-infinite half-space, the equivalent resilient modulus on the surface shall be used for structural design of the overlay. Structural verification may be executed only on overlays no matter whether direct overlaying or overlaying after either milling the existing pavement down to a structural layer or in-situ recycling the existing pavement is adopted as the treatment method.

Chemically stabilized materials have a characteristic property that strength will increase over the long term. Where the structural strength of the existing pavement is measured from cores taken out of the pavement, it might be the case that the strength of the cores is higher than design strength. In these specifications, the relevant performance model is based on the material parameters at an early stage after the pavement completion and the sample core strength of a chemically stabilized layer is used for structural verifications. Therefore there may be errors, and the strength of each structural layer shall be reduced in accordance with the number of repeated traffic loads and the state of failure. Reduced strengths should not exceed the criteria of Clause 5.4. In earlier designs, the ratio of the strength in the traveled way to that in the shoulder were used as strength reduction factor, which may be taken for reference.

8 Design of Bridge Deck Pavement

8.2.1 Surface pre-treatment by milling or abrasive blasting may remove laitance and the soft membrane on the concrete surface, improve the effects of water resistance and increase the interlayer bond strength. The required texture depth after deck surface pre-treatment is to ensure high shearing strength and bond strength between the asphalt layer and the bridge deck.

8.2.6 The sand asphalt mixture is normally used as a lower layer in a bridge deck pavement to provide waterproofing, to serve as a leveling layer and to perform fatigue resistance functions.

8.2.8 The asphalt mixture in the areas abutting curbs, barriers and other objects is difficult to be compacted well. On one hand, these places may be susceptible to water infiltration thus sealing treatment is required. On the other hand, these areas are located at the lower part of road cross slopes which may impede interlayer water from flowing, thus suitable drainage is required.

8.2.9 Longitudinal blind trenches along the edges of deck pavement improve inter-layer drainage and reduce the risk of water damage. The designers need to pay attention to the invert level of bridge weep holes that must be lower than the bottom of deck pavement.

8.3.2 In a steel bridge deck system, the materials used for waterproofing materials usually are those compatible with the deck pavement structure. For instance, an epoxy asphalt or epoxy resin is used as the waterproofing material in an epoxy asphalt concrete pavement, and rubberized asphalt emulsion is commonly used as the waterproofing material in a cast in-situ concrete deck pavement. Therefore, this edition of the Specifications requires the compatibility of selected waterproofing material to the material of the deck pavement.

Appendix A: Parameter Analysis of Traffic Loadings

A.1 Vehicle Classification

A.1.1 and A.1.2 In the previous edition, a representative vehicle approach was adopted for the traffic composition. Mixed traffic was categorized into only 5 or 6 types of representative vehicles, which was found to be rather inadequate for considering the impact of vehicle composition and over-loading on pavements.

In this Edition of the Specifications, traffic classification has been extended to11 vehicle classes in terms of vehicle configurations, axle groups and resulting damage effects to pavements, and axle load spectrum has been introduced to identify load distributions in different axle load ranges so as to analyze traffic parameters in a more accurate way.

A vehicle type is defined by axle configuration of the vehicle. For instance, type-15 truck is referred to as a truck with an axle code 1 front axle and an axle code 5 rear axle. Due to the low popularity, axle code 3(single tire tandem axle), axle code 4(tandem axles, one fitted with single tires and the other fitted with dual tires) and axle code 6 (single tire tridem axles) are merged into axle code 5 (dual tire tandem axles) and axle code 7 (dual tire tridem axles). As far as vehicle classes are concerned, Class-1 vehicles include small passenger cars or small trucks with light payload that have limited damaging effects on pavements and are therefore excluded for highway pavement design. Class-2 is a large bus, which have a damaging effect on highway pavements and thus need to be considered in pavement design. Other classes are all trucks with significant impacts on highway pavements. Apart from Class-1, Class-2 to11 vehicles are generally referred to as large buses and trucks.

A.2 Traffic Data

A.2.5 Lane distribution factor is the proportion of large buses and trucks on the design lane to the traffic volume of large buses and trucks in the traffic direction. The traffic volume on the design lane is the product of the sectional traffic volume multiplied by directional distribution factor and lane distribution factor respectively.

A.2.6 Vehicle type distribution factor, one of the important parameters reflecting traffic composition, is defined as the volume of a specific vehicle class to the total volume of vehicles in class-2 ~ class-11. Truck traffic classification (TTC) reflect the percentage of trucks or articulated trailers to the total traffic volume. AASHTO Mechanistic-Empirical Pavement Design Guide (MEPDG) classifies highway traffic composition into 17 groups. In contrast to the situations and statistical data in China, the TTC approach could be quite complicated to use in practice. Therefore, in this Edition of the Specifications, TTC groups have been simplified and reduced to 5 groups, and a corresponding vehicle type distribution factor is assigned to each TTC group.

A.3 Vehicle Conversion to Equivalent Design Axle Loads

A.3.1 Axle load conversion parameters include axle factors, tire factors and conversion factors, which are directly affected by pavement design parameters and performance models. Based on the parameter system and performance model adopted in this edition of the Specifications, axle factors, tire factors and conversion factors were developed through a large project of analytical calculations on typical structures.

To identify whether a vehicle is partially loaded or fully loaded depends on the standard total weight of the vehicle. Any vehicle equal to or less than the standard total weight is identified as a partially loaded vehicle, otherwise it is a fully-loaded one. The vehicle standard weights are as follows:

For the vehicle standard total weight given above, dual tires are used for all axles except the steering axle of two or three axle vehicles, and semi-trailers or trailers. In the cases of single tire axles, the limiting standards are reduced by 30kN; if the maximum permitted total weight of a vehicle exceeds the sum of the maximum permitted axle weights, the sum of all the maximum permitted axle weights of the vehicle shall be taken as the identification criterion.

As far as buses are concerned, most of the tolling systems in China do not toll bus axle weights but take the number of seats into account. Therefore, it has been accepted that buses with 39 seats or less are regarded as partially loaded vehicles, while those with more than 39 seats are fully loaded vehicles.

The proportions of partially loaded and fully loaded vehicles in Table A.3.1-2 and the load equivalency factors for either partially loaded or fully loaded vehicles in Table A.3.1-3 were derived by the statistical analysis of traffic loading parameters from 44 road cross-sections scattered all over the country.

APPENDIX B METHOD OF CALCULATION AND VERIFICATION OF PAVEMENT STRUCTURES

Freeze-thaw cycling in seasonal frost areas will cause additional damage to a pavement and thus reduce the pavement fatigue life. Taking this into account, an adjustment factor for seasonal frost areas is introduced into two fatigue cracking models, one for asphalt bound layers and another for chemically stabilized layers, in this edition of the Specifications.

According to researches, constant strain loading mode of fatigue cracking model is suitable for thinner layers of asphalt bound materials, while constant stress loading mode of fatigue cracking model is more suitable for a thicker layer of asphalt bound materials. For the asphalt bound layers with an intermediate thickness, a transition relation needs to be established between the two modes, thus a loading mode factor k_b for fatigue cracking is introduced in this edition of the Specifications for transition and conversion between different loading modes.

B.2.1 The fatigue cracking model of chemically stabilized granular materials and soils was formulated based on the results of 148 tests on the fatigue cracking in four different materials, including cement stabilized gravels, cement stabilized crushed stone, cement stabilized soil and lime-flyash stabilized crushed stone.

Validation of the fatigue cracking model of chemically stabilized layers was difficult due to a lack of field data. Therefore a large number of investigations have been conducted and the typical asphalt pavement structures were identified for different operational environments in terms of the factors such as highway classification, traffic load parameters and resilient modulus of the subgrade. Using the failure states of these typical pavement structures to calibrate the analytical results of fatigue cracking models discussed above, an overall field calibration factor, k_c , was introduced and used for reflecting the difference between the laboratory performance model and on-site fatigue cracking failure.

B.3.2 Based on 229 effective results of the rutting tests conducted under different temperatures and different pressures on various asphalt mixtures, a model for estimating permanent deformation in asphalt mixtures was developed. This model accounts for various parameters such as the number of repetitions of traffic loads, temperature, vertical stress, layer thickness and magnitude of permanent deformation. The model has been further corrected and verified by using the multi-year rut data collected from more than 10 highways and 5 trial sections.

In the light of the stress distribution with pavement depth and the fact that different asphalt bound layers may have different rutting performance, it is required to compute permanent deformation in layers. The difference between the cumulative permanent deformation in each asphalt layer and the overall permanent deformation of the asphalt layer as a whole is taken into account by the overall correction factor k_R.

The needs for of pavement rehabilitation during the design life shall be carefully considered in a structural analysis. A highway pavement carrying a large traffic volume and a high proportion of heavy vehicles may need to be rehabilitated to restore rutting once or several times during its design life. In such a case, N_{e3} may be the cumulative number of design axle loads during the period from the beginning of operation to the first rehabilitation to restore rutting.

Equation (B.3.2-1) assumes that the pavement voids on site and the voids of the specimen used for rutting test are similar, otherwise Equation (B-1) should be used for the prediction of permanent deformation in asphalt bound layers.

$$R_a = \sum_{i=1}^{n} R_{ai}$$

$$R_{ai} = 2.31 \times 10^{-8} k_{Ri} T^{2.93} p_i^{1.80} N_{e3}^{0.48} \left(\frac{V}{V_0}\right)^{0.83} (h_i/h_0) R_{0i}$$

（B-1）

Where V ——initial void content (%) of the asphalt bound layers at the time of construction;

V_0 ——void content of the specimen used for the rutting tests.

Other symbols have the same meaning as defined for Equation (B.3.2-1).

B.4.1 The vertical compressive strains at the top of subgrade is one of the important design indicators for both of the asphalt pavement with granular base course and the asphalt pavement with a granular subbase course and an asphalt bound base course. This indicator is introduced in this edition of the Specifications. Design methods adopted in other countries usually were to control vertical compressive strains so as to prevent excessive permanent deformation in subgrade, while the relationship of vertical compressive strains and traffic load parameters are simulated based on the data collected from trial sections or by site observation. However, an asphalt pavement with granular base course is not yet popular in China, and thus relevant field measured data are limited. Therefore, reference has been made to the information from the AASHTO trial roads, including 195 pavement structures and the number of axle load repetitions until the present serviceability index (PSI) reached 2.5. Vertical compressive strains at the top of subgrade in different structural types were derived by back-calculations, and an empirical relationship of vertical strains at the top of subgrade to the number of repeatition of 100kN axle loads was developed. After necessary revisions and corrections, the model is finally established.

B.5.1 Low temperature thermal cracking is one of the commen pavement distresses in seasonal frost areas. By using empirical methods, more than 10 highway sections in north-east China were investigated to identify and analyse the relationship of asphalt properties, pavement structure, subgrade soil types and other factors in terms of the pavement low temperature thermal cracking performance. Reference was also made to the Canadian Haas model. Based on the both above, a prediction model for pavement low temperature thermal cracking was developed.

B.6.1 Thermo-physical coefficient of each subgrade or pavement layer within the range of ground frost penetration shall be calculated by using the weighted average of layer thicknesses.

B.7.1 and B.7.2 Equation (B.7.1) is developed in accordance with the theory of vertical displacement on top of the elastic half-space under a single circular load, which is applicable to the situations of untreated subgrade fill materials. In the case where granular materials or chemically stabilized materials are used as subgrade improvement layers, the deflection value for acceptance shall be determined by analytical results and practical experience, i.e., in the light of the layer structure of the subgrade and in accordance with the multi-layered elastic system theory.

In the calculation of deflection values at the top of subgrade for acceptance, the equivalent resilient modulus of the subgrade top in moisture equilibrium status shall be used. That is, not only a moisture adjustment factor is used but a modulus reduction factor of dry-wet and freeze-thaw cycling is taken into account. In the case where differences between subgrade moisture and equilibrium moisture exist, moisture correction is required.

B.7.3 and B.7.4 In the calculation of deflection on top of the road surface for acceptance control, the equivalent resilient modulus in moisture equilibrium multiplied by a modulus adjustment factor k_l is used. The factor k_l is used for adjusting the difference between calculated deflections and field measured deflections.

For calculating the deflection values on top of a reconstructed pavement, different reconstruction proposals shall be taken into account in the selection of the subgrade modulus adjustment factor. For the existing road segments on which damage to the pavement is limited and the structure is in an acceptable condition, k_l takes the value of 1.0 to determine the subgrade modulus by back-calculation in layers; in the case where the pavement has been seriously damaged or its structural strength is obviously insufficient, Equation (7.4.4) is used to determine the equivalent resilient modulus either on existing road surface, or on a specific layer after pavement milling. If the overlays are constructed with a chemically stabilized layer or a cement concrete layer, the factor k_l shall be 0.5, otherwise k_l shall take a value of 1.0.

The dynamic compression modulus shall be used for asphalt bound material layers to which a temperature correction should be applied in accordance with pavement temperature conditions when deflections are measured.

APPENDIX C: ASPHALT PAVEMENT STRUCTURES

C.0.1 In 2011 a nationwide investigation was conducted to identify the popular pavement structures adopted in China to evaluate the practical experience and performance of these pavement structures. It was found that the pavement structure popularly used for an asphalt pavement was one with a chemically stabilized base and the other was with a chemically stabilized subbase and an asphalt bound base. The typical pavements are summarized in Tables C.0.1-1, C.0.1-2 and C.0.1-5 respectively.

Two other types of asphalt pavements, one with unbound granular base and the other with an unbound granular subbase and an asphalt bound base, are not popular in China but widely used in other countries. Pavement structural data listed in Tables C.0.1-3 and C.0.1-4 are mainly based on foreign practice or on the researches and experience of other countries.

A pavement that has a graded macadam layer placed between an asphalt bound base and chemically stabilized subbase has been used in Fujian and other provinces and has been found effective in reducing pavement cracking.

APPENDIX D STANDARD METHOD OF TEST FOR MEASURING THE RESILIENT MODULUS OF UNBOUND GRANULAR MATERIAL

D.5.2 Unbound granular materials have a significant stress dependency. In the Research on Design Indicators and Parameters for Asphalt Pavements under the MoT technical research program for Western China Transportation Development, various generalized constitutive models for resilient modulus of granular materials were reviewed with actual results. It was found that the three-parameter model which was recommended by Laboratory Determination of Resilient Modulus for Flexible Pavement Design (NCHRP 1-28) developed by the National Cooperative Highway Research Program of U.S.A, provided a better explanation of the characteristics of stress dependency of granular materials, and was thus adopted by these Specifications.

APPENDIX G TEMPERATURE FACTORS AND EQUIVALENT TEMPERATURE

G.1.1 The determination of temperature shift factors and equivalent temperature is done in two steps: firstly determining the temperature shift factors and the equivalent temperature of a reference pavement structure by referring to historic statistical data of temperature records and Table G.1.2; and secondly conducting structural thickness and modulus corrections, so as to obtain the temperature shift factors and equivalent temperatures corresponding to pavements in different structures. A reference pavement structure includes two types of structures: one is the asphalt surface course on a granular base course; the other is asphalt surface course on a chemically stabilized base course. For the reference pavement structure, the thickness of asphalt surface course h_a = 180mm; the thickness of granular base course h_b=400mm. The dynamic modulus of asphalt mixture E_a = 8000MPa; the resilient modulus of granular material E_b=400MPa; the resilient modulus of chemically stabilized material E_b=7000MPa; and the resilient modulus of subgrade E_0 = 100MPa.

G.1.2 ～ G.2.1 The temperature shift factor and reference equivalent temperature in a particular region may be either obtained by referring to Table G.1.2 or calculated by Equation (G-1) ～ Equation (G-3) using the average temperature in the hottest month and the average temperature in the coldest month, both of which are based on temperature records for last successive 10 years collected by local meteorological sectors.

Temperature shift factors are computed by Equation (G-1) and Equation (G-2).

$$\hat{k}_{Ti} = a_i x^2 + b_i x + c_i \tag{G-1}$$

$$x = \mu T_{\mathrm{a}} + d_i \Delta T_{\mathrm{a.mon}} \tag{G-2}$$

Where, temperature shift factor for reference pavement structure; subscript i=1 corresponds to the analysis of fatigue cracking in asphalt bound layer, i=2 corresponds to the analysis of fatigue cracking in chemically stabilized layers, i=3 corresponds to the analysis of vertical compressive strain on top surface of subgrade.

μT_a——annual average temperature (℃) in the region;

$\Delta T_{a.mon}$——annual maximum temperature difference of the region, which is the difference between the average temperatures in the hottest month and that in the coldest month.

a、b、c、d—— regression parameters used for calculation and retification, taken from Table G-1.

Table G-1 Regression Parameters

Design Indicators	a	B	c	d
Tensile strain at the bottom of the asphalt layer Tensile stress at the bottom of the chemically stabilized layer	0.0006	0.027	0.71	0.05
Compressive strain at top surface of subgrade	0.0013	0.003	0.73	0.08

Reference equivalent temperature is calculated by using Equation (G-3).

$$T_{\xi} = 1.04\mu T_a + 0.22\Delta T_{a,mon} \tag{G-3}$$

Where symbols are defined the same as Equations (G-1) and (G-2).

Technical Terms in Chinese and English

序号	英文词汇	
1.	Asphalt mixture	
2.	Asphalt pavement	
3.	Asphalt pavement construction	
4.	Asphalt pavement recycling	
5.	Asphaltic concrete pavement	
6.	Axle Load	
7.	Axle load spectrum	
8.	Base course	
9.	Bedding course	
10.	Binder Course	
11.	Bitumen Macadam, Bituminous macadam	
12.	Bitumen (bituminous) penetration macadam	
13.	Cape seal	
14.	Cement bound macadam	
15.	Chemically stabilized materials	
16.	Cold Laid (cold placed) asphalt	
17.	Cold mix asphalt	

18.	Damp	
19.	Deflection bowl	
20.	Directional distribution factor	
	E	
21.	Empirical values	经验值
22.	Equivalent single axle load (ESAL)	
23.	Fatigue life, number of repetitions to fatigue cracking	
24.	Frost protection layer, insulation layer	
25.	Full depth asphalt pavement	
26.	Hot Mix, Hot mix asphalt (HMA)	
27.	Hot rolled asphalt	
28.	Influence zone, zone of influence (of vehicle loads)	
29.	Intermediate course	
	L	
30.	Lane distribution factor	车道系数
31.	Low temperature thermal cracking	
32.	Moist	
33.	Moisture resistance	

34.	Nominal maximum aggregate size (NMAS)	
35.	Overlay	
36.	Pavement	
37.	Pavement design life	
38.	Pavement layer	
39.	Pavement strengthening	
40.	Pavement structure	
41.	Penetration macadam with plant mix overlay	
42.	Plant mix	
43.	Prime Coat	
44.	Rebound deflection	
45.	Reclaimed asphalt pavement (RAP)	
46.	Regulating course	
47.	Rigid base	
48.	Road mix (mixed-in-place)	
49.	Roughness	
50.	Seal Coat	
51.	Sealing layer	

52.	Sideway-Force Coefficient	
53.	Single-axle, dual-tire load	
54.	Slag	
55.	Slag base	
56.	Slurry seal	
57.	Stabilized soil base	
58.	Standard axle load(SAL)	
59.	Stone-matrix asphalt (SMA)	
60.	Structural design of pavement	
61.	Structural type of pavement	
62.	Subbase	
63.	Subgrade moisture conditions	
64.	Surface Course	
65.	Surface Dressing	
66.	Surface macro-texture	
67.	Surface micro-texture	
68.	Surface Texture	
69.	Tack coat	
70.	Temperature shift factors	
71.	Tolerable deflection	
72.	Traffic Growth Factors	
73.	Truck type	

74.	Wearing course	